Structural Analysis of Latticed Steel Transmission Towers Subjected to Nondeterministic Wind Loads

Mariana Souza Rechtman
Civil Engineering Postgraduate Programme (PGECIV), State University of Rio de Janeiro (UERJ), Brazil
mariana.rechtman@gmail.com

José Guilherme Santos da Silva
Structures and Foundations Department (ESTR), State University of Rio de Janeiro (UERJ), Brazil
jgss@uerj.br (corresponding author)

Received: 13 September 2024 | Revised: 7 October 2024 | Accepted: 9 October 2024

 DOI: https://doi.org/10.48084/etasr.8990

ABSTRACT

Lattice steel towers are commonly used to support overhead power transmission lines. However, the dynamic behavior of these structures is often overlooked in current design practices. Given that many accidents involving these towers occur even at basic wind speeds lower than those specified in the project, it is likely that dynamic actions play a significant role in these failures. This study proposes a method to accurately simulate the interaction between transmission line cables and towers under non-deterministic wind loads to assess displacements and forces in the steel towers. The study examines a transmission line system consisting of towers, conductors, shield wires, and insulators, featuring a central suspension tower of 32.86 m in height, flanked by two end towers with 450 m spans. Finite element modeling was developed to account for the dynamic characteristics of the wind. Wind loads were modeled as a random process based on their statistical properties. The results revealed significant differences in displacement and force values when comparing the results provided by static and dynamic analyses. The structural design of a base leg member indicated potential failure at higher wind velocities, highlighting the importance of considering the wind dynamic effects in the design.

Keywords-latticed steel towers; power transmission lines; nondeterministic wind loads; dynamic analysis

I. INTRODUCTION

The growing demand for electricity in Brazil has motivated the implementation of numerous transmission lines in recent decades. [1]. According to the National Electric System Operator (ONS), currently, the National Interconnected System, which is responsible for the generation and distribution of electricity in the country, has 171,640 km of transmission lines with the forecast of reaching 200,015 km in 2028 [2].

Lattice steel towers are widely used and have become essential elements for power transmission systems, due to their advantageous behavior, low self-weight, and ease of transportation and assembly [3, 4]. These towers are usually designed using a spatial truss structural model of the isolated tower, with linear static analysis being the standard approach. This approach does not consider second-order effects resulting from structural displacements. Regarding applied loads, aside from towers' self-weight, equivalent static loads are applied to represent non-modeled components, such as conductor cables, shield wires, insulators, and wind forces [5]. It is well-known that geometric nonlinear analysis leads to increased structural displacements and member forces in addition to those obtained by linear analysis [6]. Transmission towers are one piece of a system that includes multiple towers, insulators, conductor cables, and shield wires [7]. Each element has distinct dynamic characteristics and can be subjected to distinct loads over time. However, in most tower designs, the wind is typically treated as an equivalent static load, overlooking its dynamic nature, although the wind is the primary load acting on transmission towers in Brazil [8].

The use of more resistant steel materials in recent decades has allowed the construction of taller and more slender towers with lower natural frequencies, making them more vulnerable to wind effects. Designing these structures poses challenges, particularly in accounting for wind and dynamic effects, which can lead to collapse. Despite extensive experience in analyzing steel towers for power transmission lines, there have been instances of tower failures even without exceeding the design wind speeds. This suggests that dynamic loads may play a role in these incidents, leading to significant disruptions and financial losses due to power supply interruptions [9].

Having in mind a more accurate assessment of a transmission line, it is very important to analyze the effects of time-varying dynamic loads. The Spectral Representation Method (SRM) achieves this by generating wind series that simulate natural wind behavior, using harmonics with randomly generated phase angles and calculating their amplitude with a power spectrum and coherence function [10]. Damping energy losses in structural systems are essential for dynamic analysis, as they reduce movement amplitudes by dissipating mechanical energy. For transmission line structures, damping comes primarily from structural and aerodynamic sources. Accurately assessing damping is challenging, but while the viscous damping constant can account for various damping effects, aerodynamic damping is more accurately modeled considering the relative velocity of air in relation to the structure's movement [11].

Based on the relevance of nonlinear dynamic analysis and appropriate modeling of power transmission systems, this work focuses on examining the behavior of a transmission line section when subjected to non-deterministic wind loads, aiming to demonstrate the significance of the effect of the dynamic interaction of the cables and steel towers on the structural project of transmission lines to alert the designers about the potential for excessive vibrations that may cause structural failures. Therefore, the main objective of this study is to develop a method for nonlinear and non-deterministic dynamic structural analysis of lattice steel towers. This approach aims to assess the displacements and member forces in a suspension tower, comparing the results with the expected values from current design practices. Therefore, a transmission line section, comprising a central suspension steel tower and two spans of 450 m was analyzed based on three developed analysis methods (see Table I) and considering seven basic wind velocities, the generation of a series of thirty wind loads, and including the effects of structural and aerodynamic damping. The results were subjected to appropriate statistical treatment [means (μ), standard deviations (σ), and reliability indexes ($X_{95\%}$)] to analyze the tower response based on the evaluation of displacements and compression forces.

