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Structural Response Analysis of Transmission Lines Steel Towers when Subjected to Nondeterministic Wind Loadings

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Abstract

The lattice steel towers have been widely used as supports for power transmission lines. In the current project practise, the structure's dynamic behaviour usually is not considered. 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 way, this investigation proposes an analysis methodology that can accurately simulate the coupled behaviour between the transmission line cables and towers, when subjected to wind nondeterministic loadings, aiming to assess the displacements and forces in the steel towers. The investigated transmission line system, including the steel towers, conductors and shield wires, presents two spans of 450 m with a 32.86 m height main suspension tower in the centre and two towers at the ends. The conclusions pointed out to quantitative differences associated to the structural response when calculated based on a static linear analysis and compared to the results considering a geometric nonlinear and nondeterministic dynamic analysis.

Keywords: latticed steel towers, power transmission lines, dynamic analysis, nondeterministic wind action, finite element modelling, structural behaviour.

1 Introduction

The lattice steel towers present a very relevant importance as supports for overhead power transmission lines. It is well known that 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].

On the other hand, the main loading to be taken into account in the structural analysis of electrical transmission lines steel towers is produced by the wind loadings, which acts dynamically over the structural system composed by towers and cables [1-5]. In addition, it's not uncommon for slender towers to present disadvantageous dynamic properties, making them vulnerable to the wind action. Having in mind, 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 [1,5].

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) [1,4,6]. The 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. Thus, 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 [1,6].

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, aiming to assess the displacements and member forces acting in the suspension tower, comparing with the expected values indicated in current design practice methodologies. Therefore, a transmission line system section, comprising a suspension tower and two spans with total length of 900 m was analysed, based on the use of three different developed analysis methodologies (see Table 1).

Model	Structural model	Wind loads	Structural analysis
I	Isolated steel tower	Equivalent static [7]	Linear static
II	Transmission line system	Equivalent static [7]	Geometric nonlinear static
III	Transmission line system	Nondeterministic loads	Geometric nonlinear dynamic

Table 1: Performed structural analysis: static and dynamic.

2 Investigated Structural Model

The investigated transmission line system, including the steel towers, conductors and shield wire types were extracted from the study previously developed by Oliveira [8]. The analysed section of the transmission line system presents two spans of 450 m (see Figure 1), with a main suspension steel tower in the centre presenting total height of 32.86 m (see Figure 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 [8].

