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Static and dynamic analysis of telecommunication towers subjected to wind

Análise estática e dinâmica das torres de telecomunicação sujeitas ao vento

Análisis estático y dinámico de torres de telecomunicaciones sometidas al viento

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Abstract
The expansion of the telecommunications sector in Brazil in recent years and the consequent increase in demand for telecommunication towers open space for a discussion about the guidelines to be taken during the execution of projects of this type of structure. Because they are light and slender structures, wind loads become preponderant for the design of these towers and wind analysis is an important topic to be discussed. Thus, this work analyzes design aspects employing numerical modeling and parametric studies. The Brazilian standard NBR 6123:1988 was used to obtain the wind speed and its static and dynamic loads, to compare the responses generated by each of the methods. Then, this process was repeated following the European standard IEC 60826:2017, to compare the results between the two standards methods. A parametric study was also performed with towers of different heights to better understand the structure behavior. The tower responses were obtained using the finite element software SAP2000, in which a lattice tower model with a constant square section was developed, to which the wind loads calculated according to the aforementioned standards were applied. The results showed that each analysis method generates considerably different results, revealing that this is a point that should be very well studied during the design of telecommunication towers.
Keywords: Telecommunication towers; Wind loads; Numerical modeling.

Resumo
O crescimento da telecomunicação observado no Brasil nos últimos anos e o consequente aumento na demanda por torres de telecomunicação abre espaço para uma discussão a respeito das diretrizes a serem tomadas durante a execução de projetos desse tipo de estrutura. Por se tratar de estruturas leves e esbeltas, as solicitações provenientes da ação do vento se tornam preponderantes durante o dimensionamento destas torres e a análise do vento é um importante tópico a ser discutido. Assim, neste trabalho são analisados aspectos de projeto por meio de modelagem numérica e estudos paramétricos. A norma brasileira NBR 6123:1988 foi utilizada para se obter a velocidade do vento e suas solicitações estáticas e dinâmicas, a fim de se comparar as respostas geradas por cada um dos métodos. Em seguida, esse processo foi repetido seguindo norma europeia IEC 60826:2017, visando comparar os resultados entre as duas normas. Também foi realizado um estudo paramétrico com torres de diferentes alturas para se compreender melhor o comportamento da estrutura. A obtenção das respostas da torre foi feita no software de elementos finitos SAP2000, no qual se desenvolveu um modelo de torre treliçada de seção quadrada constante, sobre a qual foram aplicados os carregamentos de vento calculados segundo as normas supracitadas. Os resultados evidenciaram que cada método de análise gera resultados consideravelmente distintos, revelando que este é um ponto que deve ser muito bem estudado durante o dimensionamento de torres de telecomunicação.
Palavras-chave: Torres de telecomunicação; Forças devidas ao vento; Modelagem numérica.

Resumen
El crecimiento de las telecomunicaciones observado en Brasil en los últimos años y el consiguiente aumento de la demanda de torres de telecomunicaciones abre un espacio para la discusión sobre las directrices que deben tomarse durante la ejecución de proyectos de este tipo de estructura. Al tratarse de estructuras ligeras y esbeltas, las solicitudes procedentes del aire del viento se vuelven preponderantes durante el dimensionamiento de las torres y el análisis del

viento es un tópico importante a discutir. Por lo tanto, en este trabajo se analizan los aspectos del proyecto mediante la modelización numérica y los estudios paramétricos. Se utilizó la norma brasileña NBR 6123:1988 para obtener la velocidad del viento y sus cargas estáticas y dinámicas, con el fin de comparar las respuestas generadas por cada uno de los métodos. A continuación, se repitió este proceso siguiendo la norma europea IEC 60826:2017, con el fin de comparar los resultados entre las dos normas. También se realizó un estudio paramétrico con torres de diferentes alturas para comprender mejor el comportamiento de la estructura. Las respuestas de la torre se obtuvieron en el software de elementos finitos SAP2000, en el que se desarrolló un modelo de torre de celosía de sección cuadrada constante, sobre el que se aplicaron las cargas de viento calculadas según las normas mencionadas. Los resultados mostraron que cada método de análisis genera resultados considerablemente diferentes, lo que revela que este es un punto que debe ser muy bien estudiado durante el diseño de las torres de telecomunicaciones.
Palabras clave: Torres de telecomunicaciones; Fuerzas del viento; Modelización numérica.

1. Introduction

The world's first telecommunications systems date back further than commonly thought. In March 1876, Alexander Graham Bell, in an attempt to improve the telegraph, created the first telephone that, after decades of technological evolution, originated the smartphones that are now part of the lives of most of the Brazilian and world population. The internet reached the Brazilian public in 1994, but it had been created long before that. In the 1960s, the United States military developed the ARPANET project, which allowed the exchange of information in which each machine worked independently. It was the beginning of Internet creation (Lins, 2013).