TABLE I. DEVELOPED STRUCTURAL ANALYSIS

Model	Structural Model	Wind Loads	Structural Analysis
I	Isolated steel tower	Equivalent static [12]	Linear static
II	Transmission line system	Equivalent static [12]	Geometric nonlinear static
III	Transmission line system	Nondeterministic loads	Geometric nonlinear dynamic

II. NONDETERMINISTIC WIND LOADS

Wind loading is unstable and random, with sudden velocity variations known as gusts. To account for the uncertainties, it is necessary to use statistical models. In this research, the wind load is divided into two parts: the mean component (static load) and the fluctuating component (non-deterministic dynamic load) [13]. The mean wind component is determined using meteorological data or, if unavailable, by referring to the isopleths provided in NBR-6123 [12]. This standard offers wind velocity curves (V_0) for Brazil based on 3-second measurements at 10 m height with a 50-year recurrence period (2% annual probability). Design velocity is typically defined as the mean wind speed over 10 minutes. Equation (1) calculates the mean wind velocity ($\bar{V}$), for a height of 10 m above ground. The parameters for the S_2 factor are obtained in (2) with a terrain category II. Using these parameters in (1), it is possible to determine the mean wind velocity at 10 m height ($\bar{V}_0$), as shown in (3). The mean wind velocity at any height z ($\bar{V}_z$) is given by (4). Parameters S_1, S_2, and S_3 refer to topographic, terrain, and statistical factors, respectively. The variables p, b, and $F_{r,II}$ refer to the exponent of the potential law of S_2 variation, the meteorological parameter, and the gust factor, respectively.

$$\bar{V} = V_0\, S_1\, S_2\, S_3 \tag{1}$$

$$S_2 = bF_{r,II}\left(\frac{z}{10}\right)^p \tag{2}$$

$$\bar{V}_0 = 0.69 \cdot V_0\, S_1\, S_3 \tag{3}$$

$$\bar{V}_z = \bar{V}_0\left(\frac{z}{z_0}\right)^p \tag{4}$$

The fluctuating wind component presents a non-deterministic characteristic and is simulated based on a random, ergodic, weakly stationary, second-order process, through the Spectral Representation Method (SRM), based on the sum of a finite number of superposed harmonics with random phase angles. The structural system of a transmission line section is large enough that the uniformity of the wind loads cannot be guaranteed. Therefore, it is necessary to account for these variations through temporal and spatial correlation functions. Temporal functions are generated as random series lagged off a τ time interval, calculated using cross-covariance and autocovariance functions, which describe the interdependence of wind velocity fluctuations at different spatial points. As a result, the spatially correlated temporal functions are represented by the same series, with a time interval τ between them, defined for a fixed dimension range (ΔL), in which the section is subdivided. In this way, (5) represents the turbulent part of wind velocity $v(t)$, where N corresponds to the number of power spectrum divisions, f is the frequency in Hz, Δf is the frequency increment, θ is the random phase angle distributed in the range of [0-2π], t is the time in s, S^V is the spectral density of the wind's turbulent longitudinal component in m²/s, and f_n is the multiple of time lag τ in s. The amplitude of the harmonics can be defined according to (6).

$$v(t) = \sum_{i=1}^{N}\sqrt{2S^V(f_i)\Delta f}\;\cos(2\pi f_i\,(t+\tau_n)+\theta_i) \tag{5}$$

$$a_i = \sqrt{2S^V(f_i)\Delta f} \tag{6}$$

The power spectrum adopted in this study was the Kaimal spectrum, defined in (7) and (8), where X is a dimensionless frequency. The friction velocity u_* is given in m/s through (9), where k is the Kármán constant (approximately 0.4) and z_0 is the roughness length in m.

$$\frac{fS^V(f,z)}{u_*^{\,2}} = \frac{200X}{(1+50X)^{5/3}} \tag{7}$$

$$X(f,z) = \frac{fz}{\bar{V}(z)} \tag{8}$$

$$u_* = \frac{k\bar{V}(z)}{ln(z/z0)} \tag{9}$$

After generating temporal functions, the Davenport model is applied, where the wind pressure q is proportional to the wind velocity (10). Dynamic force $F(t)$ is calculated using (11), where C_a is the drag coefficient and A is the frontal area of the surface. The non-deterministic dynamic wind load over time is finally obtained through (12).