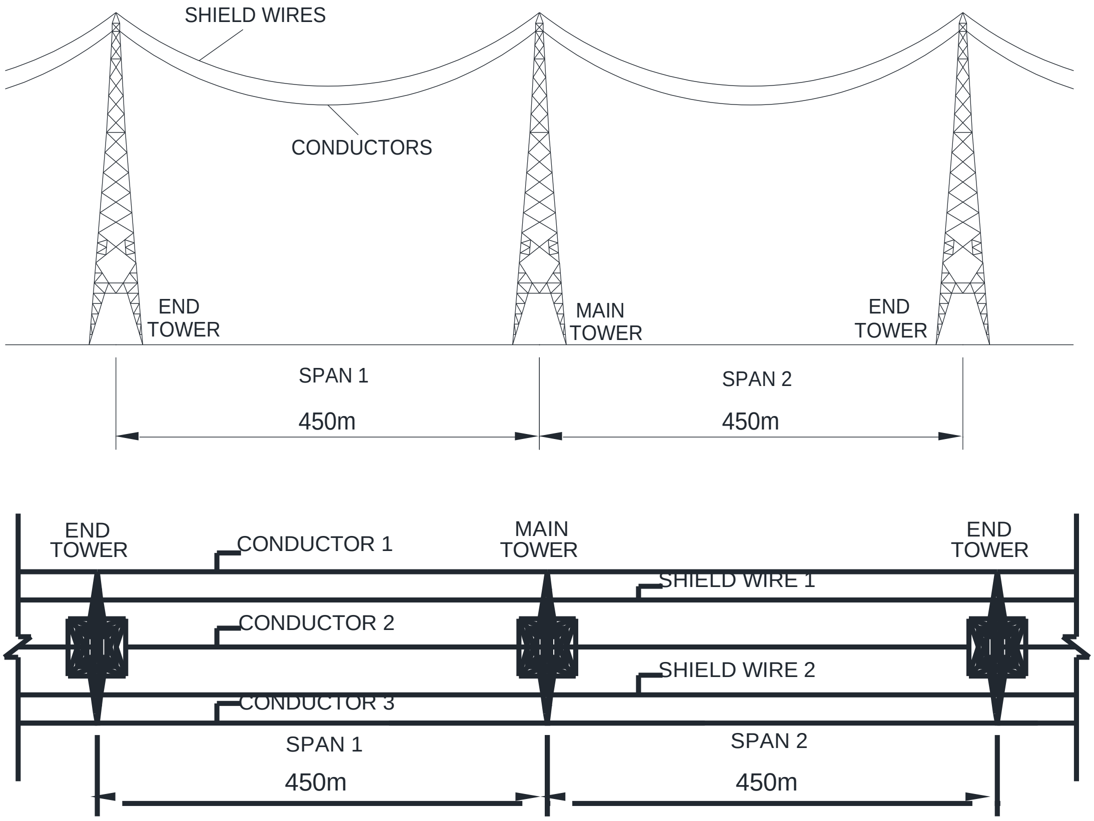


Figure 1: Investigated structural system.

3 Finite Element Modelling

In this research work, the transmission line system was modelled based on the use of the Finite Element Method (FEM), utilising the ANSYS software. This way, 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.

The finite element BEAM188 was utilised to simulate the end towers, and the linear spring finite element COMBIN14 was used to represent the transmission line continuity (see Figure 3). In this investigation, the cables were represented based on the use of BEAM188 finite elements, having in mind the complexity of the finite element numerical modelling due to the cables low stiffness against bending and compression forces.

