Dynamic structural analysis of transmission lines steel towers subjected to nondeterministic wind loadings

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Abstract. In the current design practise of steel latticed towers used to support electrical transmission lines, the structure's dynamic behaviour is not considered. However, the main loading to be taken into account in the structural analysis of electrical transmission lines steel towers is produced by the wind loads, which acts dynamically over the structural system composed by towers and cables. In addition, it's not uncommon for slender towers to present disadvantageous dynamic properties, making them vulnerable to the wind action. Considering that many accidents associated to this kind of structure occur even for wind velocities below that specified in project, it's possible that most of these accidents have been produced by dynamic actions. This research work proposes an analysis methodology that can accurately simulate the coupled behaviour between the transmission line cables and the suspension structures, when subjected to wind nondeterministic actions, including in the dynamic analysis the effects of the geometric nonlinearity and the aerodynamic damping. The results obtained in this work indicated that the dynamic response can be relevant to the system structural behaviour, and in this scenario the use of a static analysis can lead to a non-trustable design of the towers.

Keywords: latticed steel towers, power transmission lines, nonlinear dynamic analysis.

1 Introduction

Lattice steel towers have a significant importance as supports for overhead power transmission lines. The stability of the structural system is crucial to the perfect functioning and electrical safety of transmission systems [1]. In current day-to-day practice, the project of lattice steel towers used for power transmission lines considers the first-order elastic structural analysis, assuming static equivalent loads related to the own weight, the transmission line components (conductor, shield wires and insulators) and the wind action [2].

It is widely recognized that a second-order elastic structural analysis provides additional structural displacements and imposing members forces in addition to those computed in a first-order elastic analysis. Consequently, performing a second-order elastic analysis may show that towers will be subjected to additional displacements and additional forces [3]. Additionally, the dynamic characteristic of the wind action is essential for a more realistic analysis based on the use of the Spectral Representation Method (SRM). With this purpose, wind series can be generated with the wind fluctuant part determined as a sum of a finite number of harmonics with randomly generated phase angles. This way, a power spectrum and a coherence function can be used to calculate the amplitude of each harmonic, aiming to keep the resemblance to the natural wind [4].

This way, in this research work the series of nondeterministic wind dynamic loads can be used to assess the structure nonlinear geometric response, based on the displacements and forces values. Therefore, the main objective of this study is to develop an investigation regarding the structural behaviour of lattice steel towers, in order to evaluate displacements and member forces acting in the suspension tower, comparing with the expected values indicated at current design practice methodologies. Therefore, a transmission line system section, comprising a suspension tower and two spans with total length of 900m was analysed in this work, based on the use of three different developed analysis methodologies (see Tab. 1).

Table 1. Performed structural analysis: static and dynamic.

Model	Structural model	Wind loads	Analysis
Model I	Isolated steel tower	Equivalent static [5]	Linear static
Model II	Transmission line system	Equivalent static [5]	Geometric nonlinear static
Model III	Transmission line system	Nondeterministic wind loads	Geometric nonlinear dynamic

2 Investigated structural model

The analysed structural model and transmission system characteristics, including conductor and shield wire types were extracted from a simple circuit transmission line presented on the study by Oliveira in 2006 [6]. The studied section of the transmission line system presents two spans of 450m each one (see Fig. 1), comprehended a main suspension tower in the centre with total height of 32.86m (see Fig. 2), and other two towers at the ends. The cross sections of the main suspension tower present rectangular base, pyramidal body and hollow configuration at the top, where the phases and the shield wires were fixed. Angle profiles and steel ASTM A36 type were used in this structural system.

