Assessment of the structural response 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. However, the main loading to consider in structural analysis of these steel towers is produced by wind loadings. Considering that many accidents associated to this kind of structural system 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 the towers, when subjected to wind nondeterministic loadings, having in mind the assessment of the displacements and forces maximum values that occur in the steel towers. In this work, the investigated transmission line system, including the steel towers, conductors and shield wire types, presents two spans of 450 m associated to a main suspension tower in the centre with total height of 32.86 and other two towers at the ends. The conclusions of this research work pointed to relevant quantitative differences associated to the structural response, when was calculated based on a static linear analysis and compared to the results determined based on a geometric nonlinear and nondeterministic dynamic analysis.

Keywords: transmission lines steel towers; wind nondeterministic loadings; dynamic analysis.

1 Introduction

The lattice steel towers present relevant importance as supports for overhead power transmission lines. The stability of the structural system is crucial to electrical safety of transmission systems [1]. In current day-to-day practice, the project of lattice steel towers considers the first-order elastic structural analysis, assuming static equivalent loads related to the wind action [2]. It is 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, the steel towers will be subjected to additional displacements and forces [3].

Additionally, the main loading to be considered in the structural analysis of transmission lines is produced by the wind loadings, which acts dynamically over the structural system composed by towers and cables [1]. 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 [3].

Therefore, the dynamic characteristic of the wind action is essential for a more realistic analysis based on the use of the Spectral Representation Method (SRM). 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 [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 [1]. 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 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. The studied section of the transmission line system presents two spans of 450 m each one (see Fig. 1), comprehended a main suspension tower in the centre with total height of 32.86m 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.

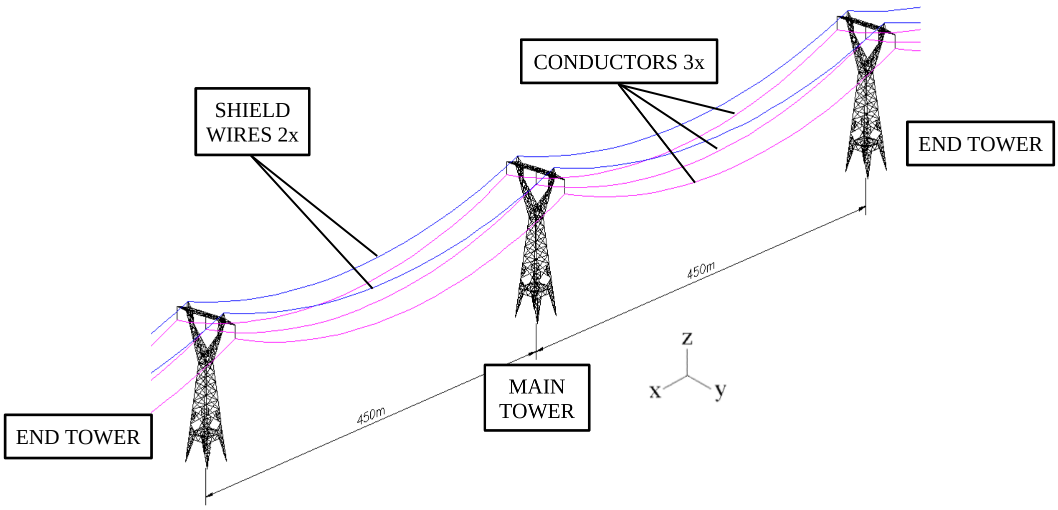


Figure 1. Investigated structural system

3 Finite element modelling of the transmission line system

The investigated transmission line system was modelled based on the use of the Finite Element Method (FEM), utilising the ANSYS software [6]. The beam finite element BEAM188 was used for modelling the main and the end steel towers, the truss finite element LINK180 was used to represent the insulators, the beam finite element BEAM189 was used for simulate the conductors and shield wires and the linear spring finite element COMBIN14 was used to represent the transmission line continuity. In this investigation, the cables were represented based on the use of the element BEAM189, having in mind the complexity of the finite element numerical modelling due to the cable's low stiffness against bending and compression forces.

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 restraints to the horizontal translational displacements related to the three global axes. The developed finite element model is illustrated in Fig. 2.

