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Comparative Analysis of Static and Dynamic Methods in Metallic Tower Design

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Abstract. This paper presents the design of a telecommunications tower, addressing various structural analysis methods: static, simplified dynamic, discrete dynamic, and modal analysis, as defined in ABNT NBR 6123:2023, for wind loads acting on the structure. The study focuses on a tower located in the region of Pato Branco, in the state of Paraná (Brazil), with a height of 50 meters, constructed with tubular profiles. The calculation procedure was carried out using *ANSYS 2018 R1* software, incorporating the mentioned methods through finite element analysis. While the static analysis does not consider dynamic effects, the dynamic methods provide results with different periods, reflecting the response of the structure. The aim of this study was to determine which method should be adopted, taking into account the design requirements for axial tension and compression. Evaluating the safety margins obtained for the structural elements, it was found that up to 20 meters in height, the static analysis presents higher loads; however, above this height, the dynamic effects of the wind are predominant. This result demonstrates the importance of dynamic wind analysis for towers over 20 meters in height. Furthermore, it is possible to verify that the simplified model using the first period of the structure according to ABNT NBR 6123:2023 is more conservative than using the period and frequencies resulting from a modal analysis by numerical finite element modeling.

Keywords: towers, static wind, dynamic wind, modal analysis.

1 Introduction

In recent years, significant advancements in the telecommunications field, particularly within the mobile phone segment, have driven substantial growth in the demand for telecommunications towers to expand signal transmission capabilities. These structures are primarily intended to support signal transmission antennas [1]. Given their exposed and elevated nature, telecommunications towers are susceptible to environmental forces such as high-speed winds, which exert substantial lateral loads on the towers. If not adequately designed and constructed, these forces can lead to structural failures [2].

To address the wind effects on these structures, ABNT NBR 6123:2023 [3] offers several methods for analyzing the dynamic effects of wind on tall and slender structures, including the Simplified Method, the Discrete Method, and the Modal Analysis Method. The Simplified Method is appropriate for structures with simple and symmetrical geometries, estimating pressure coefficients on the structure's surfaces and considering the direct action of wind in each principal direction. For more complex or unconventional geometries, the Discrete Method subdivides the structure's surface into panels and evaluates wind pressures on each panel [4].

The Modal Analysis Method, however, is particularly suited for telecommunications towers and other slender structures. It takes into account the natural frequencies and modes of vibration of the structure, applying wind forces according to these modes and considering modal coefficients. This method is especially valuable for structures subject to significant vibrations due to wind [5].

This paper aims to conduct a comparative analysis of the different design methods for telecommunications

towers, focusing on the differences among the static, simplified, discrete, and modal analysis approaches. It will investigate how these methods impact tower design, particularly in terms of safety margins for profiles designed for tension and compression, following the ABNT NBR 8800:2008 methodology [6]. The goal is to determine whether the choice of method could lead to under-dimensioned or over-dimensioned structures and to underscore the importance of thorough analysis during the calculation process to ensure the tower's stability and efficiency.

2 Theoretical framework

According to ABNT NBR 8800:2008 [6], loads can be classified as permanent and variable. Permanent loads are constant over time and include the self-weight of the structure and all fixed elements such as cladding, walls, floors, and equipment. Variable loads, or accidental loads, can vary over time and are not permanently present in the structure. These include occupancy loads such as people, furniture, vehicles, as well as wind, snow, and temperature loads.

2.1 Wind loads

The wind load for towers is calculated based on section 7.7 of the NBR 6123:2023 standard [3], which considers this action acting on lattice towers with square or equilateral triangular sections. For the proper design of these structures, wind is divided into static and dynamic approaches.

2.1.1 Static wind load

The ABNT NBR 6123:2023 [3] proposes a method to determine the equivalent static wind action based on Davenport's random vibration method. Structural vibration occurs around the position deformed by the static component of the wind. To determine the static wind force, meteorological parameters and drag coefficients must be defined. The characteristic wind speed (V_k) is calculated using Equation 1:

$$V_k = V_0 S_1 S_2 S_3 \tag{1}$$

In the previous equation, S1 represents the topographic factor, which is 1.0 for flat or slightly hilly ground. S2 denotes the terrain roughness factor, calculated based on building dimensions and height according to section 5.3 of ABNT NBR 6123:2023 [3]. S3 is the statistical factor, derived from Table 3 in section 5.4 of ABNT NBR 6123:2023 [3]. The basic wind speed (V0), which is the speed of a 3-second gust exceeded on average once in 50 years, is obtained from the wind isopleths provided in the standard. The dynamic pressure (q) is then calculated using Equation 2 in N/m².