$$q(t) = 0.613\,[\bar{V} + v(t)\,]^2 \quad (10)$$

$$F(t) = C_{ai} q(t)\, A_i \quad (11)$$

$$F(t) = 0.613\, C_{ai}\, A_i \left[\bar{V}_0 \left(\frac{z}{z_0}\right)^p + \sum_{i=1}^{N} \sqrt{2S^V(f_i)\Delta f}\ \cos(2\pi f_i\,(t + \tau_n) + \theta_i)\right]^2 \quad (12)$$

III. INVESTIGATED STRUCTURAL MODEL

This study examined a 900-m section of a power transmission system that includes steel towers, insulators, conductors, and shield wires, with two spans of 450 m each (see Figure 1). The central structure is a delta-type suspension tower (main tower), standing 32.86 m high, with additional towers at each end of the investigated section (end towers). The main tower has a rectangular base, a pyramidal body, and a hollow top, where the phases and shield wires are attached. The tower project is constructed based on the use of equal leg angle profiles, made of ASTM A36 steel. The conductor and shield wires are Grosbeak type (one cable per phase) and 3/8" EHS, respectively.

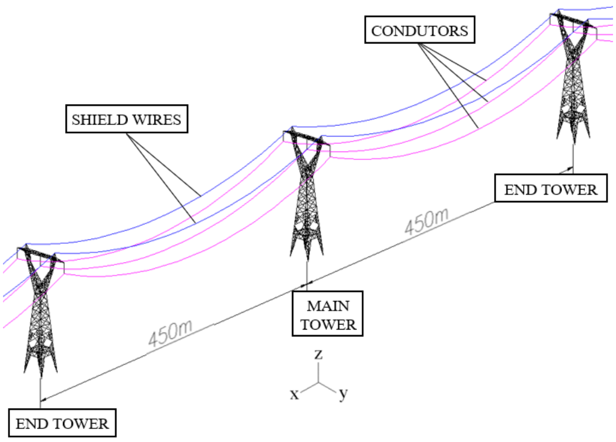


Fig. 1. Investigated transmission line section: two spans of 450 m.

IV. FINITE ELEMENT MODELLING

The 900 m section of the power transmission line was investigated and modeled based on the use of the Finite Element Method (FEM) utilizing ANSYS software, considering the development of three different structural models (Table I). The analyzed transmission line section was represented with one detailed central steel tower (main tower), insulators, conductors, shield wires, and two simplified towers at each end of the section (end towers). Linear springs were added to the cable ends to represent the continuity of the system [1]. The developed FEM was composed of 1,319 nodes and 3,207 degrees of freedom, based on the use of several finite elements (Figure 1 and Table II).

The three-dimensional beam finite elements were used to represent the cables to ensure stability, keeping in mind the modeling complexity due to the cable's low stiffness against bending and compression forces. The stability of the numerical solution can be compromised when using elements of very low or zero stiffness. The tower's members were modeled using three-dimensional beam elements aiming to enable the analysis, having in mind that the use of truss elements results in mathematical problems related to the coplanar joints, since these elements do not have, mathematically, any stiffness in the direction perpendicular to the plane (at least in a first-order linear analysis). Regarding the insulators, the three-dimensional truss elements accurately modeled these elements considering only axial forces, better representing their real structural behavior. The numerical model utilized a substructuring technique to replicate the elastic, inertial, and kinematic properties of the end towers, which involved condensing a group of finite elements into a single matrix element, referred to as a superelement [1]. The boundary conditions were applied to the nodes representing the tower foundations, restricting translational displacements along the three global axes.

TABLE II. FINITE ELEMENTS UTILIZED IN MODELING

Item	Element	Quantity
Main tower	BEAM188	900
Conductor	BEAM 189	48
Shield-wires	BEAM 189	32
Insulators	LINK 180	9
End towers	BEAM 188	18
Liner springs	COMBIN 14	10
TOTAL		1,017

Considering the transmission line components, cables are primarily influenced by aerodynamic damping, while both structural and aerodynamic damping are significant for the tower structure [14]. In this study, the fluid-structure interaction aerodynamic damping (ξ_{aj}) for cables was modeled using Davenport's 1988 procedure [14], as shown in (13), where C_D is the drag coefficient, ρ_a is the air density, in kg/m³, d is cable diameter in m, m is the cable's mass per unit length in kg/m, v is the wind speed in m/s, and f_j is the natural frequency of the cable in Hz. For the tower, a viscous damping constant of 3% damping ratio (ξ = 0.03), typical for steel structures with bolted connections, was applied to combine the effects of different damping sources [15-17].

$$\xi_{aj} = \left(\frac{C_D}{4\pi}\right)\left(\frac{\rho_a d^2}{m}\right)\left(\frac{v}{f_j d}\right) \quad (13)$$