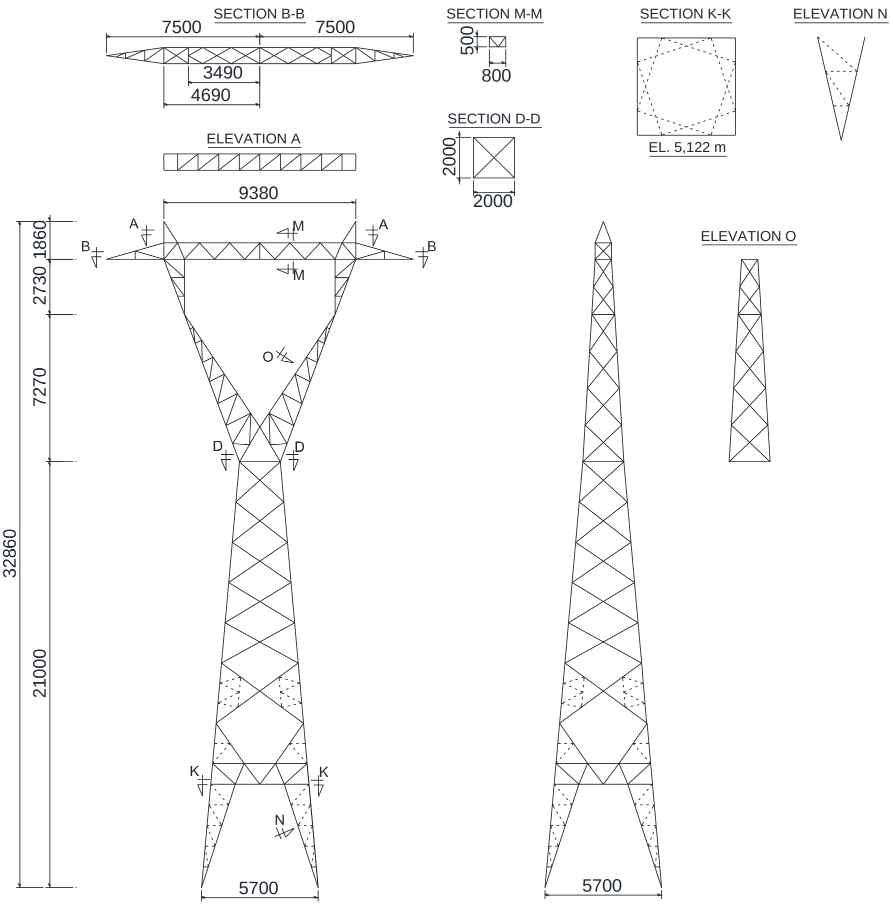


Figure 2: Main steel tower (dimensions in mm).

The numerical model utilised the substructuring technique to replicate the elastic, inertial, and kinematic properties of the end towers. Substructuring condenses a set of finite elements into a single matrix element, known as a superelement. 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 Figure 3.

4 Structural Analysis and Results Discussion

The free vibration analysis of the isolated steel tower resulted in a fundamental frequency of 2.60Hz ($f_{01} = 2.60$Hz: steel tower fundamental vibration mode). However, when the full transmission line system (steel tower and cables) was considered in this analysis, the calculated fundamental frequency was equal to 0.153Hz ($f_{01} = 0.153$Hz: 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 relevant.

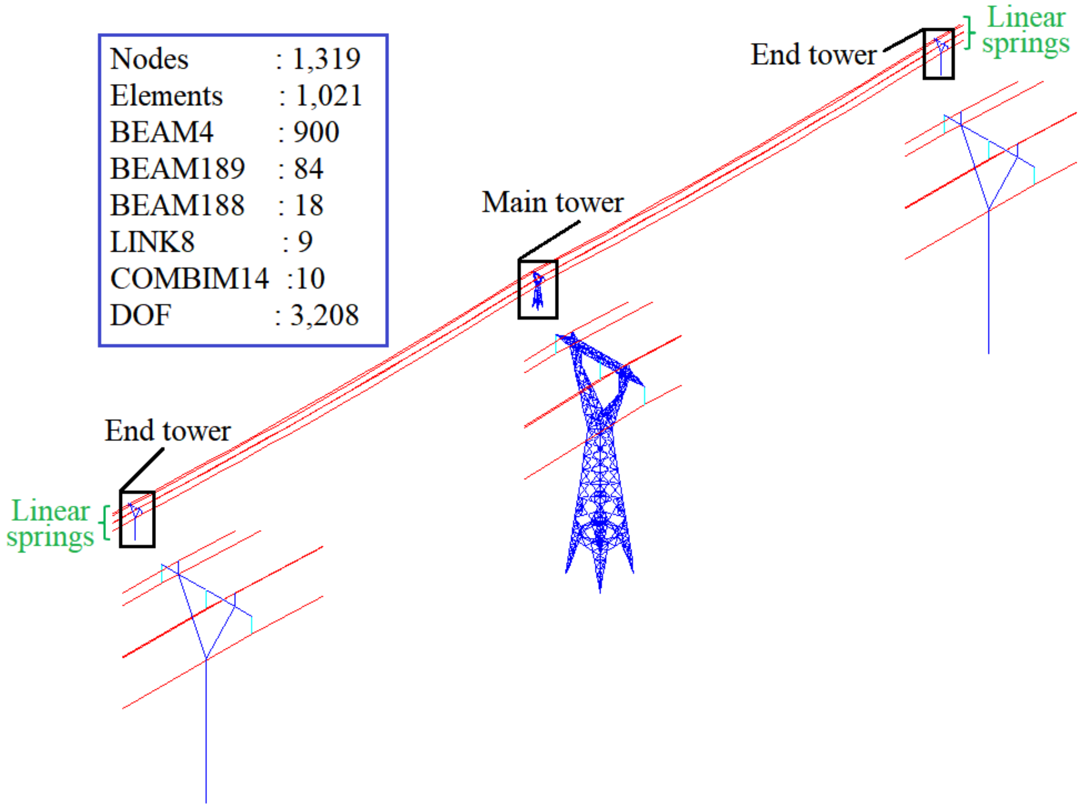
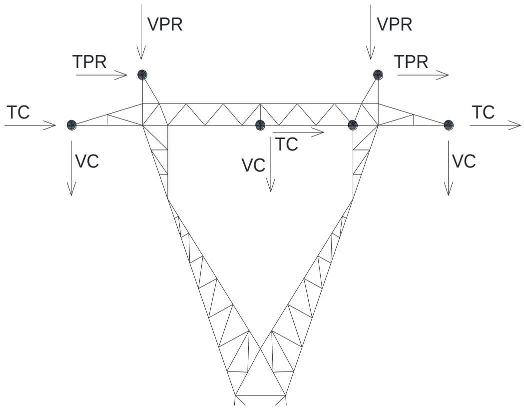