$$\mathrm{q} = 0{,}613\, V_k^2 \tag{2}$$

The global wind force on a building or part of it, F_g, is obtained by the vector sum of the acting wind forces. The component of the global force in the wind direction, the drag force F_a, can be obtained using Equation 3 in Newtons (N).

$$F_a = C_a q A_e \tag{3}$$

where C_a is the drag coefficient, determined by ABNT NBR 6123:2023 [3] and depends on the geometry, type and orientation of the structure, terrain surface and the presence of additional elements. A_e is the effective frontal area.

2.1.2 Dynamic wind load

The calculation of the dynamic impact of wind on slender structures, such as telecommunications towers, involves three main steps: wind characterization, definition of the physical and aerodynamic properties of the structure, and the combination of these factors to determine the structural response. For dynamic wind analysis on

the structure, the following approaches are used: simplified, discrete, and simplified with vibration modes obtained through numerical analysis.

The simplified dynamic model is suitable for structures with a constant cross-section, uniform mass distribution, height less than 150 meters, and support exclusively at the base. In this model, only the fundamental mode of vibration is considered, as ABNT NBR 6123:2023 [3] states that considering only the first mode results in errors of less than 10%. For this method, the component of the global force in the wind direction, the drag force Fa, can be obtained using Equation 3, which considers a dynamic amplification coefficient resulting in a portion of the fluctuating dynamic pressure and another portion as a mean value, according to section 9.3.1 of ABNT NBR 6123:2023 [3].

The calculation of the dynamic response using the discrete method in ABNT NBR 6123:2023 [3] involves the natural vibration frequency of the tower, the mass of each module, the design wind speed (Vp), the displacement of the structural influence nodes, and the dynamic amplification coefficient (ξ). The dynamic load on the structure is obtained by summing the mean force and the fluctuating component, as described in the equations in section 9.3.2 of ABNT NBR 6123:2023 [3].

For steel towers and chimneys with a constant cross-section, it is approximately considered that γ=1.7 and ζ=0.008, according ABNT NBR 6123:2023 [3]. Using Equation 4 is possible obtain the direct, though approximate, calculation of the period (T_1) and fundamental frequency (f_1) in Hz of these structures as a function of height (h), which is the height of a building above the ground, measured to the top.

$$T_1 = 0{,}29\sqrt{h} - 0{,}4 = 1/f_1 \tag{4}$$

However, f_1 and γ can also be obtained through numerical methods using a modal analysis that considers vibration theory. This modal analysis accounts for the rigidity of the structure and the localized masses.

2.2 Action combinations

Combinations of actions, along with the application of amplification and reduction coefficients as specified by ABNT NBR 8800:2008 [6], were implemented for the ultimate limit state design:

- Comb.1: 1,25× (Self weight) + 1,5× (Live load) + 1,4×0,60× (Wind 0º)
- Comb.2: 1,25× (Self weight) + 1,5× (Live load) + 1,4×0,60× (Wind 45º)
- Comb.3: 1,25× (Self weight) + 1,5×0,8× (Live load) + 1,4× (Wind 0º)
- Comb.4: 1,25× (Self weight) + 1,5×0,8× (Live load) + 1,4× (Wind 45º)

In combinations 1 and 2, the live load is the primary variable action and the dynamic wind is the secondary action. In combinations 3 and 4, wind is the primary action and the live load is the secondary action. The four combinations were used for both static and dynamic wind forces, applied to both the tower and the antennas.

2.3 Axial tension

According to ABNT NBR 8800:2008 [6], for the design of members subjected to axial tensile forces, the condition expressed in Equation 5 must be satisfied.

$$N_{t,Sd} \leq N_{t,Rd} \tag{5}$$

In this context, $N_{t,Sd}$ represents the axial tensile force to be considered in the design, obtained in this work through numerical analysis using finite element modeling with *ANSYS 2018 R1*;

$N_{t,Rd}$ is the design axial tensile strength, determined as the lower value from the equations in ABNT NBR 8800:2008 [6], which account for the ultimate limit states of yielding of the gross section and rupture of the net section. The effective net area was calculated according to sections 5.2.3 and 5.2.5 of the standard, with a net area reduction coefficient of 1.00 for circular tubular sections when the tensile force is transmitted through a concentric plate, provided the connection length is at least 1.30 times the bar's external diameter.