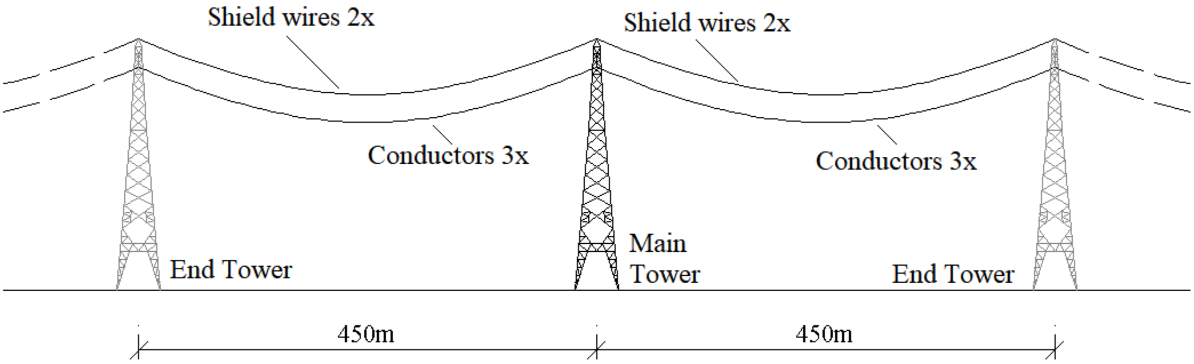


Figure 1. Investigated structural system [6].

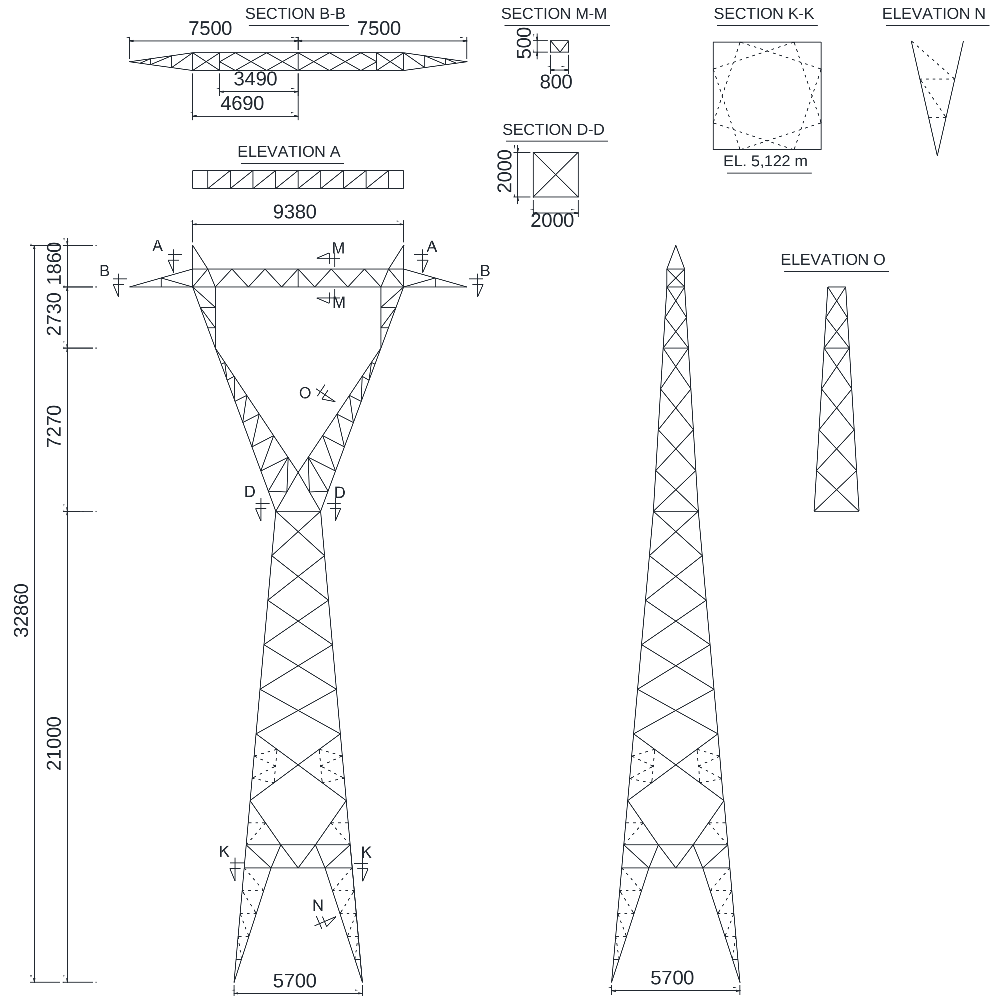


Figure 2. Main steel tower (dimensions in mm).