The vast telecommunications network we have today in Brazil works through data transmitted by antennas. To ensure proper operation, these antennas must be highly elevated, which rises the demand for structures that properly perform this role (Filipe, 2012). In this context, self-supporting lattice towers are widely used in Brazil. This structure type enables the elevation of antennas to considerable heights, mainly due to its low weight. As these structures are light and slender, the analysis of wind action becomes a determining factor in their design.

According to Blessmann (2013), the wind is the movement of air over the earth's surface and one of the reasons for this movement is the difference in temperature of the air masses due to the variation in atmospheric pressure. The wind is a natural and random event, so it must be properly analyzed. Thus, some authors argue that the wind analysis as a static request may not be satisfactory and suggest that this factor should be analyzed dynamically, to obtain results more consistent with the reality.

Within this context, it can be seen that the constant technological development and the growth of telecommunications in Brazil brings an increase in demand for self-supporting structures (towers) that, in large part, are metallic, with low self-weight and elevated height and slenderness. Therefore, the wind action becomes preponderant in the design of structures of this nature, which can be done through static and dynamic analysis. Thus, it is of utmost importance to study and understand the two analyses, since in certain cases the dynamic analysis presented up to 243.59% greater than the static analysis (Bronzatto, 2012). Furthermore, there is no specific standard for the design of telecommunication structures, as well as no consensus on what situations require the static or the dynamic analysis.

Therefore, this paper compares the static and dynamic analysis of telecommunication towers subjected to wind. With this in view, the methodology consists of a succinct literature review concerning the state of the art about finite element numerical analyses of self-supporting towers and, briefly, the design criteria for lattice steel structures are presented. Furthermore, the methodology is complemented by numerical modeling and parametric studies, as well as performed by Antunes et al. (2012), Oliveira et al. (2019), Szafran et al. (2019), Reis (2020), and, recently, Bezas et al. (2022). Models of a telecommunication tower were developed in the SAP2000 software, which made it possible to evaluate the effects caused by winds according to static and dynamic analyses, as recommended by the NBR 6123 standard (ABNT, 1988) and according to the static analysis by the European standard IEC 60826 (IEC, 2017). In all cases, linear and nonlinear analyses were performed.

2. The State of the Art

2.1 Static and dynamic analysis of lattice structures

Because they are slender structures with low self-weight, the wind action and its correct determination become fundamental for a safe design of telecommunication towers. When it comes to the analysis of wind actions in slender structures, one of the main points of debate is which analysis method should be performed. The Brazilian standard NBR 6123 (ABNT, 1988) presents the guidelines for the design of wind action both statically and dynamically, but there is no guidance in this standard about which method is the most appropriate for certain situations. The standard highlights that for buildings with a fundamental period equal to or less than 1 second, the responses due to fluctuations in wind speed are small and are already implicitly considered in the range adopted for the the terrain roughness factor S_2.

Almeida and Vidoto (2013) performed a comparative study on the wind action calculation methods provided by NBR 6123 applied to slender reinforced concrete buildings, where the static analysis proved to be more conservative, especially at lower heights. The same authors also noticed that with increasing height, the results of the two methods of analysis became closer, with the dynamic method showing higher internal loads in some cases. It highlights the need for a deeper evaluation of these situations since both analyses presented preponderant results when designing the structure. In this same context, Reis (2020) points out that the maps provided by Brazilian standards for obtaining the basic wind speed can be considered obsolete and generate unsatisfactory results because the databases used in their confection are very old.

According to Silva (2018), a dynamic action can suffer variations of magnitude, direction, and place of the application over time. Thus, the wind is understood this way because it is a random natural phenomenon.

To characterize the dynamic analysis, three fundamental properties are considered: the natural modes of vibration, the damping factors, and the natural frequencies of the structure. The natural frequency indicates the rate of vibration of the structure in its free state, that is when loading is ceased. The oscillation can occur in several ways and several directions and is designated as the natural mode of vibration. For each mode of vibration there is a natural frequency, and the first, smallest, and most important, is called the fundamental frequency (Bolina et al., 2015).

Generally, the response of a structure subjected to dynamic loading is presented in the form of displacements (Silva, 2018). According to Machado and Pinto (2016), the analysis of dynamic effects is done in three steps. The first of them is the analysis of the wind, determining its speed. The second step is the analysis of the physical and aerodynamic properties of the structure, and the third step consists of combining the two previous steps to determine the response of the structure. The authors concluded in their comparative study between static and dynamic analysis of wind action that the effects of dynamic analysis generate greater efforts and displacements in the structure.