2.4 Axial compression

ABNT NBR 8800:2008 [6] provides criteria for the design of prismatic members subjected to axial compression, taking into account buckling effects. For the design of these members, the condition presented in Equation 6 must be satisfied.

$$N_{c,Sd} \leq N_{c,Rd} \tag{6}$$

$N_{c,Sd}$ is the design axial compression force, obtained in this work through numerical analysis using finite element modeling with *ANSYS 2018 R1*.

$N_{c,Rd}$ is the design axial compression strength, obtained according to ABNT NBR 8800:2008 [6], which takes into account effects associated with ultimate limit states of instability due to bending, torsion, or combined bending and torsion, as well as local buckling.

3 Materials and Methods

For the development of the comparative analysis between static and dynamic wind methods, a 50-meter-tall telecommunications tower was used, with a square base composed of circular tubular profiles. As peripherals, 16 m² of antennas were installed, with 8 m² located at a height of 40 meters and the remaining 8 m² at 50 meters. Additionally, it was necessary to consider the inclusion of accessories with a rate of 0.50 m²/m along the tower, such as maintenance ladders for the antennas. This tower is to be installed in the region of Pato Branco, Paraná, Brazil.

The calculation of wind loads was performed according to ABNT NBR 6123:2023 [3] by dividing the structure into 5 modules of 10 meters each. A calculation procedure was implemented using *Python 3*. For the analysis of the forces on the tower, *ANSYS 2018 R1* software was used.

3.1 Geometry

The tower geometry consists of a 40-meter conical section followed by a 10-meter straight section. At the top of the conical section, there is a 2-meter opening. K-type modulators were used for the conical section, and X-type modulators were used for the straight section.

The design of the tower's members considered the circular tubular profiles manufactured by Vallourec in accordance with the EN 10210-2:2006 standard [7], and these profiles are presented in Table 1. The nomenclature represents MSH KHP (Outer diameter) x (Wall thickness) in mm.

Table 1. Cross-sections of tower members

Height [m]	Vertical members	Diagonals	Horizontals	Bracing
0 – 5	MSH KHP 168.3x8.8	MSH KHP 101.6x4	MSH KHP 60.3x3.6	MSH KHP 60.3x3.6
5 – 10	MSH KHP 168.3x8.8	MSH KHP 101.6x4	MSH KHP 60.3x3.6	MSH KHP 60.3x3.6
10 – 15	MSH KHP 168.3x7.1	MSH KHP 101.6x4	MSH KHP 42.4x3.6	MSH KHP 60.3x3.6
15 – 20	MSH KHP 168.3x7.1	MSH KHP 101.6x4	MSH KHP 42.4x3.6	MSH KHP 60.3x3.6
20 – 25	MSH KHP 168.3x6.3	MSH KHP 101.6x4	MSH KHP 33.7x3.6	MSH KHP 42.4x2.9
25 – 30	MSH KHP 168.3x6.3	MSH KHP 101.6x4	MSH KHP 33.7x3.6	MSH KHP 42.4x2.9
30 – 35	MSH KHP 139.7x5.6	MSH KHP 60.3x4	MSH KHP 26.9x2.9	MSH KHP 33.7x3.6
35 – 40	MSH KHP 139.7x5.6	MSH KHP 60.3x4	MSH KHP 26.9x2.9	MSH KHP 33.7x3.6
40 – 45	MSH KHP 76.1x4.5	MSH KHP 42.4x2.9	MSH KHP 26.9x2.9	MSH KHP 26.9x2.9
45 – 50	MSH KHP 76.1x4.5	MSH KHP 42.4x2.9	MSH KHP 26.9x2.9	MSH KHP 26.9x2.9

3.2 Material

In the design of the tower's members, all circular tubular profiles made of VBM300 steel were considered. The mechanical properties of this material are listed in Table 2.