3 Finite element modelling

In this work, the transmission line system was modelled based on the use of the Finite Element Method (FEM), utilising the ANSYS software. The beam finite element BEAM4 was used for modelling the main steel tower, the truss finite element LINK8 was utilised to represent the insulators, the beam finite element BEAM189 was used for simulate the conductors and shield wires, and the BEAM188 was adopted to modelling the end towers (see Fig. 3). In this investigation, the cables were represented based on the use of BEAM189 finite elements, having in mind the complexity of the numerical modelling due to the cables low stiffness against bending and compression forces. The boundary conditions were applied to the nodes that represent the towers foundations, considering restrictions to the horizontal translational displacements related to the three global axes. The developed finite element model is illustrated in Fig. 3.

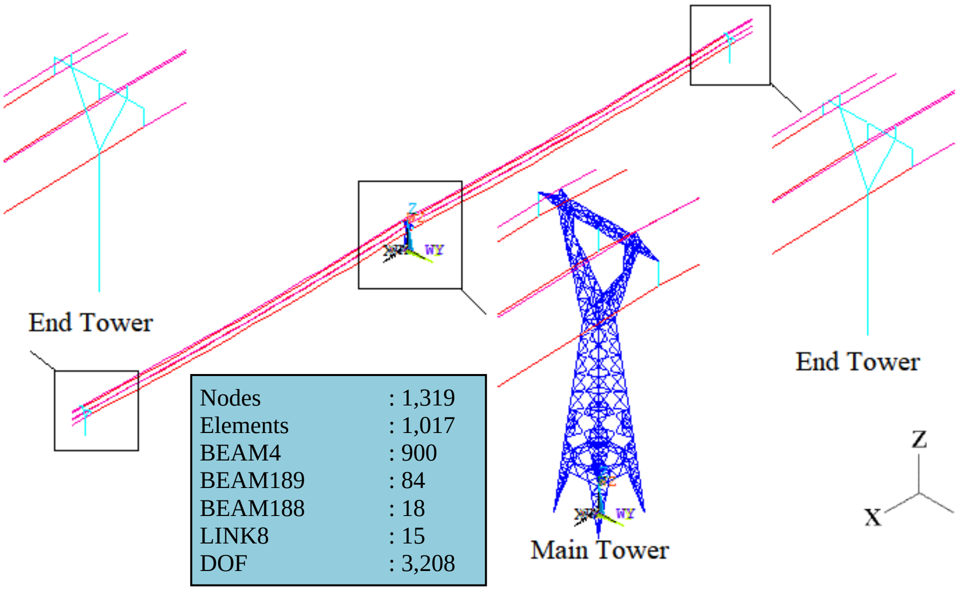


Figure 3. Finite element model of the investigated structural system.

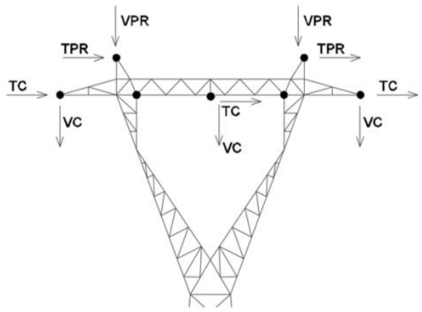
4 Developed structural analyses

Initially, the free vibration analysis of the isolated steel tower resulted in a fundamental frequency equal to 2.60Hz (f_{01} = 2.60Hz: steel tower fundamental vibration mode). However, when the full transmission line system (steel tower and cables) was considered in the free vibration analysis, the calculated fundamental frequency was equal to 0.153Hz (f_{01} = 0.153Hz: cables and steel tower fundamental vibration mode). It was concluded that the cables (conductors and shield wires) have influenced significantly the first vibration modes of the transmission line system. Considering that the conductors, shield wires and insulator chain present a relatively elevated weight (elevated mass) when compared with their low stiffness, the influence of the cables on the transmission line non-linear dynamic behaviour is very important.