In a comparative study between the static and dynamic methods performed by Bronzatto (2012) it was concluded that, in a specific case of a self-supporting lattice tower, at 97 meters high, the dynamic stress found was 243.59% higher than in the static analysis.

Another factor to be mentioned is the choice of the standard to be used. When comparing the calculation methods of the NBR 6123 and NBR 5422 standards in a railway bridge, for example, Reis et al. (2021) point out that the nonlinear analysis of the model obtained results for the bridge support reactions of up to 15 % higher compared to the linear analysis. Oliveira et al. (2019), in turn, conducted a comparative study between the two standards applied to 32 dynamic analyses involving the structural behavior of transmission line cables. In this study, results were found to be up to 28% higher for dynamic analysis when compared to static analysis.

2.2 Design criteria for lattice structures

In Brazil, there is no standard dedicated to the design of telecommunication structures, and the NBR 8800 - Design of steel structures and steel-concrete composite structures for buildings (ABNT, 2008) is very conservative in its results, using the limit state method. This standard considers that the collapse of steel structures occurs when the load applied to the structure exceeds an ultimate limit state (ELU), i.e., when the internal loads are greater than the strength load, either in the structure or in the connections. Thus, the collapse can occur by tension in the bars, shearing of bolts, tearing or crushing of the sections of the profiles, or buckling of the bars, the latter being the most common (Torres & Inoue, 2021). For this reason, several works found in the literature point to the use of standards from other countries, such as the American ASCE 10-15 (ASCE, 2015).

According to Mendes (2020), lattice structures can be considered rigid and end-labeled due to the low bending stiffness of the connecting bolts, thus, the bars of the structures would only be subject to axial compressive and tensile stresses, requiring only structural processing based on the linear-elastic regime and without considering geometric nonlinearity. However, in the case of flexible structures, the deformed position of the structure must be considered, because the effects of geometric nonlinearity can be significant.

According to ASCE 10-15 standard (ASCE, 2015), for the case of compression, the design of the bars is done by comparing the admissible stress (derived from the critical buckling stress, for the elastic and inelastic regime cases) and the acting stress (obtained from the different combinations of actions). In addition to global buckling, it is also necessary to check local buckling in the elastic and inelastic regimes. For tensioned bars, the AISC 360-10 (AISC, 2010) and AISI S100 (AISI, 2016) standards suggest that the strength should be verified according to the yielding of the net area of the cross-section, while the Brazilian standard NBR 8800 (ABNT, 2008), for the limit state method, considers the yielding of the gross area and the rupture of the net area.

When it comes to design, it is known that the wind action is responsible for the main demands on telecommunication towers, and there are some calculation methods for the estimation of its speed and strength. In Brazil, the calculation of wind loads is regulated by the Brazilian standard NBR 6123 (ABNT, 1988), and is also recommended in the standard NBR 5422 (ABNT, 1985), which deals specifically with the design of overhead transmission lines. In Europe, the main standard used is IEC 60826 (IEC, 2017).