Table 2. Mechanical properties

Properties	Value / Unit
Steel density	7850 kg. /m³
Yield strength	300 MPa
Ultimate strength	415 MPa
Modulus of elasticity	205GPa
Shear modulus	79 GPa
Poisson's ratio	0,3

3.3 Permanent loads

The self-weight was calculated automatically based on the geometry and material. Additionally, the mass of the antennas, 25 kg (250 N), was included, distributed along the straight section of the tower, positioned at heights of 40 meters and 50 meters. A mass of 200 kg per module, corresponding to 2000 N, was assigned, covering the platforms, ladders, walkways, and antenna supports [8].

3.4 Variable loads

The variable loads include both the live load and wind action. For the live load, an equivalent mass of four people, each weighing 70 kg (4 x 70 kg = 280 kg, corresponding to 2800 N), was considered, plus an additional 150 kg (1500 N) for equipment, totaling 430 kg (4300 N) of live load. This load was applied at the most critical position of the tower, which is the top of the structure, located at the platform 50 meters high [8].

Wind loads were applied at angles of 0° and 45°, as illustrated in Figure 1. For the design, the ultimate limit state combination was employed. Four independent analyses were performed, considering different wind loading cases: the first used the static method as outlined in section 7.7 of ABNT NBR 6123:2023; the second applied the simplified dynamic method per section 9.3.1 of ABNT NBR 6123:2023; the third utilized the discrete dynamic method according to section 9.3.2 of ABNT NBR 6123:2023; and the fourth case used the simplified method, incorporating the natural frequencies and displacements of the first six vibration modes obtained through modal analysis with ANSYS 2018 R1.

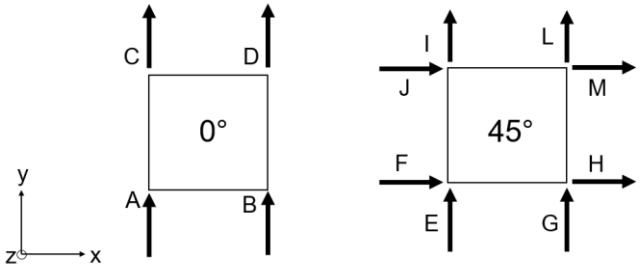


Figure 1. Wind loads for 0° and 45° directions

3.5 Profile analysis and safety margin

With the finite element model developed in *ANSYS 2018 R1*, the tensile and compressive forces for all elements, as well as displacements and stresses, were obtained. Using these results and the permissible values obtained from ABNT NBR 8800:2008, safety margins were calculated for both static wind and dynamic wind conditions. The safety margins ($Sm_{\%}$) were calculated to demonstrate how close a component is to structural failure, and can be obtained using Equation 7.

$$Sm_{\%} = \left(\frac{N_{Rd}}{N_{Sd}} - 1\right) \times 100 \tag{7}$$

4 Results

After verifying the profiles for tension and compression, the safety margins were plotted. In Figure 2 it is illustrated the safety margins of the vertical members as a function of height for static wind, simplified dynamic, discrete dynamic, and modal dynamic conditions. In this graph, a higher safety margin value indicates that the profile is less stressed.