Figure 3: Finite element model of the investigated structural system.

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.

Having in mind the dynamic analysis performed based on the use of 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.

In this work, 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 Figure 4), and calculated based on the use of the Brazilian standard NBR 5422 entitled "Design of overhead power transmission lines" (in Portuguese) [9]. The displacement at structural section A and forces in element B was determined (see Figure 5).



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

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

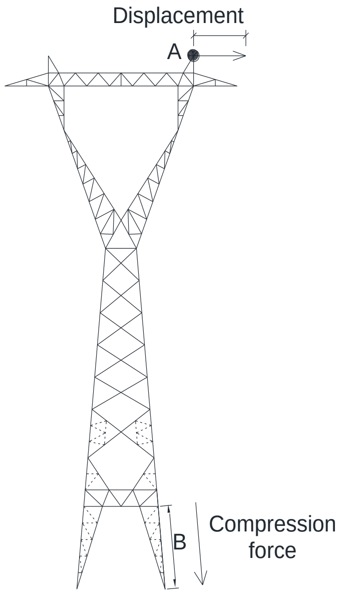


Figure 5: Displacement and force.

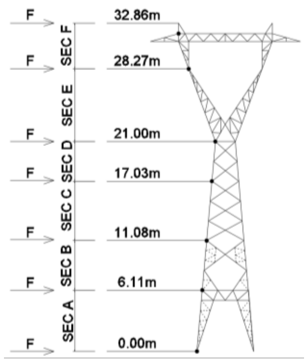
The wind loads applied on the main tower (Model I), and the transmission line system (Model II), were determined based on the use of the Brazilian standard NBR 6123 entitled “Forces due to wind on buildings” [7] (see Figure 6).

The nondeterministic dynamic wind loads applied on the Model III (see Figure 6) were modelled considering 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].

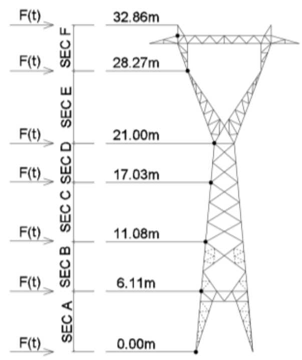
In this work, seven wind basic velocities ($\upsilon = 50$ m/s, $\upsilon = 45$ m/s, $\upsilon = 40$ m/s, $\upsilon = 35$ m/s, $\upsilon = 30$ m/s, $\upsilon = 25$ m/s and $\upsilon = 20$ m/s) were considered based on significant wind velocities applied to Brazilian transmission lines regions, with mean of 3 seconds, height at 10 meters from the ground, and return period of 50 years [7].

The wind series were generated as lagged random series from a time interval τ, calculated from the use of the auto covariance and covariance functions [1,4,6]. The structural damping applied to Model III was considering through to the Rayleigh proportional damping formulation, according to this formulation, the structural system damping matrix [C] is proportional to the mass and stiffness matrix [1].

