Structural analysis of steel towers used in power transmission lines subjected to wind induced dynamic actions

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Abstract. The lattice steel towers have been widely used as supports for power transmission lines becoming essential elements since their stability contributes to a better functioning and electrical safety of the transmission systems. Although the main loading applied to this type of structure be produced by the wind acting dynamically on the elements of the transmissions system (conductors, shield wires, insulators and steel towers), in the current design practice the dynamic structural behaviour of the structural system (towers-conductors) is not considered. Therefore, the main objective of this investigation is to develop an analysis regarding the behaviour of power transmission lines when subjected to wind dynamic loadings, having in mind the assessment of the forces and displacements of the steel towers of the system. In this research work, a power transmission line, composed by the main tower, two adjacent towers, conductors, shield wires and insulators was studied based on a finite element modelling, considering the wind non-deterministic dynamic characteristic, where the wind loads were modelled by an aleatory process based on their statistical properties. The results obtained along the analysis have shown relevant quantitative differences associated to the forces and displacements values when the structural response of the investigated power transmission line was calculated based on a static analysis and a dynamic non-deterministic analysis.

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

1 Introduction

Lattice steel towers have been extensively used as supports for overhead power transmission lines. These towers have become essential elements to the transmission systems and their stability contributes to the perfect functioning and electrical safety [1].

In current design practice, lattice steel towers used in power transmission lines are currently analysed using a first-order elastic structural analysis, assuming that, in addition to the own weight, static equivalent loads related to transmission line components (conductor, shield wires and insulators) and the action of wind [2].

It is common knowledge that second-order elastic structural analysis provides additional structural displacements developing and imposing members forces in addition to those obtained in a first-order elastic analysis. Therefore, a second-order elastic analysis may show that towers will be subjected to additional displacements and its members to additional forces [3].

Additionally, the dynamic characteristics of the wind is also important to a more realistic analysis and the spectral representation method can be used. With this purpose, wind series can be generate with the fluctuant part of the wind determined as a sum of a finite number of harmonics with randomly generated phase angles. A power spectrum and a coherence function to calculate the amplitude of each harmonic and maintaining resemblance to the natural wind is used on this methodology [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 a structural analysis regarding the structural behaviour of lattice steel towers, in order to evaluate member forces and displacements, comparing with the expected values indicated at current design practice methodologies. Therefore, a steel tower with height of 32.86 m was analysed, based on the use of three different analysis methodologies, described in the Table 1, in order to determine the displacement and forces values.

Table 1. Analyses performed

Model	Wind load	Analyse
Model I	Static equivalent [1]	Static linear
Model II	Dynamic nondeterministic	Dynamic linear
Model III	Dynamic nondeterministic	Dynamic geometric nonlinear

[1] NBR 6123 "Forças devidas ao vento em edificações" (in Portuguese) [5].

2 Investigated structural model

The analysed structural model and transmission system characteristics, including wind velocities, conductor and shield wire types were extracted from a simple circuit transmission line presented on the study by Oliveira in 2006 [6]. The studied structure has truss structural system and comprise height of 32.86 m, as can be seen in Figure 1, with dimensions in millimetres. The cross sections of the tower have rectangular base, pyramidal body and hollow configuration at the top, where phases and shield wires are fixed. Angle profiles and steel ASTM A36 type was used.