After that, the linear elastic analysis was performed to Model I and nonlinear geometric analysis to Models II and III. Basically, the geometric nonlinearity was included in the structural analysis based on the total Lagrangian formulation, which allows large displacements and rotations, and the Newton-Raphson method was utilised. Considering the Model III, the Newmark's time integration method was adopted for the solution of the dynamic equilibrium equations. The Newton-Raphson method was used along with Newmark's formulation. This strategy for solving the nonlinear equations is based on the implicit time integration method, which despite being more complicated in terms of calculation, is the most appropriate, given the problem high nonlinearity.

The load hypotheses are related to the forces imposed on the system associated to the basic wind velocity acting at 0º with the line direction. Considering the Model I, the loads related to the cables, shield wires and insulators were applied to the attachment points of the main tower (see Fig. 4), and calculated based on the use of the Brazilian standard NBR 5422 "Design of overhead power transmission lines" (in Portuguese) [7]. The displacement at point A and forces in element B was determined (see Fig. 5).

The wind loads applied on the main tower (Model I), and the transmission line system (Model II), were determined based on the use of the Brazilian standard NBR 6123 “Forces due to wind on buildings” (in Portuguese) [5] (see Fig. 6). The nondeterministic dynamic wind loads applied on the Model III (see Fig. 6) were modelled by an aleatory process based on the statistical properties. This way, the nondeterministic wind load series were generated using the Spectral Representation Method (SRM) [1,4,6]. In this study, four wind velocities (υ = 50 m/s, υ = 45 m/s, υ = 40 m/s and υ = 35 m/s) were considered and selected based on significant wind velocities applied to Brazilian transmission lines regions, with mean of 3s, height at 10 meters from the ground, and return period of 50 years [5]. The wind series were generated as lagged random series from a time interval τ, calculated from the use of the autocovariance and covariance functions [1,4,6].



T_C: Transversal load (conductor)
T_{PR}: Transversal load (shield wire)
V_C: Vertical load (conductor)
V_{PR}: Vertical load (shield wire)

Figure 4. Loads: conductors and shield wires (Model I).

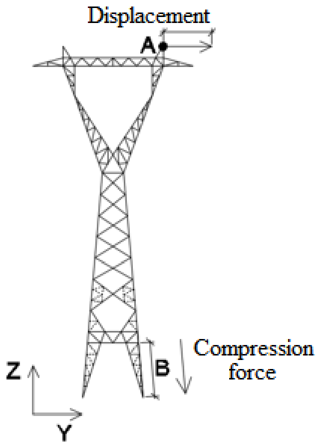
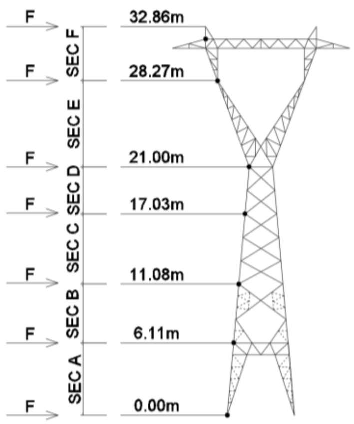
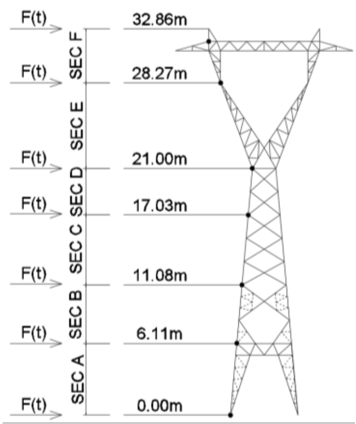