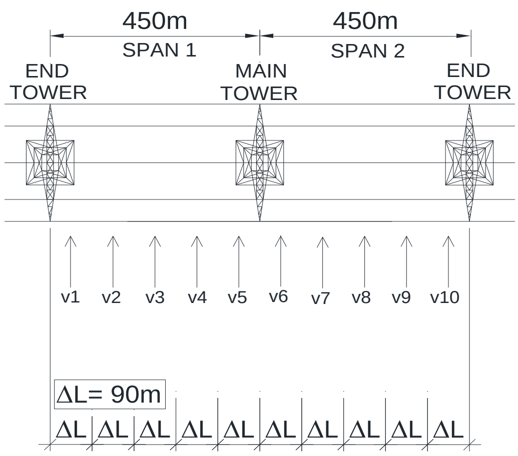
In sequence, Figure 7 presents a typical example of the tower displacement in time domain, when subjected to non-deterministic dynamic wind loads. Figure 8 illustrates this displacement in frequency domain determined through Fast Fourier Transform (FFT), where it is possible to see the displacement amplitude associated to the fundamental frequency of the transmission line system [$f_{01} = 0.153$ Hz: 1st vibration mode (Model III)]. It must be emphasized that the developed analysis methodology used to calculate the structural response is illustrated in Figure 9.



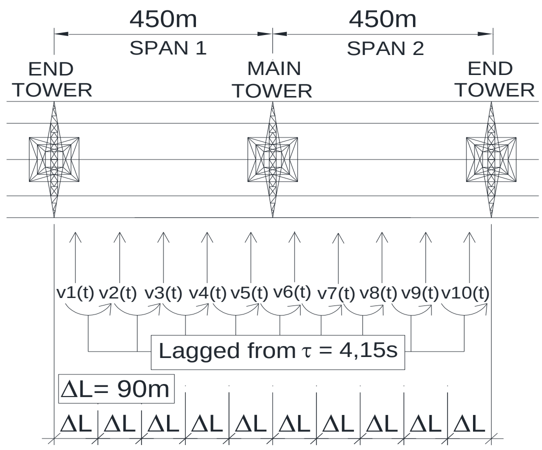
a) Static equivalent wind loads applied on the main tower: Models I and II.



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.

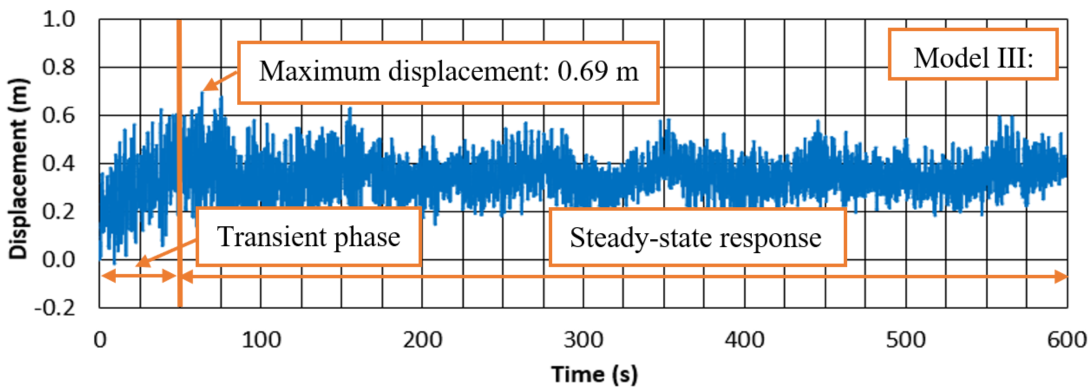


Figure 7: Typical horizontal translational displacement. Structural section A (see Figure 5): time domain.

