Structural Analysis of Guyed Steel Telecommunication Towers for Radio Antennas

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The usual structural analysis models for telecommunication and transmission steel tower design tend to assume a simple truss behaviour where all the steel connections are considered hinged. Despite this fact, the most commonly used tower geometries possess structural mechanisms that could compromise the assumed structural behaviour. A possible explanation for the structure stability is related to the connections semi-rigid response instead of the initially assumed pinned behaviour. This paper proposes an alternative structural analysis modelling strategy for guyed steel towers design, considering all the actual structural forces and moments, by using three-dimensional beam and truss finite elements. Comparisons of the above mentioned design models with a third alternative, that models the main structure and the bracing system with 3D beam finite elements, are made for three existing guyed steel telecommunication towers (50m, 70m and 90m high). The comparisons are initially based on the towers static and dynamic structural behaviour later to be followed by a linear buckling analysis to determine the influence of the various modelling strategies on the tower stability behaviour.
Keywords*: telecommunication and transmission towers, guyed steel structures, spatial structures, structural analysis, computational modelling, modal analysis, stability analysis.*

Introduction

The Brazilian telecommunication and electrical power transmission systems expansion were the main reasons for efficient and cost-effective transmission and telecommunication steel towers. Steel truss towers have been used to support transmission antennas or to enable electrical power transmission lines to be built, interconnecting the vast Brazilian territory. However, structural collapses, mainly associated with the wind action, are not uncommon to this particular structural solution (Blessmann, 2001; Carril Júnior 2000).

Despite these facts, most of the traditional structural analysis methods for telecommunication and transmission steel towers still assume a simple truss behaviour, where all connections are considered hinged. On the other hand, structural mechanisms, that could compromise the assumed structural response, can be present in various commonly used tower geometries whenever truss type models are adopted (Policani 2000, 2000a; Silva et al. 2000, 2002).

A usual solution to overcome this problem is the use of dummy structural bars to prevent the occurrence of the unwanted degrees of freedom. These bars, possessing a small axial stiffness, are generally employed to prevent the occurrence of structural mechanisms, enabling the use of standard finite element programs. A possible explanation for the structure stability is related to the semi-rigid, instead of the assumed hinged joint behaviour. In fact, most major steel tower constructors still rely on full-scale tests to determine which design and fabrication details can provide a good test correlation with the assumed simple truss model results.

This paper proposes an alternative structural analysis modelling strategy, based on a less conservative model combining 3D beam finite elements in the main structure and 3D truss elements in the bracing system and eliminating the use of dummy bars present in the traditional analysis.

Further comparisons of the two above mentioned methods and another design alternative only using 3D beam finite elements on three existing guyed steel telecommunication towers (50m, 70m and 90m high) are described. The comparison is focused on the tower structural response in terms of displacements, bending moments, stresses, natural frequencies and buckling loads.

Nomenclature

σ_{max} = Maximum stress of the tower;
u_{max} = Maximum horizontal displacements of the tower
f_{01} = First natural frequency of the tower,
f_{02} = Second natural frequency of the tower,
f_{03} = Third natural frequency of the tower,
f_{04} = Fourth natural frequency of the tower,
f_{05} = Fifth natural frequency of the tower,
λ_{01} = First buckling load of the tower,
λ_{02} = Second buckling load of the tower,
λ_{03} = Third buckling load of the tower,
λ_{04} = Fourth buckling load of the tower,
λ_{05} = Fifth buckling load of the tower,

The Structural Modelling

Several authors have contributed with theoretical and experimental investigations to access the best modelling strategy for steel transmission and telecommunication towers. It is fair to

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mention the investigations made by: Albermani and Kitipornchai 1993 and 2003; Albermani et al 2004; Carril Júnior 2000; El-Ghazaly and Al-Khaiat 1995; Kahla 1994 and 2000, Kitipornchai and Albermani 1992; Madugula and Wahba 1998; Menin 2002, Rao and Kalyanaraman 2001; Saxena et al 1989; Wahba et al 1996 and Wahba et al 1998.

Kahla 1994, numerically modelled the dynamical effects present in guyed steel towers including the cable galloping effects. Later the same author, Kahla 2000, dynamically modelled the rupture of a cable present in guyed steel towers. The analysis indicated that the guyed steel towers cable rupture, disregarding the wind actions, was one of the most severe critical load hypotheses for the investigated structures.

Wahba et al 1996, considered the dynamical nature of the load acting in guyed steel towers like wind, earthquakes and cable gallop. The finite element method was used to model the tower bars as 3D truss and 3D beam elements obtaining the structural models dynamical characteristics. In a subsequent phase these results were compared to experiments. This paper also described the results of experiments made to identify the main parameters that influence the guyed steel towers natural frequencies, as well as, their and associated vibration modes.