Geometric nonlinear static analysis is used to assess structures with large displacements, and the equilibrium equations are formulated through the deformed configurations. In this context, the equilibrium equations can be developed through the principle of virtual work, given by the equality of the internal and external work variation ($\delta W_{int} = \delta W_{ext}$). Thus, the geometric nonlinearity of the investigated transmission line is considered based on the Total Lagrangian Formulation. The Newton-Raphson method is used to solve nonlinear equilibrium equations. It involves linearizing the equilibrium equations and iteratively updating the solutions until convergence is achieved. Convergence criteria are based on displacements and the number of maximum iterations. The geometric nonlinear analysis was performed in two stages. Initially, self-weight was applied, ensuring that both conductor and shield wires were subjected to the horizontal design tension (EDS tension), resulting in the final calculated sag. Subsequently, additional loads were applied [18].

Nonlinear dynamic analysis is utilized to assess the behavior of structures when subjected to time-varying loads, considering, in addition to the dynamic effects, the effects of geometric nonlinearity. In this way, the nonlinear dynamic analysis considers the solution of the nonlinear motion equations (14) through a step-by-step integration in a coupled approach. $[\mathrm{M}]$, $[\mathrm{C}]$, $[\mathrm{K}]$, $\{\mathrm{F}\}$, $\{\ddot{\mathrm{u}}\}$, $\{\dot{\mathrm{u}}\}$, and $\{\mathrm{u}\}$ represent the mass matrix, damping matrix, stiffness matrix, applied load vector, acceleration vector, velocity vector, and displacement vector, respectively.

$$[\mathrm{M}]\{\ddot{u}(t)\} + [\mathrm{C}]\{\dot{u}(t)\} + [\mathrm{K}]\{u(t)\} = \{F(t)\} \qquad (14)$$

The approach for determining the nonlinear dynamic response of the investigated structural system combines the Newton-Raphson method for solving nonlinear equations with the Newmark method for integrating the system's motion equations over time. These methods allow the calculation of the structural response at each time increment. In linear models, the effective stiffness matrix remains constant, regarding nonlinear models, and the effective stiffness $[\mathrm{K}^{\mathrm{eff}}]$ is recalculated at each time step depending on the displacement values (15). $[\mathrm{K_T}]$ is the tangent stiffness matrix, and a_0 and a_1 are numerical parameters to integrate the equilibrium equations [19].

$$[\mathrm{K}]^{eff} = a_0[\mathrm{M}] + a_1[\mathrm{C}] + [\mathrm{K_T}] \qquad (15)$$

The strategy to determine the nonlinear dynamic response (Model III) of the transmission tower combines the Newton-Raphson method to solve nonlinear equations with the Newmark method for time integration. Geometric nonlinearity is accounted for using the Total Lagrangian Formulation. For the linear dynamic analysis (Model II), only the Newmark method is used.

Transverse wind actions, perpendicular to the transmission line axis, are important to assess the dynamic behavior of the structural system, imposing forces along the entire cable span and on towers, causing oscillations and vibrations. This study evaluated seven different wind velocities (u = {50, 45, 40, 35, 30, 25, 20} m/s), resulting in a total number of 224 analyses. Lattice steel towers are composed of slender members that are susceptible to buckling, making compression the most critical structural check. The base leg members endure the highest compressive stresses, as they bear the combined weight of the tower and external forces such as the wind. Therefore, a careful evaluation of these components is essential to ensure the overall stability of the tower [5]. This study focused on two key structural behaviors: the horizontal translational displacement in section A and the base leg member compression forces in structural element B (Figure 2). Dynamic analyses included thirty series of nondeterministic wind loads (Model III) for each wind velocity studied, underwent statistical evaluation and presented with reliability indices ($X_{95\%}$).

Considering Model I, the loadings related to cables, shield wires, and insulators were applied to the attachment points of the main tower and calculated based on the NBR 5422 [20]. The wind loads applied on the main steel tower (Model I) and the transmission line system (Model II) were determined based on the use of the Brazilian standard NBR 6123 [12]. The nondeterministic dynamic wind loads applied to Model III were modeled considering the analysis method described in this section.

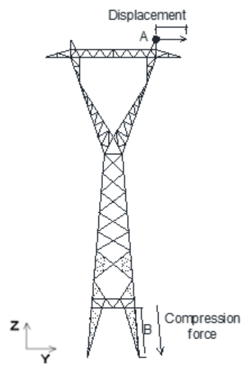


Fig. 2. Investigated structural sections: displacements and forces.