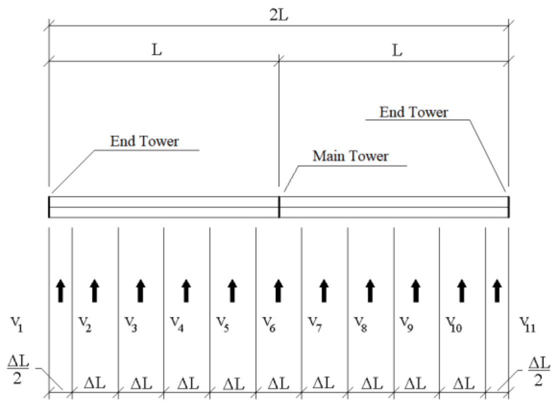
Figure 5. Calculated displacement and force.



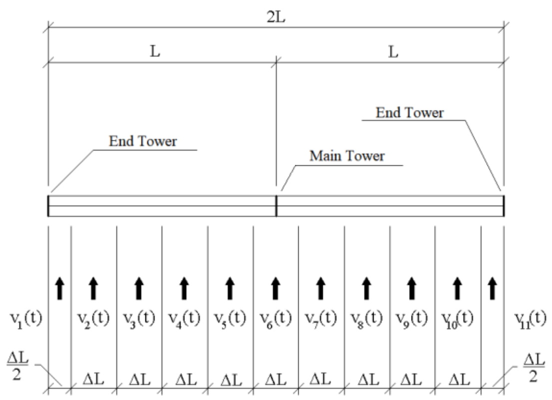
a) Static equivalent wind loads applied on the main tower: Models I and II.



b) Nondeterministic dynamic wind loads applied on the main tower: Model III.



c) Static equivalent wind loads applied on the conductors and shield wires: Model II.



d) Nondeterministic dynamic wind loads applied on the conductors and shield wires: Model III.

Figure 6. Definition of the applied wind loads: static equivalent and nondeterministic wind loads.

In sequence, Fig. 7 presents a typical example of the tower displacement in time domain, when subjected to non-deterministic dynamic wind loads. Figure 8 illustrates this displacement in frequency domain determined through Fast Fourier Transform (FFT), where it is possible to see the displacement amplitude associated to the fundamental frequency of the transmission line system [f_{01} = 0.153 Hz: 1st vibration mode (Model III)].

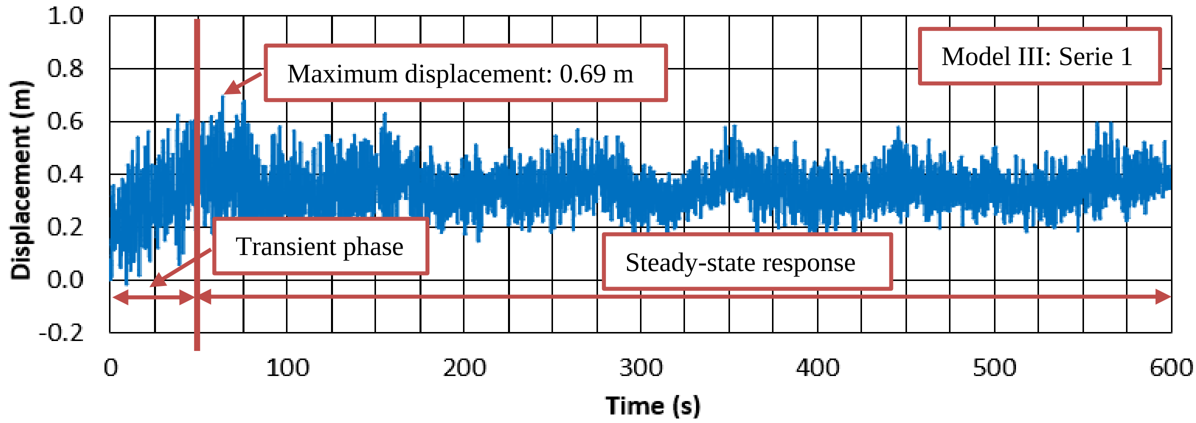


Figure 7. Typical horizontal translational displacement on point A (see Fig. 5): time domain