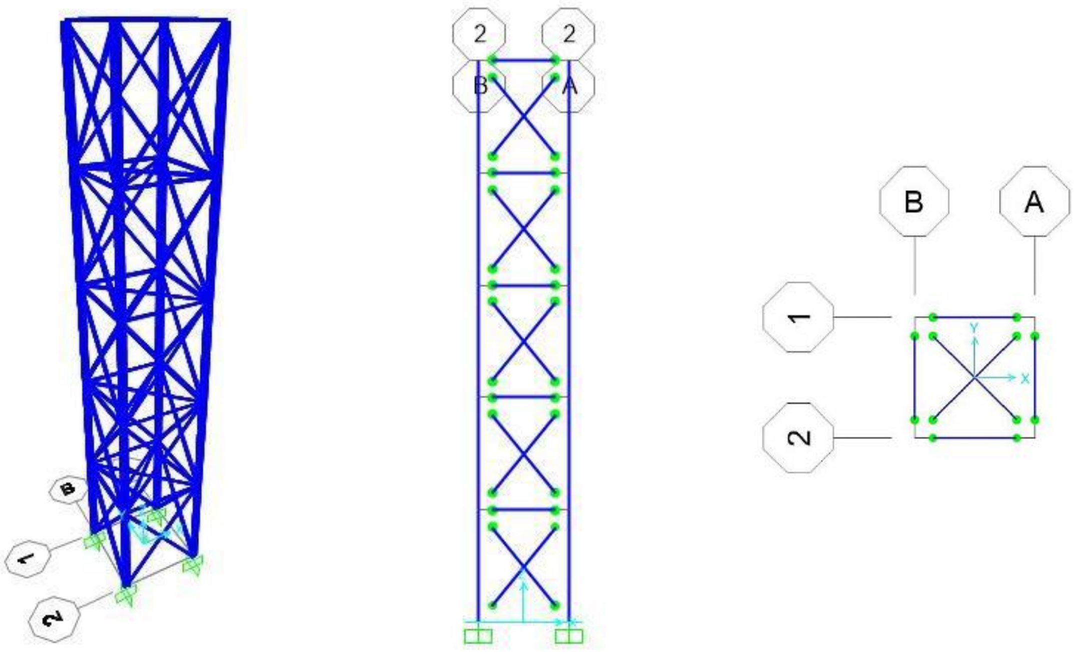
3. Numerical Modelling

To perform the numerical analyses considering the static and dynamic actions recommended by the above-mentioned standards, a fictitious model of a steel lattice telecommunication tower was taken, located in Barroso, Minas Gerais (Brazil).

The tower was modeled in the finite element software SAP2000, student version. A constant square section of 4x4 meters and 25 meters high was considered, discretizing it into 5 panels of 5 meters high each. Following the studies conducted by Carvalho (2015), Oliveira (2019) and Reis (2020), the legs of the tower were modeled with space frame elements and the other elements with space truss. As recommended by ASCE 10-15, all elements could be modeled as a truss, however, this type of structure can present instability by not being locked in the perpendicular planes (REIS, 2020). In addition, all the bases were crimped, and the tower feet were not considered to be supported on flexible bases (soil-structure interaction).

Figure 1a presents the three-dimensional numerical model of the tower, while Figure 1b shows the tower in one of its vertical planes (the four faces are equal) and Figure 1c presents its horizontal plane. In these last two images, it can be observed the difference between the frame and truss elements mentioned above.

Figure 1: Numerical model of the lattice steel tower.

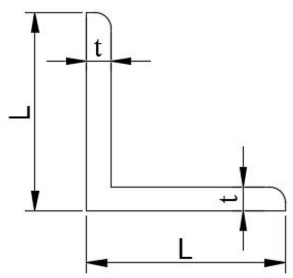


(a) (b) (c)

Source: Authors (2021).

In addition, for all the bars, symmetrical L-type angle brackets were adopted (Figure 2). The dimensions of the width and thickness of the profiles adopted were 102 mm and 6.4 mm, in this order. The profiles are composed of ASTM A-36 carbon steel, which has a modulus of elasticity equal to 200 GPa, Poisson coefficient of 0.3 and specific mass equal to 7850 kg/m^3.

Figure 2: L-type angle section.



Source: Oliveira and Silva (2016).

In this sense, this topic defines the efforts generated by the wind to be applied to the analyzed model. The wind loads were verified according to the static and dynamic analyses defined by the Brazilian standard NBR 6123 and according to the static analysis recommended by the European standard IEC 60826, considering linear and nonlinear analyses for all cases. Besides the wind action, the action of the structure's self-weight was also considered, being submitted to the ultimate normal combination in the analyses, whose increase factors, according to NBR 8800 (ABNT, 2008), are 1.25 and 1.4, for self-weight and wind action respectively.

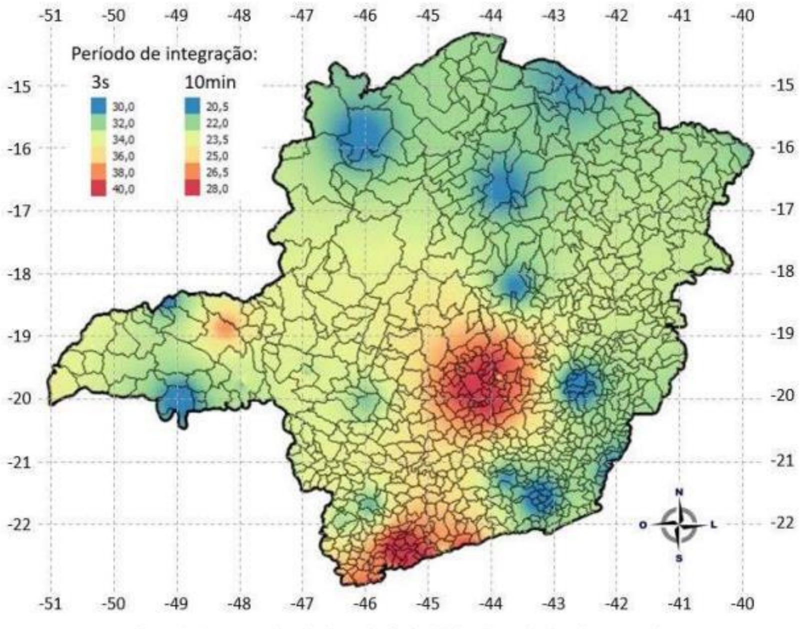
3.1 Static analysis according to NBR 6123

For measuring the static wind loads according to NBR 6123, the equations of this standard were used directly and so they are not explicated in this manuscript.