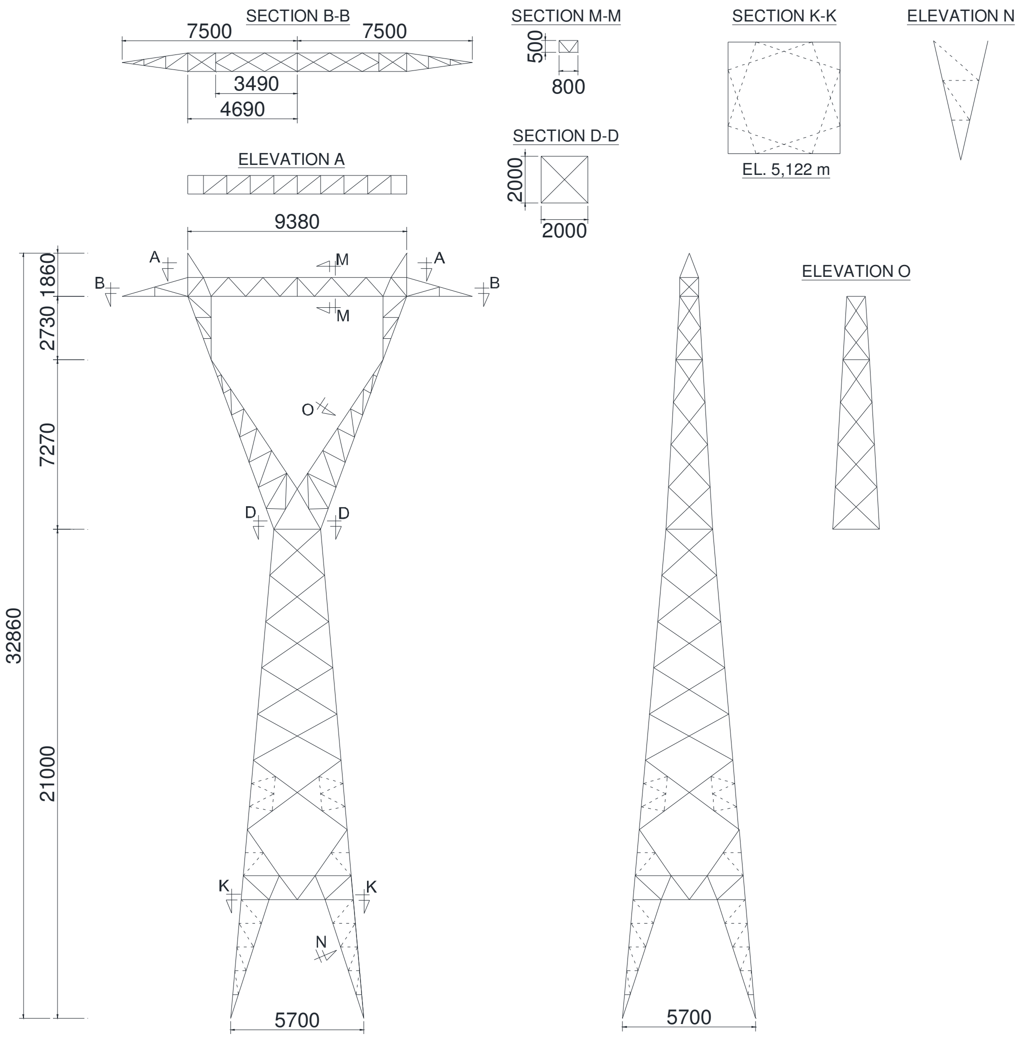
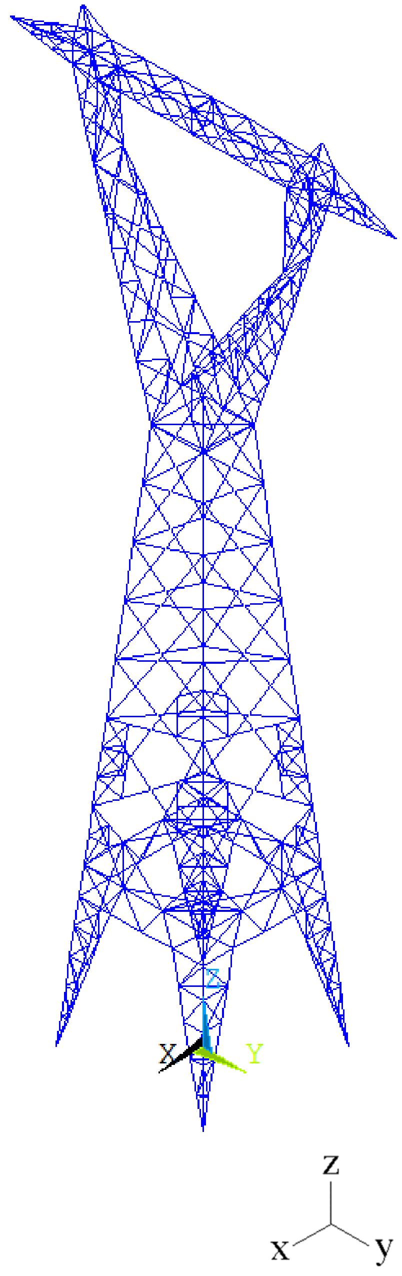
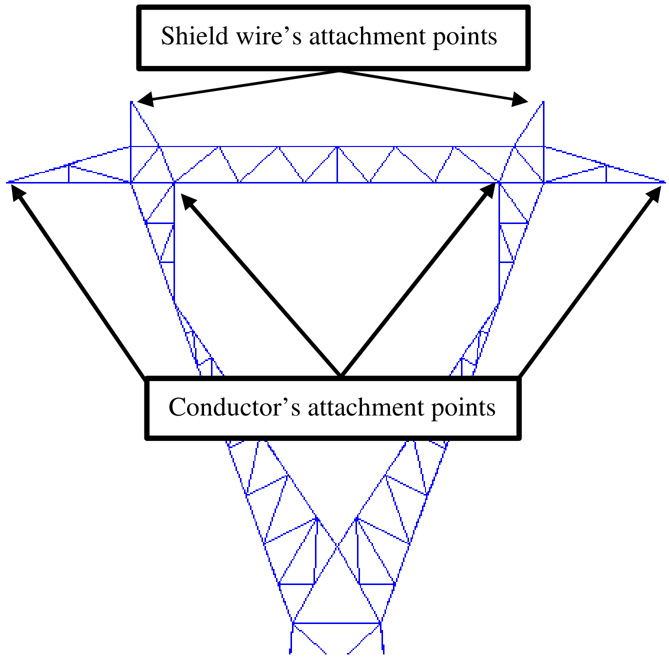