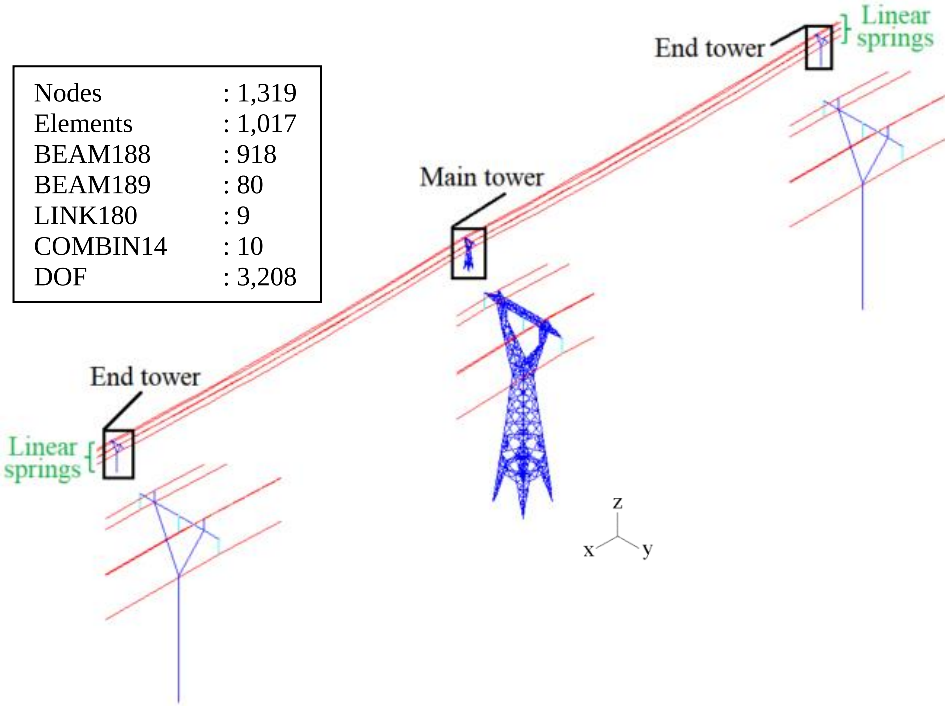


Figure 2. Finite element model of the investigated structural system

4 Structural analysis and results discussion

Initially, the free vibration analysis of the isolated steel tower resulted in a fundamental frequency equal to 2.60Hz. However, when the full transmission line system was considered, the fundamental frequency was equal to 0.153Hz. Considering that the conductors, shield wires and insulators present elevated weight compared with their low stiffness, those have influenced significantly the first vibration modes of the transmission line system.

After that, the linear elastic analysis was performed to Model I and nonlinear geometric analysis to Models II and III. 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 studied are 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. 3) and calculated based on the Brazilian standard NBR 5422 "Design of overhead power transmission lines" [7]. The displacement at point A and forces in element B was determined (see Fig. 4).

The wind loads applied on the main tower (Model I), and the transmission line system (Model II), were determined based on the Brazilian standard NBR 6123 "Forces due to wind on buildings" [5] (see Fig. 5). The nondeterministic dynamic wind loads applied on the Model III (see Fig. 5) 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].

In this work, several basic wind velocities (υ = 50 m/s, υ = 45 m/s, υ = 40 m/s, υ = 35 m/s, υ = 30 m/s, υ = 25 m/s and υ = 20 m/s) were considered, with mean of 3 seconds, height at 10 meters from the ground, and return period of 50 years [5]. The wind load series were generated as lagged random series from a time interval τ, calculated from the use of the autocovariance and covariance functions [1] (see Fig. 5).

The horizontal translational displacement at the main tower (A) and compression force acting on member B (see Fig. 4) determined based on the Mode I and Model II are presented in Tab.2. Tables 3 and 4 present the statistical analysis associated to the structural dynamic response [mean value (μ); standard deviation (σ); reliability index (D95% and F95%)], calculated considering 30 series for each studied wind velocity based on Model III. The developed analysis methodology used to calculate the structural response is illustrated in Fig. 6.

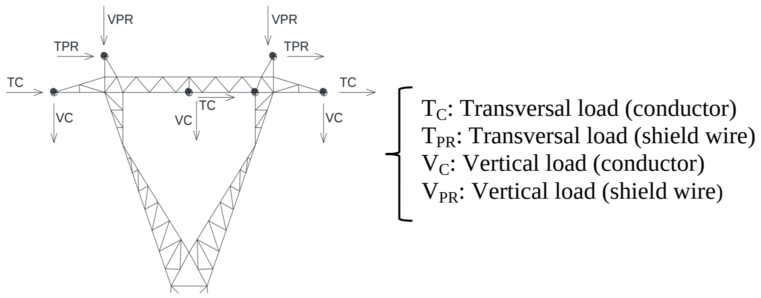


Figure 3: Model I (conductors and shield wires)