The basic wind speed (V_0) provided in its isopleth map (ABNT, 1988) was used. For the geographic coordinates of Barroso, Minas Gerais (Brazil), 21º S and 44º W, a value of 34 m/s was assumed (integration period equal to 3 seconds).

In this context, it is worth mentioning the study of the wind climatology of the state of Minas Gerais developed by Reis (2020). In his work, the author proposed a map of basic speeds for the state based on the recommendations of the European standard IEC 60826:2017, considering a database with the wind speeds of the main national meteorological networks, which was properly treated to obtain consistent series. As can be seen in Figure 3, the value of the basic speed adopted in this work (34 m/s) is compatible with that suggested by this author's map, which is more recent and accurate than that provided by NBR 6123.

Figure 3: Heat map of the basic wind speed in Minas Gerais.



Source: Reis (2020).

Besides the topographic factor S_1, equal to 1.0, the terrain roughness factor (S_2) compatible with Category II and Class B terrain and the parameters "b", "Fr" and "p" were obtained directly from NBR 6123 (ABNT, 1988). As this factor (S_2) depends on the height (z) of wind incidence on the structure, it has different values for each panel of the tower. Finally, for the S_3 factor, a value of 1.10 was adopted, because the collapse of this structure can interrupt local telecommunication (ABNT, 1988).

The characteristic wind speed (V_k) and the dynamic wind pressure (q) were obtained for each load application height.

To obtain the wind strength that is acting on the tower, it was necessary to obtain the exposed area index (ABNT, 1988). Thus, the effective area was determined by multiplying the width of the angle profile by its length. The overlap of the profiles at the connections was disregarded, as recommended by the NBR 6123 standard.

The drag coefficient (C_a) was obtained directly from the standard, taking into account the obtained index of the exposed area, arriving at the value of 3.2. Then, the loads acting on each face were determined by applying the protection coefficient η found according to the dimensions of the structure. According to NBR 6123, this value was 0.9, resulting in load

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decomposition factors of 0.53 for the face I (first face perpendicular to the wind) and 0.47 for face III (second face perpendicular to the wind).

Finally, the static wind load was calculated (ABNT, 1988). It was applied to the two upper nodes of each panel, incident at 90º. It is noteworthy that the wind at 0º was not considered, since the tower is doubly symmetrical, i.e., its magnitude would be the same at 90º.

Figure 4 presents a scheme of the static loading, in which the loads presented in Table 1 are applied to faces I and III, in the Y-Z plane. It should be noted that each load is applied to two nodes, so in Figure 4 (vertical view), the values shown represent half of the total value. Table 1 presents the parameters necessary to obtain the static wind loads according to NBR 6123 and the respective values of these loads.

Table 1: Parameters used for measuring the static wind loads according to NBR 6123.

Panel	**z (m)**	**S_2**	**V_k (m/s)**	**q(z) (N/m²)**	**F_a (kN)**	**$F_{a,face-I}$ (kN)**	**$F_{a,face-III}$ (kN)**
1	5	0.92	34.44	726.89	6.36	3.37	2.99
2	10	0.98	36.65	823.49	7.20	3.82	3.39
3	15	1.02	38.01	885.83	7.75	4.11	3.64
4	20	1.04	39.01	932.91	8.16	4.33	3.84
5	25	1.06	39.80	971.15	8.50	4.50	3.99

Source: Authors (2021).

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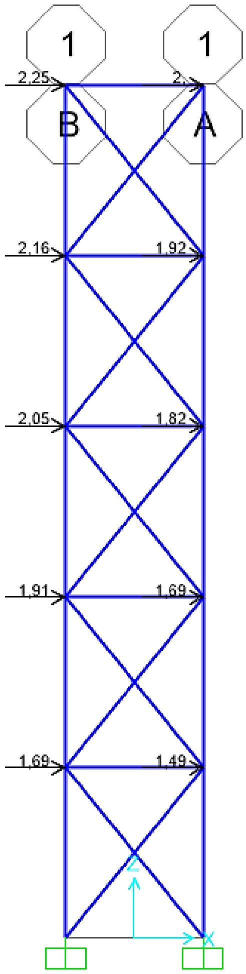
Figure 4: Static loading diagram according to NBR 6123.



Source: Authors (2021).