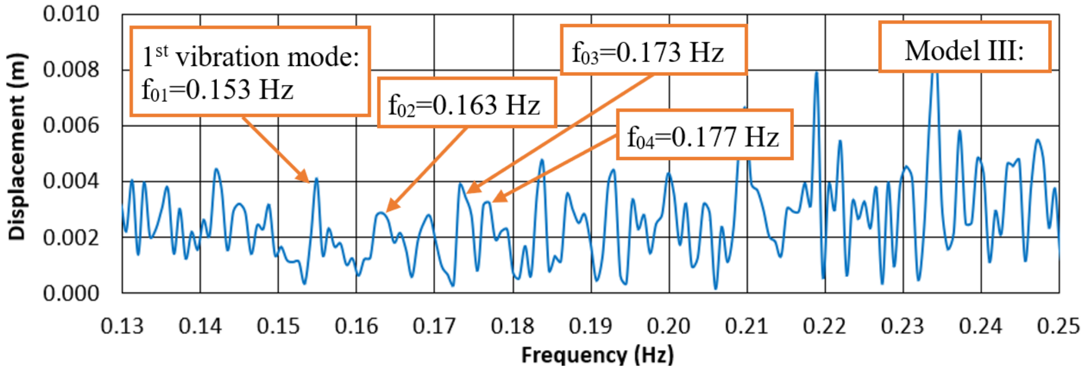


Figure 8: Typical horizontal translational displacement. Structural section A (see Figure 5): frequency domain.

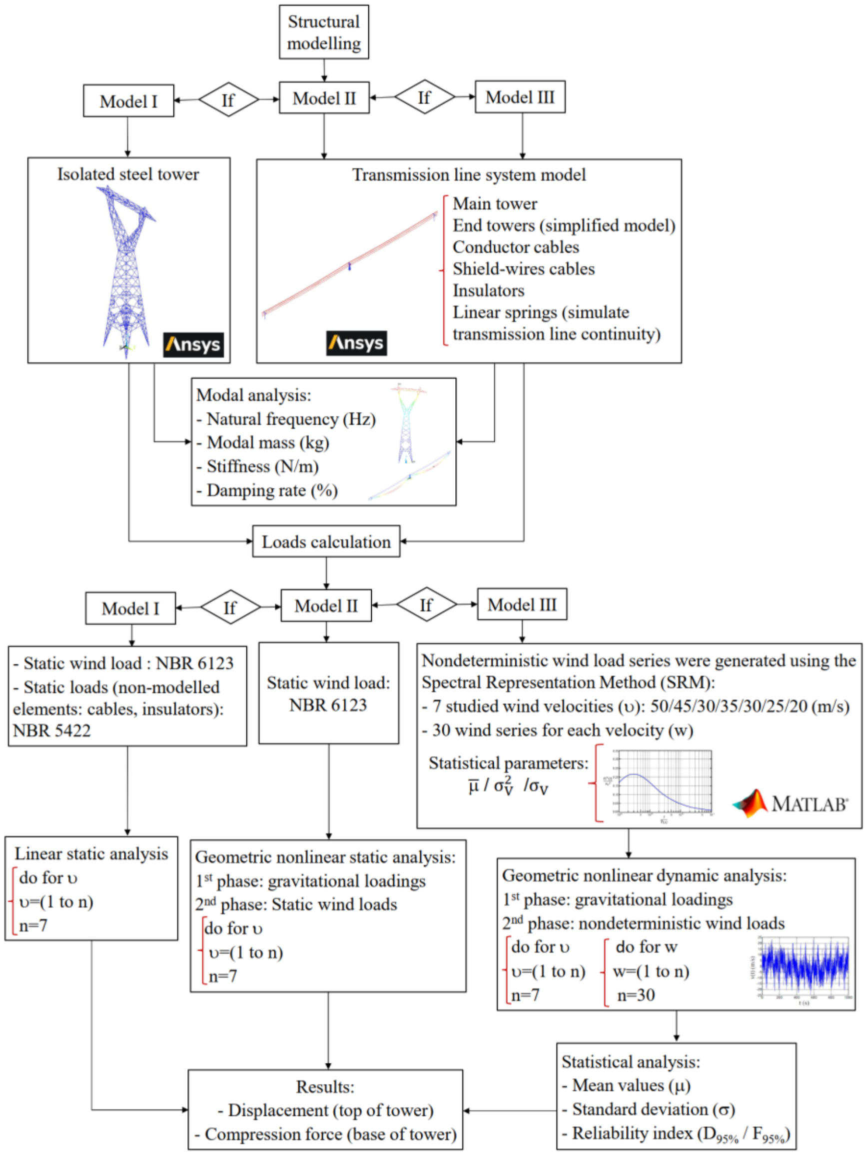


Figure 9: Proposed analysis methodology to generate the project response.