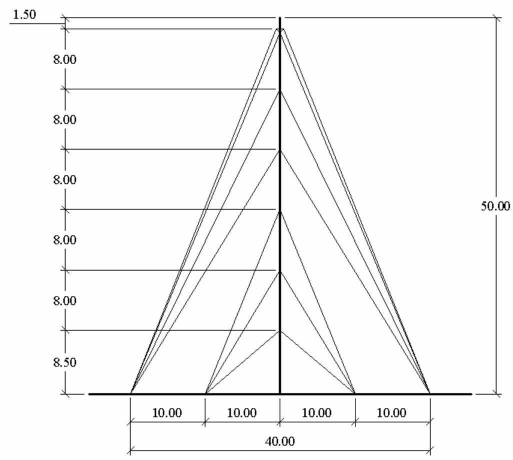
Ghazalyt and Khaiatz 1995, evaluated telecommunication guyed steel tower designs based on discussions of the various non-linear aspects involved on their numerical modelling. This paper also contemplated the development and comparisons of the results of a 3D model for a 600 meter height guyed steel tower.

Wahba et al 1998, performed an investigation of the numerical models used in telecommunication guyed steel towers. The authors stressed the relevance of considering the non-linear effects present even at service load levels. In a subsequent paper, Madugula and Wahba 1998, described two different finite element models for the dynamical simulation of guyed steel towers. This paper also contemplated an experimental modal analysis of reduced-scale guyed steel towers models that produced results in consonance with the developed numerical models.

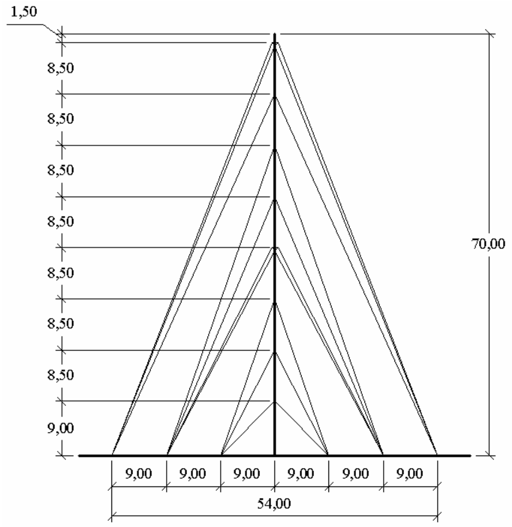
Menin 2002, evaluated telecommunication guyed steel towers from their static and dynamical structural responses. The static analysis compared linear and non-linear mathematical models. The dynamical analysis employed the Monte Carlo simulation method including the wind load floating parcel producing interesting results.

Albermani and Kitipornchai 2003, used the finite element method by means of a geometrical and physical non-linear analysis to simulate the structural response of telecommunication and transmission steel towers. This was followed by a later work of Albermani et al 2004, that investigated the possibility of strengthening steel truss towers from a restructure and rearrangement of their bracing systems. The adopted solution consisted on the addition of axially rigid systems to intermediate transverse planes of the tower panels.

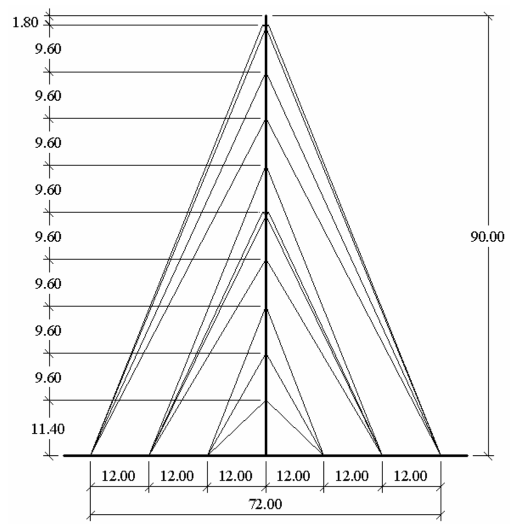
The main purpose of the adopted modelling strategies was to investigate the structural behaviour of the guyed steel towers, preventing the occurrence of spurious structural mechanisms that could lead to uneconomic or unsafe structure. The towers investigated in the present paper (50m, 70m and 90m), have a truss type geometry with a square cross section. Hot rolled angle sections connected by bolts compose the main structure as well as the bracing system. Prestressed cables support the main structure, which must be always in tension. Some of these cables are linked to a specific set of bars arranged to improve the system torsional stiffness. The geometry configuration of the three guyed towers are depicted in Fig. 1.



(a) 50m high



(b) 70m high



(c) 90m high

Figure 1. Towers' basic geometric data.

This investigation considered as acting vertical loads: structure self-weight, stairs, antennas, cables, etc. The steel tower wind effects were the main horizontal loads. These horizontal loads were calculated according to the procedures described on the Brazilian code NBR 6123 (NBR 6123, 1988) and applied to the guyed tower nodes. Two wind load cases related to actions perpendicular and diagonal to the towers face were considered in this analysis. The adopted guy prestress loads were in accordance to the values described in the Canadian Code CSA S37-94 (CSA S37-94, 1994).

A lateral view of the tower and its corresponding idealized structural model are presented in Fig. 2. It can be shown that the idealized structural model cannot only be represented by 3D truss finite elements.