3.2 Dynamic analysis according to NBR 6123

For measuring the dynamic wind loads according to NBR 6123, the equations of this standard were used directly and so they also are not explicated in this manuscript.

To determine the dynamic response of the structure, the simplified continuous model was adopted, since the adopted model is less than 150 meters high and has a constant section. According to the NBR 6123 standard (ABNT, 1988), this method, which uses only one vibration mode, leads to results with errors lower than 10%. In this work, the vibration mode was extracted directly from the SAP2000 software, resulting in 0.24255 seconds of the period and 4.12288 Hz of frequency.

The determination of the design speed (V_p) was performed through a process similar to the static method, using the same factors S1 and S3, but for this analysis, the basic wind (V0) was determined for the integration period equal to 10 minutes, as determined by the standard.

The value of the dynamic pressure (q) was calculated according to the standard, from which the values of the coefficients γ, ζ, and ξ were obtained.

Finally, the value of the wind load acting on the structure results from the product between the pressure q(z), the width of the structure (l_1), and the drag coefficient (C_a). The width used was the sum of the widths of the tower profiles, which when multiplied by the pressure and the drag coefficient resulted in a load distributed along the lengths of the bars.

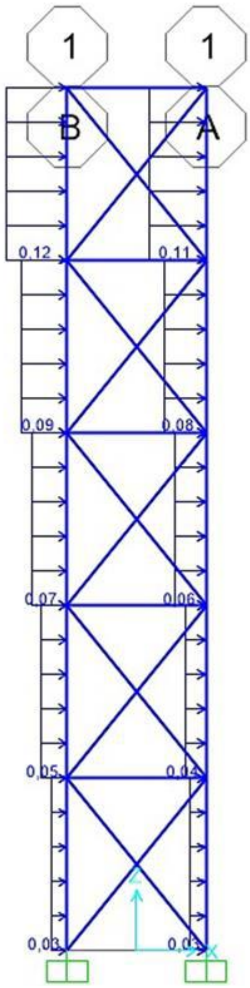
Table 2 presents the loads found for the dynamic analysis, while Figure 5 shows the loading scheme of the incident wind on the structure.

Table 2: Dynamic wind loads according to NBR 6123.

Panel	z (m)	q(z) (N/m²)	F_a (kN)	$F_{a,face-I}$ (kN)	$F_{a,face-III}$ (kN)
1	5	187.52	0.06	0.03	0.03
2	10	289.61	0.09	0.05	0.04
3	15	408.69	0.13	0.07	0.06
4	20	547.41	0.18	0.09	0.08
5	25	705.80	0.23	0.12	0.11

Source: Authors (2021).

Figure 5: Dynamic loading diagram according to NBR 6123.



Source: Authors (2021).

3.3 Static analysis according to IEC 60826

For measuring the static wind loads according to IEC 60826, the equations of this standard were used directly and so they are not explicated in this manuscript.

The methodology proposed by the European standard IEC 60826 for static analysis of lattice structures subjected to the wind is similar to that of NBR 6123. In this work, we chose method 1 specified in this standard (IEC, 2017), because this method requires the tower to be discretized into different panels, resembling the previous analyses.

The IEC 60826 standard recommends that the value of the reference speed (V_R) be obtained from a database of the main weather stations present at the structure site. Therefore, for the reference speed, the map in Figure 3 was used,

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considering the average speed for 10 minutes. Thus, a speed equal to 23.5 m/s was adopted. As a category B terrain was considered, the terrain roughness factor (K_R) is equal to 1.0. Thus, the reference speed for this category (V_{RB}) was equal to V_R.

Next, the dynamic wind pressure was calculated (IEC, 2017). The correction factor τ equal to 0.89 was adopted, considering an altitude of 1000 meters and an ambient temperature of 15 ºC.

The value of the coefficient χ resembles the exposed area index (φ) of NBR 6123, having the same value, 0.14. The drag coefficient of 3.2 was obtained directly from the standard, as well as the G_t coefficient, obtained for each height of load application (IEC, 2017).

Finally, the wind load (F_a) was calculated for each height of the different tower panels, whose values are given in Table 3. These values were divided equally between the two nodes at the top of each panel and each node of the structure in the direction of wind incidence, as shown in Figure 6.

Table 3: Static wind loads according to IEC 60826.

Panel	**z (m)**	**G_t**	**F_a (kN)**	**$F_{a,faceI}$ (kN)**	**$F_{a,faceIII}$ (kN)**
1	5	1.9	5.00	2.50	2.50
2	10	1.9	5.00	2.50	2.50
3	15	2.1	5.53	2.77	2.77
4	20	2.2	5.79	2.90	2.90
5	25	2.3	6.06	3.03	3.03

Source: Authors (2021).