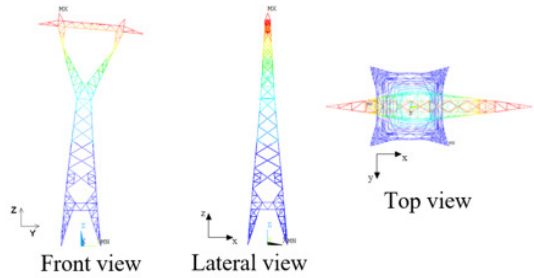
V. MODAL ANALYSIS

The natural frequencies (eigenvalues) and vibration modes (eigenvectors) of the isolated tower and the entire transmission line system (steel tower, cables, and insulators) were calculated using modal analysis (free vibration) in ANSYS software. Figures 3 and 4 show the first five calculated natural frequencies and vibration modes for an isolated tower and the transmission line system (tower and cables), respectively.

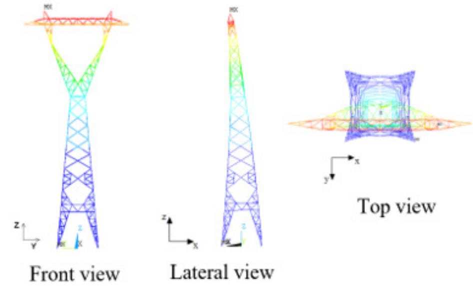
The free vibration analysis of the isolated tower revealed a fundamental frequency of 2.46 Hz (f_{01} = 2.46 Hz). However, considering the entire transmission line system, the fundamental frequency dropped significantly to 0.154 Hz (f_{01} = 0.154 Hz), a reduction of approximately 17 times. This indicates that the cables heavily influence the first vibration mode due to their high mass relative to their low stiffness. The isolated tower's fundamental frequency suggests that it is unlikely to experience significant resonance under wind excitation, as typical wind oscillation frequencies are much lower. However, analyzing the entire transmission line system is fundamental, as its natural frequencies can align with wind-induced resonant components, leading to significant structural responses.

VI. STATIC AND DYNAMIC ANALYSES

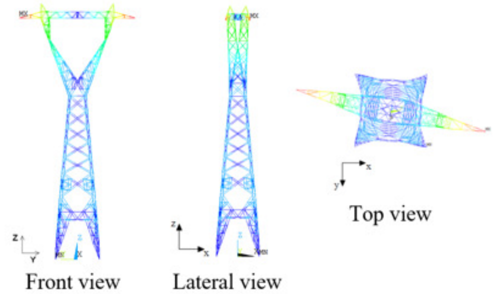
Linear elastic analysis was performed in Model I, nonlinear geometric analysis in Model II, and transient dynamic nonlinear geometric analysis in Model III. Table III shows the horizontal translational displacement in section A (top of the main tower - see Figure 2) and the compression force acting on member B (base leg member of the main tower - see Figure 2).



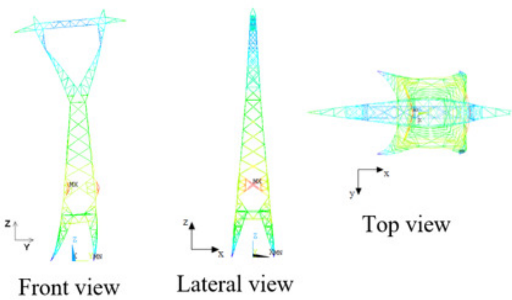
a) 1sr vibration mode (f_{01}=2.46 Hz).



b) 2nd vibration mode (f_{02}=2.58 Hz).

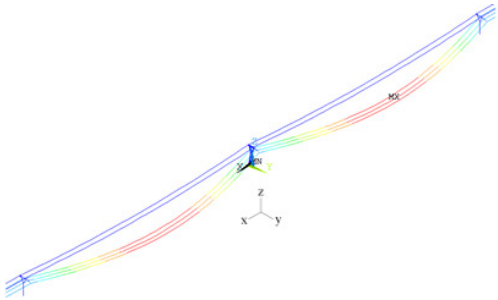


c) 3rd vibration mode (f_{03}=3.97 Hz).

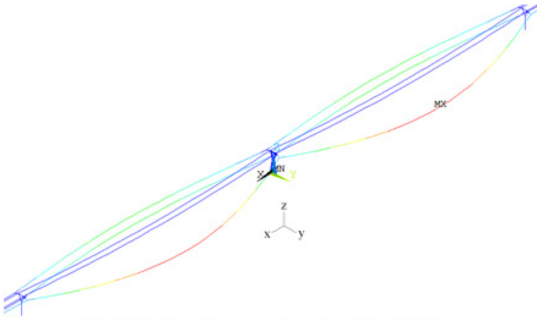


d) 4th vibration mode (f_{04}=6.83 Hz).