Figure 1. Tower (mm)

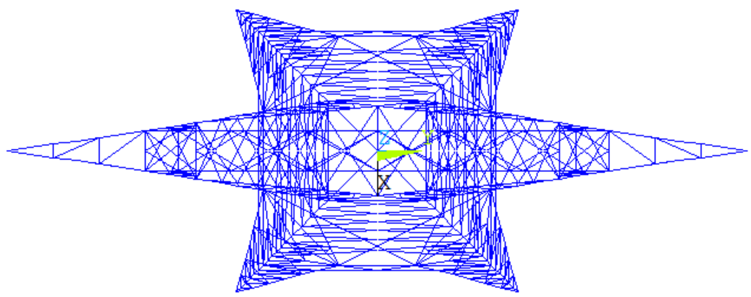
3 Finite element modelling

In the present study, the steel transmission tower was modelled applying the Finite Element Method (FEM), using ANSYS computational program. Beam finite element BEAM 4 was used for modelling the tower. Figure 2 illustrates the tower finite element structural model. Boundary conditions were applied to the 4 nodes that represent the towers foundations, with restrictions to displacements in each of the three axes. The model has 1130 nodes and 900 elements.





b) Attachment points of conductors and shield wires



c) Finite element model: top view

a) Finite element model

Figure 2. Finite element model

4 Structural analysis

The linear analysis (Models I and II) and nonlinear geometric analysis (Model III), adopted Newmark's time integration method for the solution of the equilibrium equations of structural dynamics, and for the nonlinear solutions, the Newton-Raphson method was employed along with Newmark's formulation. This strategy for solving the nonlinear equations are 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 geometric nonlinearity of the steel tower was based on the total Lagrangian formulation, which allows large displacements and rotations. The load hypotheses considered are related to the forces imposed by basic wind velocity acting at 0º with line direction. Loads in cables, shield wires and insulators were defined by the standard NBR 5422 "Projeto de linhas aéreas de transmissão de energia elétrica" (in Portuguese) [7] being applied to their attachment points, as shown in Figure 4, where T_C, T_{PR}, V_C and V_{PR} assume the values of 7.76 kN, 2.40 kN, 13.31 kN and 4.44 kN respectively. The displacement in point A and forces in element B, as show in Figure 3 were determined.