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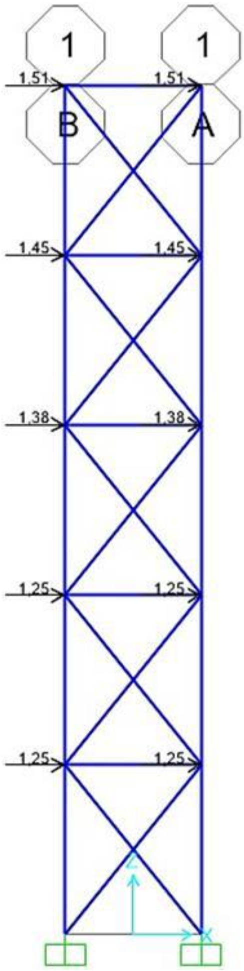
Figure 6: Static loading diagram according to IEC 60826.



Source: Authors (2021).

3.4 Additional parametric studies

In order to better understand the behavior of the structure facing the static and dynamic analyses, a parametric study was performed using the same finite element computational model, varying only the height of the tower. In other words, it was intended to evaluate whether the type of response of the structure can be influenced by its height, since, as shown earlier, the wind loads vary with height. Therefore, two other models were analyzed, with 50 and 100 meters high. The calculation of the efforts of these cases was done in the same way as those previously presented. Therefore, for these studies, only the results are presented.

4. Results and Discussion

For all the cases presented above, linear and nonlinear analyses were performed. The interpretation of the structural behavior of the tower was made from the responses obtained numerically, namely: support reactions in the direction of force (F_x), moments about the axes X (M_x) and Y (M_y) at the base, and maximum displacement of the tower (d_{max}), at its highest point. The results are shown in Table 4. Table 5 presents the relative differences between the analyses, using as reference parameter the NBR 6123 static linear analysis. The results obtained in the complementary parametric studies, in turn, are presented separately in Table 6 since they portray atypical situations (telecommunication towers up to 100 meters high).

Table 4: Results of the numerical analysis of the lattice tower subjected to wind.

Case		F_x (kN)	M_x (kN.m)	M_y (kN.m)	d_{max} (cm)
Static analysis (NBR 6123)	Linear	-53.14	-7.62	-821.19	2.79
	Nonlinar	-53.14	-7.63	-821.99	2.79
Dynamic analysis (NBR 6123)	Linear	-25.52	-3.78	-401.4	1.38
	Nonlinar	-25.52	-3.79	-401.8	1.38
Static analysis (IEC 60826)	Linear	-38.30	-5.42	-585.72	1.98
	Nonlinar	-38.30	-5.43	-586.29	1.98

Source: Authors (2021).

Table 5: Relative differences of the analyses in relation to the NBR 6123 static linear analysis.

Case		F_x	M_x	M_y	d_{max}
Static analysis (NBR 6123)	Linear	-	-	-	-
	Nonlinar	0.00%	0.24%	0.10%	0.00%
Dynamic analysis (NBR 6123)	Linear	-51.98%	-50.41%	-51.12%	-50.54%
	Nonlinar	-51.98%	-50.29%	-51.07%	-50.54%
Static analysis (IEC 60826)	Linear	-27.92%	-28.88%	-28.67%	-29.03%
	Nonlinar	-27.92%	-28.71%	-28.61%	-29.03%

Source: Authors (2021).

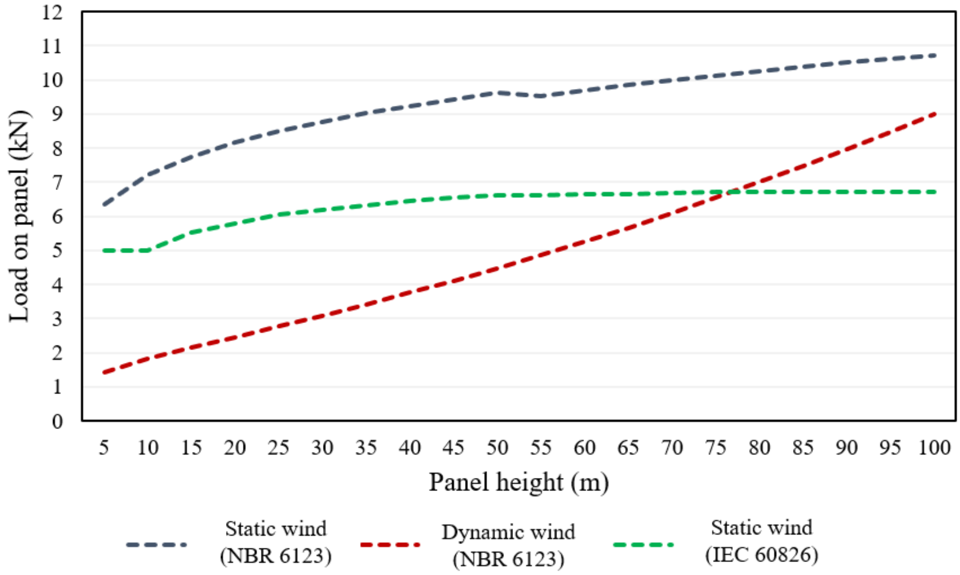
Table 6: Results of the additional studies (50 and 100 meters high).