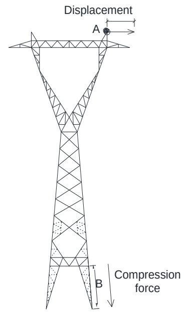
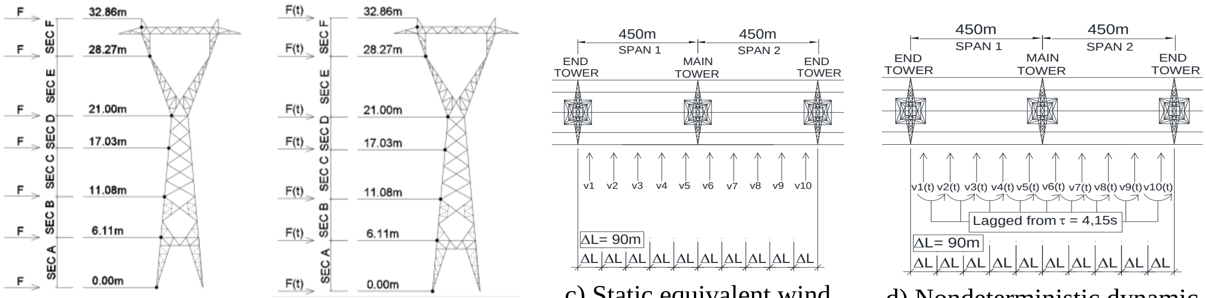


Figure 4: Displacement and force



a) Static equivalent: Models I and II

b) Nondeterministic dynamic: Model III

c) Static equivalent wind loads: Model II

d) Nondeterministic dynamic wind loads: Model III

Figure 5: Definition of the applied wind loads: static equivalent and nondeterministic wind loads

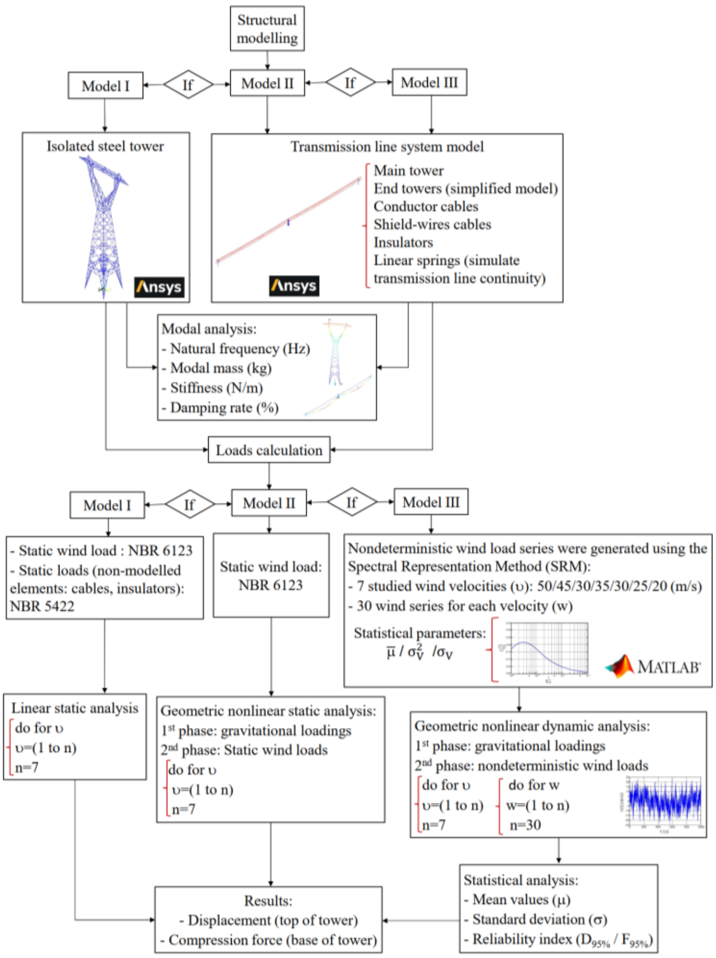


Figure 6: Proposed analysis methodology to generate the wind load series

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

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

Series	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

It should be noted that the static structural analysis (Model I and Model II) provided lower values when compared to those determined based on the dynamic structural analysis (Model III) (see Tab. 2 to Tab. 4). This way, the dynamic amplification factor (DAF) related to displacements and compression forces of the investigated models are approximately 2.5. The differences between the results of Models I and II are not significant. On the other hand, it can be seen from Tab. 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 [8], 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 Tab. 5).

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	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 5. Assessment of the load capacity of the investigated structural element B (see Fig. 4)

Models	Member force ratio (%)						
	υ = 50m/s	υ = 45m/s	υ = 40m/s	υ = 35m/s	υ = 30m/s	υ = 25m/s	υ = 20m/s
I [1]	68	56	45	36	28	21	15
II [1]	69	58	48	40	33	26	21
III [1 2]	174	148	120	88	61	43	28

[1] The element B structural capacity of 242 kN was calculated according to the Brazilian standard NBR 8850 [8].
[2] Member capacity ratio are based on the results related to the reliability index (F95%) (see Tab. 4).

The results obtained in this investigation indicated relevant differences between the displacement and force values according to the chosen finite element model and calculated according to current design practice methodology. It is important to emphasize that the structural member's capacity analysis shows that the maximum member force ratio is equal to 174% [F95% = 422 kN > 242 kN]. 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 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 900 m, 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.

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