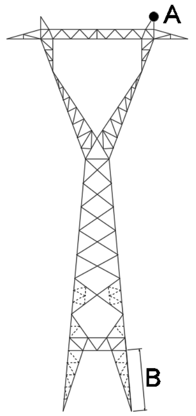


Figure 3. Measurement: displacement and force

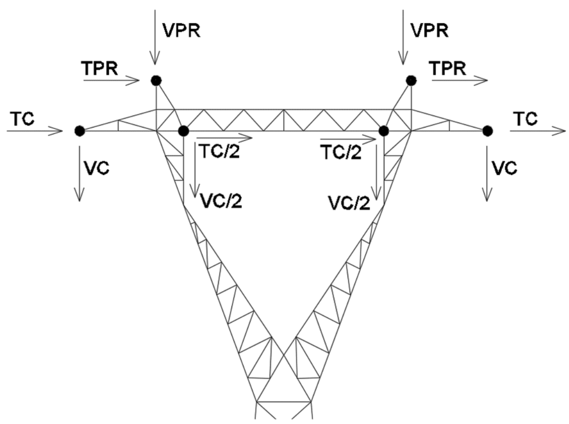
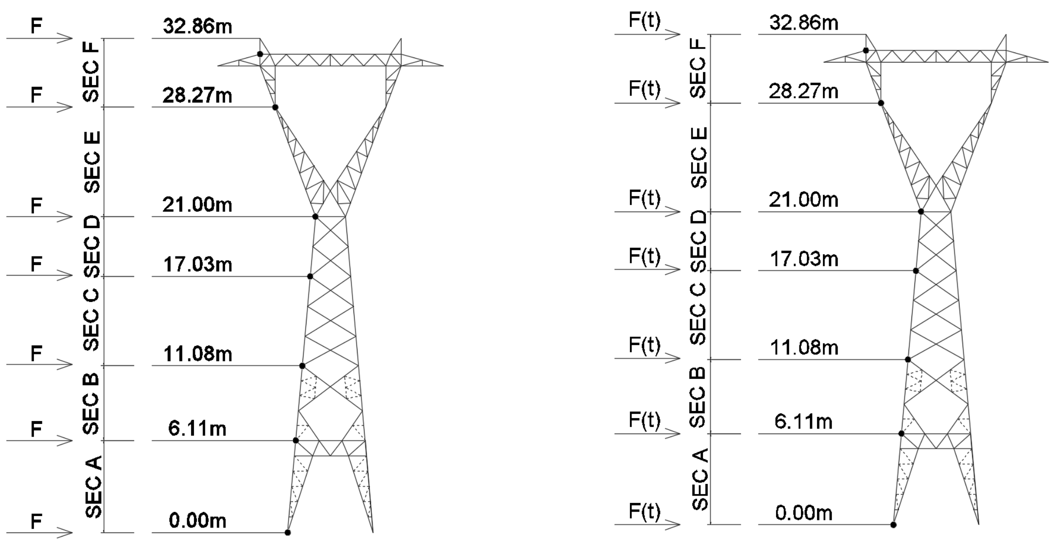


Figure 4. Load applied: conductors and shield wires

Wind loads imposed in the tower, were applied to the top and base of wind panels, as shown in Figure 5, and were defined by the standard NBR 6123 "Forças devidas ao vento em edificações" (in Portuguese) [5] (Model I) and by the nondeterministic wind model (Models II and III). The nondeterministic wind load was modelled by an aleatory process based on their statistical properties with the methodology presented by Barile, Bastos and Silva in 2020 [4] to generate 10 nondeterministic wind series that were applied to Models II and III.



a) Static equivalent wind load (Model I)

b) Dynamic wind load (Models II and III)

Figure 5. Wind load on tower

An example of tower displacements over time, when subjected to non-deterministic wind loads, can be observed in Figure 6. Figure 7 illustrates the displacement in the frequency domain obtained thought Fast Fourier Transform (FFT) and it is possible to see the maximum amplitude of displacement that occurs in the first natural frequency of the tower (f_{01}=2.472 Hz) [6] (Model III).