Height (m)	Case		F_x	M_x	M_y	d_{max}
50	Static analysis (NBR 6123)	Linear	-112.6	-102.0	-3125.4	39
		Nonlinar	-112.6	-103.7	-3145.5	39
	Dynamic analysis (NBR 6123)	Linear	-54.8	-55.0	-1631.5	21
		Nonlinar	-54.8	-55.9	-1642.3	21
	Static analysis (IEC 60826)	Linear	-134.9	-123.9	-3772.6	47
		Nonlinar	-134.9	-126.0	-3797.8	47
100	Static analysis (NBR 6123)	Linear	-393.6	-1834.6	-18909.1	846
		Nonlinar	-393.6	-2032.8	-19681.5	889
	Dynamic analysis (NBR 6123)	Linear	-137.0	-742.7	-7263.9	343
		Nonlinar	-137.0	-826.7	-7585.7	362
	Static analysis (IEC 60826)	Linear	-315.7	-1439.3	-14956.9	664
		Nonlinar	-315.7	-1601.7	-15584.1	699

Source: Authors (2021).

Analyzing the results presented, it is evident that the loads resulting from the static analysis of NBR 6123 present values significantly higher than the other analyses. On the other hand, it is notable that the growth of the loads as a function of height variation is greater in the dynamic case, exceeding the static load of IEC 60826 from approximately 75 meters, as shown in the graph of Figure 7. These results show that for a structure with more relevant vertical dimensions, the dynamic analysis presents larger reactions at the base, which corroborates the results obtained in the work developed by Almeida and Vidotto (2013).

Figure 7: Forces as a function of height.



Source: Authors (2021).

Regarding the linear and nonlinear analyses, it is noted that both presented very similar results at lower heights. This fact can be attributed to the small maximum displacement verified at the top of the tower, since the structure is very rigid and not susceptible to the increase of stresses due to geometric nonlinearities, arising from the deformed position of the structure itself, a fact that becomes untrue at elevated heights, due to the exacerbated slope of the structure. Moreover, the probable cause for the occurrence of large displacements verified in the 50 and 100-meters high towers, especially the latter, may be associated with the high wind intensity at these heights and the increase in the vertical dimension of the tower not accompanied by the use of more robust profiles for the legs.

5. Conclusions

This paper presented a comparative study between the static and dynamic analyses proposed by the Brazilian standard NBR 6123 and the European standard IEC 60826 of telecommunication towers subjected to wind loads. From numerical computational models and parametric studies, carried out in the finite element software SAP2000, responses of the structure were obtained and showed the importance of considering the wind in the most correct way possible, for each specific case.

For smaller towers, it is notable the superiority of the responses coming from the static analysis proposed by NBR 6123 and, consequently, the structure's reactions were also considerably higher. On the other hand, it is observed that the increase in the height of the tower leads to an increase in the dynamic analysis responses, which become significant, surpassing the results obtained through the IEC 60826 standard from 75 meters on. Furthermore, it is evident that, with a continuous increase in the vertical dimensions of the tower, the dynamic analysis will eventually overcome the static one, a behavior that was also found by other works in the literature.

With this perspective, the results highlight the importance of a detailed study during the design of the structure. As an example, a 25-meter tower subjected to the static wind of the Brazilian standard will probably be in favor of the safety criteria, but for a 150-meter tower, it may be necessary to analyze the wind dynamically. The European standard, in turn, showed more consistent results, with a smaller variation as a function of height, which may justify its increasingly frequent use even in structures built in Brazil.

Another important point of observation is the comparison between linear and nonlinear analysis. For smaller and, therefore, stiffer models, the variation in internal forces from these two analyses was insignificant, which is due to the low displacement of the top of the structure. As the height increases, the displacements become predominant, and second-order forces arise, as shown in the nonlinear analysis, which must be considered when designing the structure.

For future research, it is suggested to evaluate the geometric nonlinear behavior of other topological alternatives of telecommunication structures, using a real structure to validate the model, as well as studying the influence of the tower stiffness on the dynamic response and including the soil-structure interaction in the analyses.

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