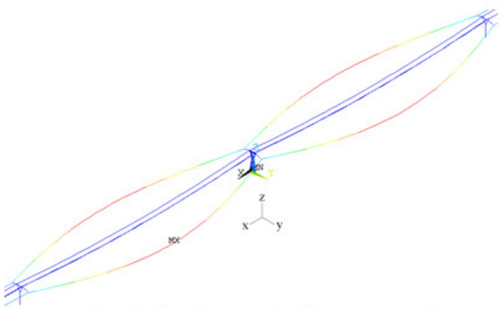
Fig. 3. Vibration modes: isolated steel tower.



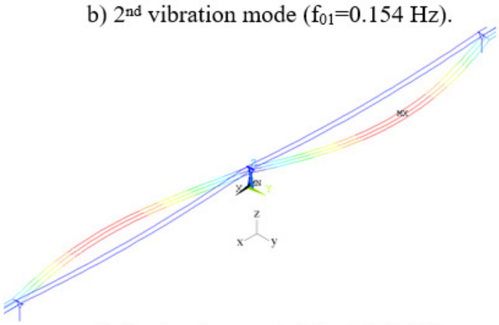
a) 1sr vibration mode (f_{01}=0.154 Hz).



b) 2nd vibration mode (f_{01}=0.154 Hz).



c) 3rd vibration mode (f_{01}=0.154 Hz).



d) 4th vibration mode (f_{01}=0.163 Hz).

Fig. 4. Vibration modes: full transmission line system (tower and cables).

The geometric nonlinear analysis of the tower cable system was found to result in higher horizontal translation displacement and force values compared to the linear analysis of the isolated tower. The results in Table III indicate that the horizontal translation displacement values calculated using the traditional method employed in current design practice (Model I: linear static analysis) differ substantially from those obtained with Model III (geometric nonlinear dynamic analysis), showing increases up to 95%. Similarly, the compression forces differ significantly, with increases of approximately 93%. These findings underscore the sensitivity of structures to varying load configurations and highlight the significant impact of wind dynamic forces on the structural analysis of transmission lines.

TABLE III. DISPLACEMENTS AND COMPRESSION FORCES

Model I							
Wind velocity *u* (m/s)	**20 m/s**	**25 m/s**	**30 m/s**	**35 m/s**	**40 m/s**	**45 m/s**	**50 m/s**
Displacement (m)	0.05	0.07	0.10	0.14	0.18	0.23	0.29
Force (kN)	36	50	67	87	110	136	165
Model II							
Wind velocity *u* (m/s)	**20 m/s**	**25 m/s**	**30 m/s**	**35 m/s**	**40 m/s**	**45 m/s**	**50 m/s**
Displacement (m)	0.07	0.09	0.12	0.15	0.19	0.23	0.29
Force (kN)	52	64	79	96	117	140	167
Model III							
Wind velocity *u* (m/s)	**20 m/s**	**25 m/s**	**30 m/s**	**35 m/s**	**40 m/s**	**45 m/s**	**50 m/s**
Displacement ($D_{95\%}$) (m)	0.09	0.14	0.18	0.26	0.35	0.45	0.54
Force ($F_{95\%}$) (kN)	58	80	110	156	203	263	319

a. $D_{95\%}$ and $F_{95\%}$: response characteristic values related to thirty series of nondeterministic wind loads.

Figure 5 shows a typical example of the tower's displacement over time when subjected to non-deterministic dynamic wind loads. Figure 6 illustrates this displacement in the frequency domain, determined using the Fast Fourier Transform (FFT), highlighting the displacement amplitude associated with the fundamental frequency of the transmission line system [f_{01} = 0.154 Hz: first vibration mode (Model III)].

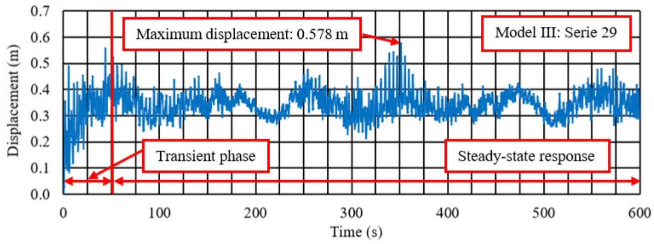


Fig. 5. Typical displacement. Point A (see Figure 2): time domain.

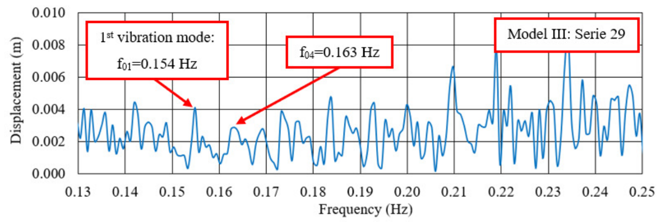


Fig. 6. Typical displacement. Point A (see Figure 2): frequency domain.