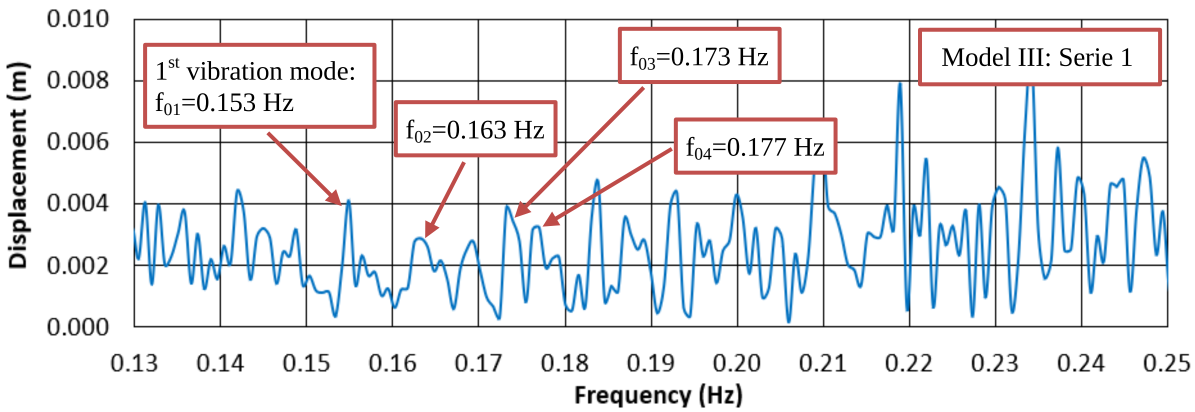


Figure 8. Typical horizontal translational displacement on point A (see Fig.): frequency domain

The horizontal translational displacement at the main tower structural section A (see Fig. 5) and compression force acting on member B (see Fig. 5) determined based on the use of the Mode I (linear static analysis) and Model II (nonlinear static analysis) are presented in Tab. 2.

On the other hand, Tab. 3 and Table 4 present the statistical analysis of the investigated structural system dynamic response [mean values (μ); standard deviation (σ); reliability index ($D_{95\%}$ and $F_{95\%}$)], associated to the horizontal translational displacement at the section A (see Fig. 5) and compression force related to the structural element B (see Fig. 5), respectively, calculated considering ten series of nondeterministic wind loads based on the use of the Model III.

It must be emphasized that the element B structural capacity (see Fig. 5), which was calculated equal to 242kN, was determined according to the Brazilian standard NBR 8850 "Design and execution of lattice steel towers for transmission lines - Procedure" (in Portuguese) [8]. This way, is possible to assess the investigated member capacity ratio based on the results related to the reliability index ($F_{95\%}$) (see Tab. 4 and Tab. 5).

Table 2. Displacement at point A and compression force on member B (see Fig. 5): Model I and Model II.

Models	Model I				Model II			
Velocity υ (m/s)	50m/s	45m/s	40m/s	35m/s	50m/s	45m/s	40m/s	35m/s
Displacement (m)	0.29	0.23	0.19	0.15	0.29	0.23	0.19	0.15
Force (kN)	165	137	112	94	167	140	117	96

Table 3. Horizontal translational displacements in (m) at point A (see Fig. 5): Model III.