The horizontal translational displacement at the main tower structural section A and compression force acting on member B (see Figure 5) determined based on the use of the Mode I (linear static analysis) and Model II (nonlinear static analysis) are presented in Table 2. Tables 3 and 4 present the statistical analysis associated to the structural system dynamic response [mean value (μ); standard deviation (σ); reliability index ($D_{95\%}$ and $F_{95\%}$)], related to the horizontal displacement (section A: see Figure 5) and compression forces (element B: see Figure 5), calculated considering 30 series of nondeterministic wind loads based on the use of Model III.

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

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 Tables 2 to 5). This way, the dynamic amplification factor (DAF) related to displacements and compression forces of the investigated models are approximately three (DAF = 2.5). The differences between the results calculated based on the use of Models I and II are not significant (see Table 2).

On the other hand, it can be seen from Table 5 results, that the differences between the model’s response (Model I; Model II; Model III), in terms of members force ratio could be relevant and up to 106%. It must be emphasized that according to the Brazilian standard NBR 8850 [10], the admitted maximum force ratio is equal to 93%, and clearly the results provided by Model III have surpassed this limit for higher wind velocities (see Table 5).

The results obtained in this investigation indicated 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 174% [$F_{95\%}$ = 422 kN > 242 kN] (see Table 5). This member force ratio value is enough to surpass the structural member capacity determined by NBR 8850 [10] and could cause structural failure.

Model I							
Velocity υ (m/s)	50m/s	45m/s	40m/s	35m/s	30m/s	25m/s	20m/s
Displacement (m)	0.29	0.23	0.18	0.14	0.10	0.07	0.05
Force (kN)	165	136	110	87	67	50	36
Model II							
Velocity υ (m/s)	50m/s	45m/s	40m/s	35m/s	30m/s	25m/s	20m/s
Displacement (m)	0.29	0.23	0.19	0.15	0.12	0.09	0.07
Force (kN)	167	140	117	96	79	64	52

Table 2: Displacement (A) and compression force (B) (see Figure 5): Model I and II.

Horizontal translational displacements in (m) at point A (see Figure 5): Model III							
Wind Serie	υ = 50m/s	υ = 45m/s	υ = 40m/s	υ = 35 m/s	υ = 30m/s	υ = 25m/s	υ = 20 m/s
1	0.693	0.579	0.469	0.360	0.254	0.164	0.104
2	0.700	0.552	0.461	0.327	0.242	0.169	0.100
3	0.727	0.547	0.402	0.351	0.256	0.164	0.105
4	0.635	0.626	0.454	0.354	0.253	0.193	0.100
5	0.772	0.569	0.427	0.331	0.237	0.153	0.088
6	0.770	0.516	0.501	0.336	0.252	0.162	0.108
7	0.704	0.587	0.491	0.359	0.240	0.172	0.099
8	0.590	0.570	0.523	0.355	0.263	0.155	0.105
9	0.647	0.597	0.411	0.351	0.289	0.171	0.096
10	0.664	0.619	0.462	0.351	0.248	0.144	0.089
11	0.638	0.558	0.430	0.316	0.230	0.175	0.101
12	0.747	0.567	0.454	0.337	0.218	0.212	0.107
13	0.631	0.576	0.470	0.313	0.279	0.161	0.089
14	0.702	0.529	0.488	0.316	0.248	0.159	0.100
15	0.696	0.622	0.516	0.341	0.197	0.183	0.107
16	0.658	0.535	0.520	0.386	0.243	0.148	0.110
17	0.602	0.526	0.398	0.350	0.273	0.151	0.100
18	0.619	0.549	0.473	0.021	0.218	0.156	0.107
19	0.719	0.608	0.444	0.358	0.220	0.156	0.138
20	0.616	0.631	0.448	0.360	0.232	0.169	0.121
21	0.649	0.528	0.497	0.327	0.232	0.143	0.098
22	0.654	0.635	0.457	0.351	0.224	0.179	0.102
23	0.611	0.632	0.541	0.354	0.200	0.161	0.103
24	0.722	0.533	0.434	0.331	0.230	0.160	0.113
25	0.620	0.559	0.476	0.336	0.220	0.165	0.103
26	0.644	0.586	0.483	0.359	0.238	0.174	0.106
27	0.733	0.588	0.488	0.355	0.249	0.184	0.110
28	0.714	0.596	0.540	0.351	0.222	0.156	0.099
29	0.708	0.635	0.444	0.351	0.229	0.157	0.121
30	0.635	0.538	0.453	0.316	0.216	0.171	0.105
μ	0.674	0.576	0.469	0.337	0.238	0.166	0.105
σ	0.050	0.037	0.037	0.313	0.021	0.014	0.010
$D_{95\%}$	0.692	0.590	0.482	0.316	0.246	0.171	0.108