The Dynamic Amplification Factor (DAF) is an important parameter to evaluate the structure response when subjected to dynamic loads compared to static loads. It quantifies the increase in structural response due to the dynamic nature of applied loads, such as wind, and is calculated as the ratio between the structure's maximum response under dynamic loading and its response under an equivalent static load. It is important to emphasize that the static structural analyses (Models I and II) yielded a lower horizontal translational displacement (section A: see Figure 2) and base leg member compression force values (element B: see Figure 2) when compared to the dynamic structural analysis (Model III). The differences between Models I and II were negligible (see Table III). Table IV presents the differences and the DAF values between Models I and III, considering horizontal translational displacement (section A: see Figure 2) and the base leg member compression force (element B: see Figure 2) considering the basic wind speeds investigated. The results reveal that the differences in displacement at point A of the main tower and the compression force in member B (see Figure 2) can reach up to 95% and 93%, respectively. Additionally, the DAF for displacements in the analyzed models is approximately 2 (DAF = 2), while for forces, it ranges from 1.6 to 1.9 (DAF = 1.6 to 1.9), with higher values observed at increased wind speeds.

TABLE IV. DAF: DYNAMIC AMPLIFICATION FACTOR

Displacements in (m). Section A (see Figure 2).							
Velocity *u* (m/s)	**20**	**25**	**30**	**35**	**40**	**45**	**50**
D (Model I)	0.05	0.07	0.10	0.14	0.18	0.23	0.29
$D_{95\%}$ (Model III)	0.09	0.13	0.18	0.26	0.35	0.45	0.54
Increase (%)	78	83	81	87	92	95	85
DAF	1.8	1.8	1.8	1.9	1.9	1.9	1.8
Compression forces in (kN). Structural element B (see Figure 2).							
Velocity *u* (m/s)	**20**	**25**	**30**	**35**	**40**	**45**	**50**
F (Model I)	36	50	67	87	110	136	165
$F_{95\%}$ (Model III)	58	80	110	156	203	263	319
Increase (%)	61	60	64	79	85	93	93
DAF	1.6	1.6	1.6	1.8	1.8	1.9	1.9

VII. CONCLUSION

This study evaluated the dynamic structural response of a transmission line system section with a span of 900 m, which includes two spans of 450 m and comprises lattice steel towers, conductors, shield wires, and insulators. The analysis method introduced an evaluation of the dynamic structural behavior of a power transmission line when subjected to nondeterministic wind actions considering the effects of the geometric nonlinearity, aiming to study the dynamic interaction related to the entire tower-cables system. In this way, to ensure an even more dependable analysis of the transmission line system dynamic response, the wind loads were modeled based on the use of an ergodic second-order weakly stationary process. Thus, the mean maximum values associated with the displacements and compression forces of the investigated lattice steel towers were determined through an extensive statistical analysis of the entire tower-cables system steady-state response. The investigation developed three different analysis methods: (i) static linear analysis of the isolated main tower (Model I), (ii) static geometric nonlinear analysis of the transmission line system section (Model II), and (iii) geometric nonlinear dynamic analysis of the transmission line system section (Model III). Seven basic wind speeds (50, 45, 40, 35, 30, 25, and 20 m/s) were considered. Based on the results obtained, the following conclusions can be drawn:

- The global structural behavior of the investigated transmission line system changed when considering the effects of geometric nonlinearity, based on the modeling of the cables and the incorporation of the dynamic interaction related to the full tower-cables system, with modifications on the horizontal translational displacement and compression force values.

- The results revealed significant quantitative differences between the horizontal translational displacement and maximum compression force values predicted by the design standards and those obtained by geometric nonlinear dynamic analysis. Comparisons between the structural response of the investigated transmission line system, based on Model I (static linear analysis), Model II (static geometric nonlinear analysis), and Model III (geometric nonlinear dynamic analysis) show increases of up to 95% in displacements and 93% in member compression forces.
- This study demonstrated the fundamental role of geometric nonlinear dynamic analysis in understanding the global structural behavior, force distribution, stability, and design of transmission line systems. Based on the assessment of seven basic wind speeds, this study provides valuable information and emphasizes the need to take into account the dynamic effect of wind loads on the design of transmission lines.

However, it should be noted that the effect of the dynamic interaction associated with the full tower-cables system is very relevant to the global structural behavior of the power transmission line and the evaluation of the steel tower structure. The problem is certainly much more complicated and is influenced by the lattice steel tower type. In this way, further research should be conducted on this topic.

ACKNOWLEDGMENT

The authors gratefully acknowledge the financial support provided by the Brazilian Science Foundation CAPES, CNPq, and FAPERJ for this investigation.

REFERENCES

[1] M. S. Rechtman, "Structural response analysis of transmission lines steel towers when subjected to nondeterministic wind loadings," (in Portuguese), Ph.D. dissertation, State University of Rio de Janeiro, Brazil, 2024.