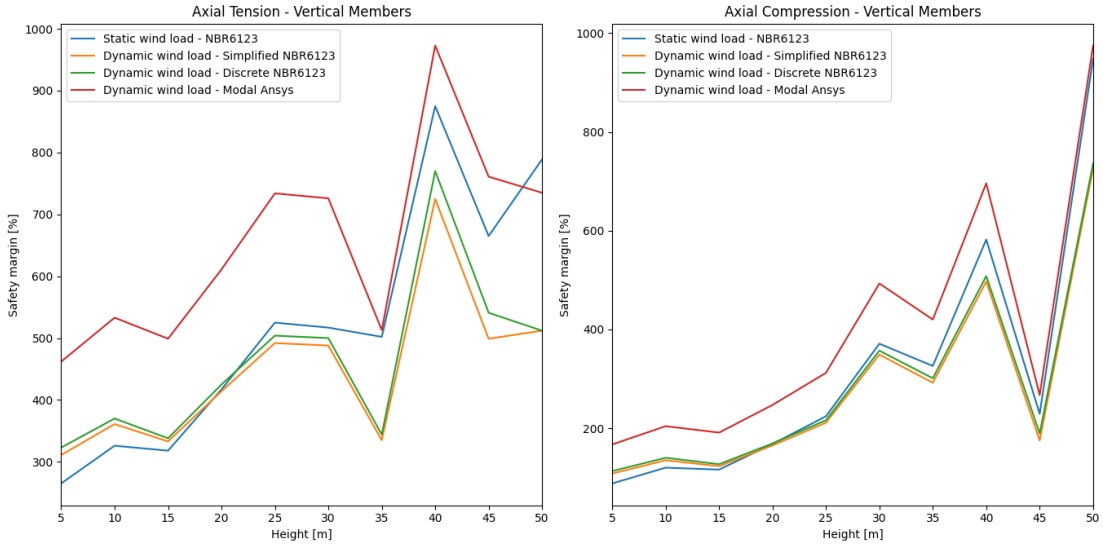
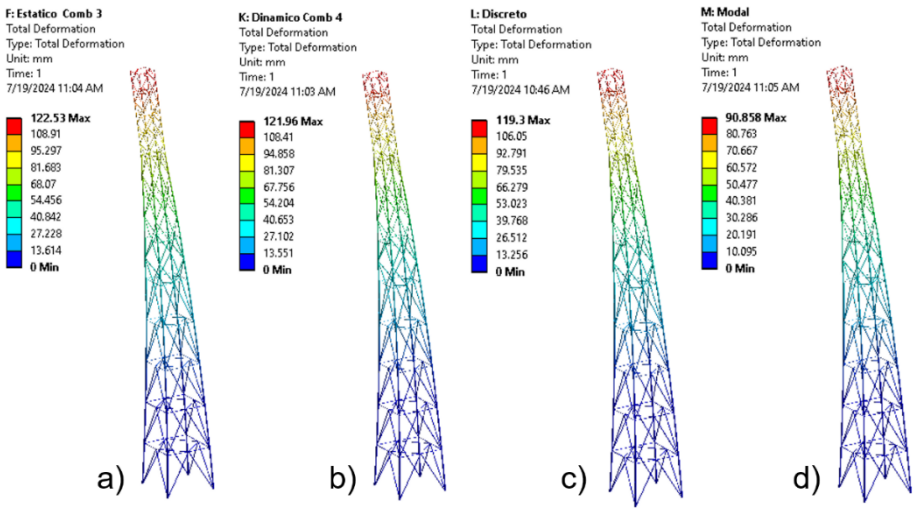


Figure 2. Safety margin of vertical members depending on height

In the profile verification, bending effects were not considered, as the trussed structure is subjected only to tensile and compressive forces. For the four loading conditions, maximum displacements were evaluated, following the Telebrás 240-410-600/1997 standard [8], which establishes a displacement limit of 1° and 40'. For the tower in question, this limit is 1454.9 mm. The maximum displacements for each loading condition are presented in Figure 3.



Maximum displacements

a) Static wind b) Simplified dynamic wind c) Discrete dynamic wind d) Dynamic wind by modal analysis

5 Conclusion

This study compared the methods proposed by ABNT NBR 6123:2023 to evaluate the behavior of a self-supporting metal tower subjected to wind loads. The methods assessed were: static, simplified dynamic, discrete dynamic, and simplified dynamic with modal analysis. Using *ANSYS 2018 R1*, a finite element model was created to obtain tensile and compressive forces, which were subsequently evaluated according to ABNT NBR 8800:2008.

The results indicate that dynamic analysis is essential for this type of structure, as above 20 meters in height, the safety margins of the profiles start to decrease for dynamic loads compared to the static method. Additionally, the model evaluated through numerical modal analysis presents higher safety margins because it considers the structure's stiffness and localized masses in calculating the natural frequency. For the tower in question, the period $T1$ by the simplified method was 1.65 seconds, while the period of the first vibration mode obtained from numerical analysis was 0.345 seconds. Since the dynamic amplification factor takes into account the natural frequency, which is a function of the period, shorter periods reduce the amplification factor, thereby decreasing the loads.

According to ABNT NBR 6123:2023, retaining only the first mode in the simplified solution leads to errors of less than 10%. This behavior was confirmed, as there was an approximate 8% difference between the simplified and discrete methods. The absence of structural collapse can be attributed to the design that considers the tower's fabrication and the connections between the elements, resulting in high safety margins for some components.

However, it is possible to graphically observe the influence of the dynamic component with the increase in tower height, indicating that dynamic wind effects should always be considered. It is up to the engineer to choose between a more conservative approach, using the equations provided by ABNT NBR 6123:2023, or a more optimized approach, utilizing numerical or experimental analysis to determine the vibration modes and natural frequencies of the structure, and thus find the dynamic amplification factor for wind loads.

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