Table 3: Horizontal translational displacements in (m). Structural section A (see Figure 5). Model III.

Compression forces in (kN) acting on member B (see Figure 5): Model III.							
Wind Serie	υ = 50m/s	υ = 45m/s	υ = 40m/s	υ = 35 m/s	υ = 30m/s	υ = 25m/s	υ = 20 m/s
1	416	342	279	231	152	100	65
2	436	322	285	236	146	101	64
3	458	316	241	208	150	100	67
4	393	368	267	190	149	117	64
5	471	341	252	206	145	93	59
6	471	316	304	214	150	98	70
7	423	355	297	208	145	104	64
8	365	341	318	210	152	93	66
9	391	368	244	210	174	105	61
10	401	382	273	222	146	88	58
11	375	336	257	223	136	107	65
12	447	346	276	217	133	127	68
13	369	351	280	205	167	96	59
14	430	310	295	199	145	97	65
15	421	396	302	214	118	111	68
16	394	327	318	188	145	92	69
17	368	320	239	210	161	91	65
18	373	338	273	213	130	96	67
19	440	371	266	196	133	96	85
20	377	394	266	202	139	102	76
21	391	318	306	216	139	87	62
22	392	369	276	210	132	108	64
23	372	376	335	211	119	97	66
24	433	328	267	205	135	100	70
25	377	339	293	187	133	102	66
26	383	354	282	200	139	106	68
27	462	364	284	184	151	113	69
28	446	368	325	185	136	96	64
29	442	386	261	204	140	97	77
30	380	320	266	226	130	105	66
μ	410	349	281	208	142	101	66
σ	34	25	24	13	12	8	5
$F_{95\%}$	422	358	290	212	147	104	68

Table 4: Compression forces in (kN). Structural element B (see Figure 5). Model III.

Model	Member force ratio (%)						
I	50m/s	45m/s	40m/s	35m/s	30m/s	25m/s	20m/s
II	68	56	45	36	28	21	15
III	69	58	48	40	33	26	21

Table 5: Assessment of the load capacity of the structural element B (see Figure 5).

5 Conclusions

The conclusions of this research work are presented considering 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 III (geometric nonlinear dynamic analysis), it is possible to verify differences: up to 257% (displacements), 263% (member's compression forces), and 106% (member force ratio).

2. 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 for higher wind velocities, 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.

3. This investigation has revealed that the geometric nonlinear dynamic analysis is very important to understand the structural behaviour, loads distribution, structural stability and design of transmission lines. This work considered a case study, based on four seven velocities (50m/s, 45m/s, 40m/s, 35m/s, 30m/s, 25m/s and 20m/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.

Acknowledgements

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