[2] "ONS - Operador Nacional do Sistema Elétrico," (in Portuguese), *National Electric System Operator (ONS) - Brazil.* http://www.ons.org.br/paginas/sobre-o-sin/o-sistema-em-numeros.

[3] D. Pylarinos, "Overhead Transmission Line Easement and Right-of-Way Cases in Crete, Greece: A Statistical Analysis of 1220 Cases from 1974 to 2019," *Engineering, Technology & Applied Science Research*, vol. 10, no. 3, pp. 5581–5589, Jun. 2020, https://doi.org/10.48084/etasr.3591.

[4] M. S. Rechtman, J. N. M. Motta, A. Menezes, J. G. S. da Silva, and O. Longo, "Structural Analysis of Steel Transmission Towers for Large River Crossing of Overhead Transmission Lines: A Case Study in Brazil," *Engineering, Technology & Applied Science Research*, vol. 10, no. 6, pp. 6438–6444, Dec. 2020, https://doi.org/10.48084/etasr.3735.

[5] S. Kalaga and P. Yenumula, Design of Electrical Transmission Lines: Structures and Foundations. London, UK: CRC Press, 2016.

[6] R. R. Roman, L. F. F. Miguel, F. Alminhana, and J. Pimenta, "Numerical simulation of full-scale tests in transmission line towers," in *Proceedings of the XLI Ibero-Latin-American Congress on Computational Methods in Engineering*, Natal, Brazil, Nov. 2020.

[7] D. Pylarinos, "Overhead Transmission Line Maintenance in Crete and Rhodes: 2016-2020," *Engineering, Technology & Applied Science Research*, vol. 11, no. 1, pp. 6833–6844, Feb. 2021, https://doi.org/10.48084/etasr.4036.

[8] B. A. S. Oliveira and J. G. S. Da Silva, "Modelling of the dynamic structural behaviour of steel telecommunications towers subjected to the non-deterministic dynamic action of wind loads," presented at the XXXVII CILAMCE: Iberian Latin-American congress on computational methods in engineering, Brazilia, Brazil, 2016.

[9] J. A. S. Almeida, "Comparative analysis between European standards and North American standards in the characterization of the dynamic action of wind on high-rise telecommunications towers," (in Portuguese), M.S. Thesis, University of Porto, Portugal, 2012.

[10] P. Hadimani and S. Kulkarni, "Static and dynamic analysis of transmission line tower," *International Journal of Emerging Research in Management e Technology*, vol. 6, no. 2, pp. 147–154, 2017.

[11] R. W. Clough and J. Penzien, *Dynamics of Structures*, Subsequent edition. New York, NY, USA: McGraw-Hill College, 1995.

[12] "Forces due to wind on buildings," (in Portuguese), Brazilian Association of Technical Standards (ABNT), Rio de Janeiro, Brazil, NBR 6123, 1988.

[13] L. S. Bastos, "Assessment of the non-deterministic dynamic structural behaviour of tall buildings considering the effect of the transverse action of the wind and the modelling of the soil-structure interaction," (in Portuguese), Ph.D. dissertation, University of Rio de Janeiro, Brazil, 2020.

[14] A. M. Loredo-Souza and A. G. Davenport, "The effects of high winds on transmission lines," *Journal of Wind Engineering and Industrial Aerodynamics*, vol. 74–76, pp. 987–994, Apr. 1998, https://doi.org/10.1016/S0167-6105(98)00090-7.

[15] A. K. Chopra, *Dynamics of Structures: Theory and Applications to Earthquake Engineering*, 2nd edition. Upper Saddle River, NJ, USA: Prentice Hall, 2000.

[16] J. Kaminski Jr., L. F. Miguel, and R. C. R. Menezes, "Relevant aspects in the dynamic analysis of LT towers subjected to cable rupture," presented at the XVIII SNPTE National Seminar on Electric Energy Production and Transmission, Curitiba, Brazil, 2005.

[17] V. R. Da Silva, J. D. Riera, J. Blessman, L. F. Nani, and E. E. Galindez, "Experimental Determination of the Basic Dynamic Properties of a 230 kV Transmission Tower," (in Portuguese), presented at the VII SNPTE: National Seminar on Electric Energy Production and Transmission, Brazilia, Brazil, 1984.

[18] A. Alves Filho, The basis of CAE technology: Dynamic analysis. Sao Paolo, Brazil, 2005.

[19] S. N. R. Remala, "Nonlinear transient finite element simulations of beam parametric response including quadratic damping," M.S. Thesis, University of Kentucky, USA, 2005.

[20] "Design of overhead power transmission lines," (in Portuguese), Brazilian Association of Technical Standards, Rio de Janeiro, Brazil, NBR 5422, 2024.