Series	υ = 50m/s	υ = 45m/s	υ = 40m/s	υ = 35 m/s
1	0.693	0.615	0.470	0.388
2	0.745	0.544	0.489	0.396
3	0.727	0.562	0.502	0.341
4	0.738	0.612	0.497	0.331
5	0.776	0.569	0.461	0.350
6	0.775	0.537	0.501	0.360
7	0.737	0.587	0.491	0.355
8	0.736	0.623	0.522	0.397
9	0.830	0.652	0.493	0.361
10	0.775	0.619	0.463	0.382
μ	0.753	0.592	0.489	0.366
σ	0.0353	0.0363	0.0183	0.0221
$U_{95\%}$	0.775	0.615	0.500	0.380

Table 4. Compression forces in (kN) acting on member B (see Fig. 5): Model III.

Series	υ = 50m/s	υ = 45m/s	υ = 40m/s	υ = 35 m/s
1	416	372	279	232
2	452	323	298	236
3	458	331	290	208
4	458	383	298	196
5	474	341	271	209
6	474	316	304	214
7	452	355	297	208
8	448	373	318	234
9	513	395	298	210
10	484	382	273	222
μ	463	357	293	217
σ	24.36	26.41	13.74	12.73
$F_{95\%}$	478	374	301	225

Table 5. Assessment of the load capacity of the investigated structural element B (see Fig. 5).

Models	Member force ratio (%)			
	υ = 50m/s	υ = 45m/s	υ = 40m/s	υ = 35m/s
I	68	57	46	39
II	69	58	48	40
III	197	154	124	93

It should be noted that the static structural analysis (Model I and Model II) provided lower values of displacements and compression forces when compared to those determined based on the dynamic structural analysis (Model III) (see Tab. 2 to Tab. 5). This way, the dynamic amplification factor (DAF) related to displacements and compression forces of the investigated models are approximately three (DAF = 3). The differences between the results calculated based on the use of Models I and II are not significant (see Tab. 2).

On the other hand, it can be seen from Table 5 results, that the differences between the models response (Model I; Model II; Model III), in terms of members force ratio could be relevant and up to 130%. It must be emphasized that according to the Brazilian standard NBR 8850 [8], the admitted maximum force ratio is equal to 93%, and clearly the results provided by Model III have surpassed this limit (see Tab. 5).

The results obtained in this investigation reflect relevant differences between the displacement and force values according to the chosen finite element model and structural analysis. It is important to emphasize that the structural member's capacity analysis shows that the maximum member force ratio is equal to 197% [$F_{95\%}$ = 478 kN > 242 kN] (see Tab. 5). This member force ratio value is enough to surpass the structural member capacity determined by NBR 8850 [8] and could cause structural failure.

5 Conclusions

The final conclusions on this research work are presented based on the structural response assessment of a transmission line system section comprising a suspension tower and two spans with total length of 900m, based on the development of three different analysis methodologies: static linear analysis considering the main isolated tower (Model I); static geometric nonlinear analysis based on a transmission line system section (Model II); geometric nonlinear dynamic analysis associated to a transmission line system section (Model III). This way, the following conclusions can be drawn from the results presented in this study:

1. The results have shown relevant quantitative differences between the displacement and force values established by the design standards and those calculated through a geometric nonlinear dynamic analysis. Based on the comparisons between the results calculated from Model I (static linear analysis), Model II (static geometric nonlinear analysis) and Model II (geometric nonlinear dynamic analysis), it is possible to verify differences: up to 267% (displacements), 290% (member's compression forces), and 129% (member force ratio).

3. It is important to notice that the structural member's capacity analysis shows that the force ratio increase is enough to surpass the structural member capacity, when the Model III (geometric nonlinear dynamic analysis) was considered, as result of the differences between the forces provided by the standard methodology and those obtained from the finite element analysis.

4. This paper revealed that the geometric nonlinear dynamic analysis is important to understand the structural behaviour, loads distribution, structural stability and design of transmission lines. This work considered a case study, based on four wind velocities (50m/s, 45m/s, 40m/s and 35m/s), which can be used as a reference for similar studies, highlighting the importance of considering the wind dynamic effects on the design of transmission lines.

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