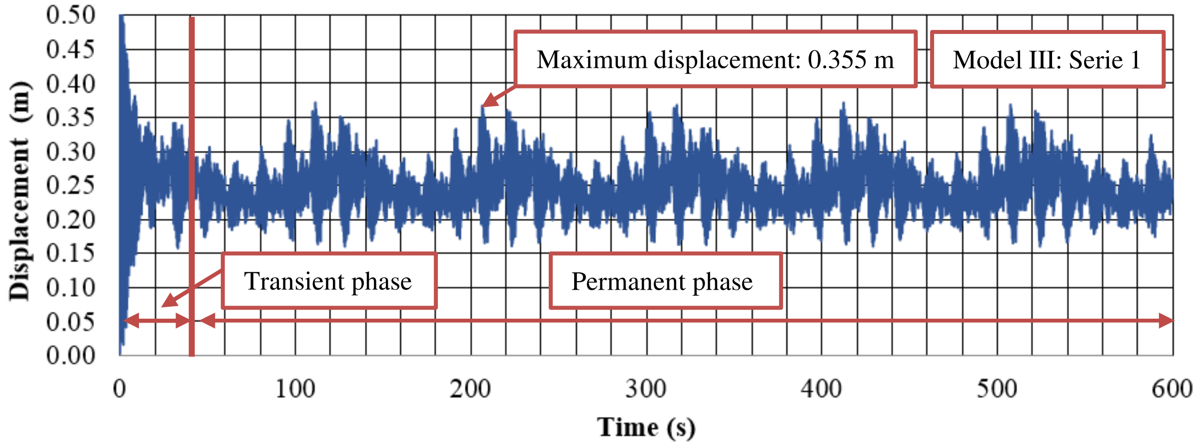


Figure 6. Example of horizontal displacement on point A (see Figure 3)

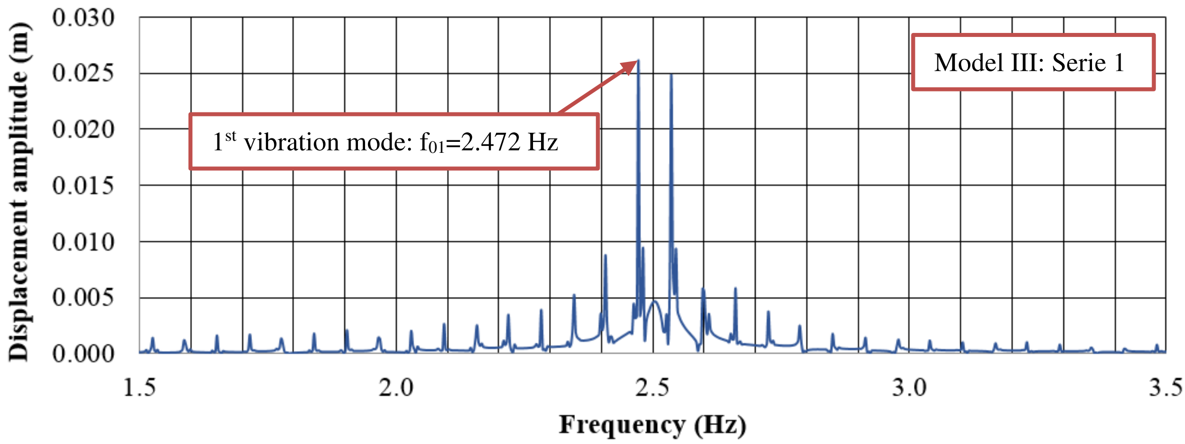


Figure 7. Example of horizontal displacement on point A (see Figure 3) in the frequency domain

The displacement in point A (see Figure 3) and compression force in member B (see Figure 3) obtained with the linear static analysis were 0,245 m and 146 kN respectively. Additionally, Tables 2 and 3 present the results of displacements in point A (see Figure 3) and compression forces in member B (see Figure 3) obtained from series of nondeterministic wind loads applied to Models II and III (transient phase was not considered in this study). Values of mean, root mean square (RMS), peak and mean ten peaks are present to each series and mean, standard deviation and reliability rate of 95% ($U_{95\%}$) to evaluate series outputs.

It is possible to observe that the values of displacements between the studied models are at most 45%. Additionally, the values of compression force the studied models are at most 58%. Considering Table 2, it should be noted that static analysis (Model I) provides lower values of displacements and compression forces compared with the dynamic analysis (Models II and III). The nonlinear geometric analysis (Model III) modifies natural frequencies of the structure, reflecting on the energy transfer and structural response and in this study provides higher values of displacements and compression forces than linear analysis (Model II), although this difference is not significant, with 5% difference at most.

Table 2. Displacement on point A (see Figure 3) (Models II and III)

Series	Mean		RMS		Peak		Mean ten peaks	
	Model II	Model III	Model II	Model III	Model II	Model III	Model II	Model III
1	0.245	0.247	0.248	0.249	0.355	0.355	0.349	0.350
2	0.247	0.249	0.250	0.251	0.331	0.345	0.325	0.338
3	0.246	0.248	0.249	0.250	0.352	0.353	0.339	0.345
4	0.247	0.249	0.249	0.251	0.356	0.367	0.344	0.357
5	0.247	0.248	0.249	0.251	0.347	0.348	0.341	0.344
6	0.247	0.249	0.250	0.251	0.355	0.356	0.349	0.351
7	0.245	0.247	0.248	0.249	0.354	0.354	0.348	0.349
8	0.246	0.248	0.248	0.250	0.347	0.352	0.337	0.345
9	0.247	0.249	0.249	0.251	0.355	0.356	0.347	0.350
10	0.245	0.247	0.248	0.249	0.357	0.358	0.347	0.352
Mean	0.246	0.248	0.249	0.250	0.351	0.354	0.342	0.348
Standard deviation	0.001	0.001	0.001	0.001	0.007	0.006	0.007	0.005
$U_{95\%}$	0.247	0.249	0.249	0.251	0.355	0.358	0.347	0.351

Table 3. Compression force on member B (see Figure 3) (Models II and III)

Series	Mean		RMS		Peak		Mean ten peaks	
	Model II	Model III	Model II	Model III	Model II	Model III	Model II	Model III
1	146	152	148	154	228	236	227	235
2	148	154	149	155	213	221	212	220
3	147	153	149	155	225	232	223	231
4	148	154	149	155	221	238	220	237
5	147	153	149	155	226	231	225	230
6	148	154	149	155	219	230	219	230
7	146	152	148	154	221	229	220	228
8	147	153	148	154	220	226	219	225
9	148	154	149	155	220	230	219	230
10	146	152	148	154	219	229	218	228
Mean	147	153	149	155	221	230	220	229
Standard deviation	1	1	1	1	4	5	4	4
$U_{95\%}$	147	153	149	155	224	233	223	232

Element B structural capacity (see Figure 3) was calculated according to the NBR 8850 "Projeto e execução de torres metálicas treliçadas para linhas de trasnmissão – procedimento" (in Portuguese) [8], using the peak value (Table 3) to verify it´s capacity ratio, as shown in Table 4.

Table 4. Structural design of member B (see Figure 3)

Model	Design force (kN)	Member capacity (kN)	Member force ratio (%)
1	146	242	60
2	224	242	92
3	233	242	96

It can be seen from Table 4, that the difference between models in terms of member force ratio could be up to 36% and according to the referred standard NBR 8850 [8], the maximum force ratio admitted is 93%, therefore to the Model III overcomes the admitted ratio.

The results obtained along the presented paper reflect differences between values of displacements and forces according to the static linear (Model I), dynamic linear (Model II) and dynamic geometric nonlinear (Model III) finite element model analysis. It is important to notice that the structural members capacity shows an increase up to 36% of the member force ratio. Though the 36% increase, related to the force ratio is enough to overpass the structural member capacity specified by NBR 8850 [8], this increase could cause structural failure.

5 Conclusions

The final remarks are presented based on the investigated structural finite element models having in mind the analyses performed: static linear (Model I), dynamic linear (Model II) and dynamic geometric nonlinear (Model III) on lattice steel tower and the associated design verifications of member selected. This way, the following conclusions can be drawn from the results presented in this study:

1. The results obtained have shown relevant quantitative differences between the values of the forces established by the design standards and those calculated through a dynamic geometric nonlinear analysis based on finite element models.

2. Based on the obtained results it is possible to see that static linear (Model I), dynamic linear (Model II) and dynamic geometric nonlinear (Model III), presented up to 45% increase, in terms of displacement, up to 58% of member's compression force and up to 36% in member ratio.

3. It is important to notice that the structural member's capacity analysis shows that the force ratio increase is enough to overpass the structural member capacity as result of the differences between the forces provided by the standard methodology and obtained from the finite element analysis.

4. This investigation indicated that the dynamic geometric nonlinear analysis is essential to understand structural behaviour, distribution of loads, design and overall structural stability. The nonlinear geometric analysis modifies natural frequencies of the structure, reflecting on the energy transfer and structural response. There is an interest in studying more deeply this phenomenon.

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