The steel tower modelling presented problems related to the loss of continuity, in some parts of the structure, due to the presence of hinges associated with the 3D truss finite elements. The vertical angles (main legs) are usually connected by a three-bolt connection leading to a rigid design assumption. Using truss elements to model the angles a moment discontinuity occurs, since the trusses connections are considered flexible. On the other hand, using only one bolt in the diagonal or horizontal versus leg angle connections, give rises to another discontinuity. Although the in-plane behaviour can be considered flexible, the out-of-plane behaviour disregards the torsion and bending continuity present in the structure.

To overcome this problem, dummy bars, without mass and with low axial stiffness, were incorporated to the structure. In this process every new bar represents the suppression of an internal degree of freedom. Although the towers have only four bars in the horizontal plane defined by a typical transversal section (Fig. 2) it is still necessary to add a fifth dummy bar to create two isostatic triangles. If this restraint was not considered a simple structural mechanism collapse would occur. Another reason for using these bars is to improve the structure torsion stiffness.

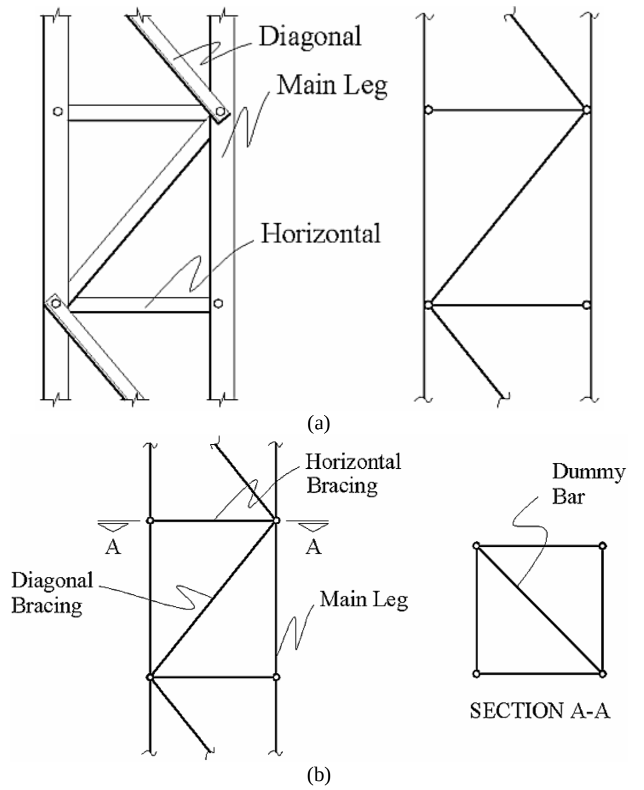


Figure 2. (a) Tower main structure (a) and (b) associated idealized model and dummy bars location.

All the above-mentioned aspects allied to all the difficulties associated with the investigated tower geometry and to the truss finite element characteristics highlight the fact that the traditional truss design is not the best-recommended methodology to be used. It should be stressed that the large number of dummy bars, adopted to enable the structural analysis to be performed, is the major disadvantage of this structural modelling strategy. The used 3D truss finite element is presented in Fig. 3.

This structural modelling strategy is characterised by the use of 3D beam finite elements with rigid connections. The adopted beam finite elements presented six degrees of freedom per node associated with translation and rotation displacements in space, respectively, as shown in Fig. 3.

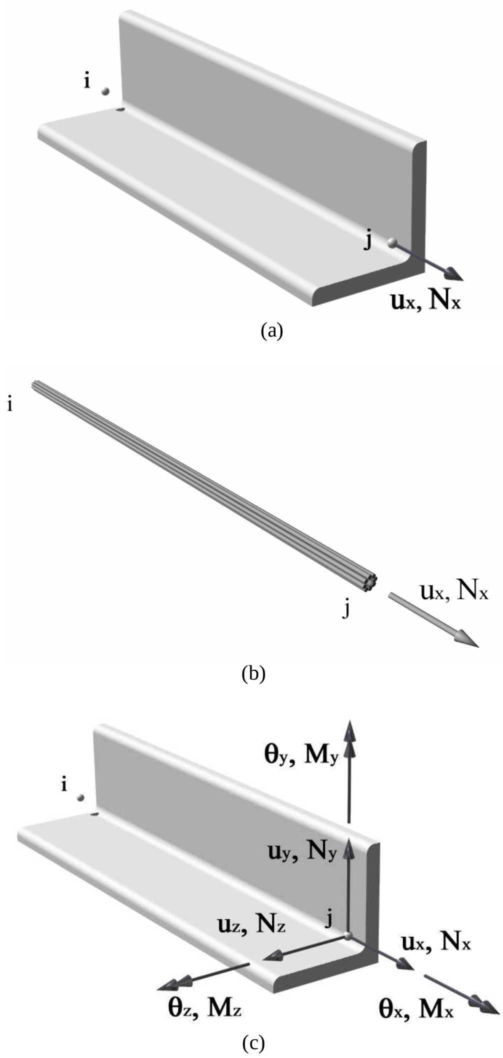


Figure 3. (a)Adopted spatial truss, (b) cable (tension only) and (c) spatial beam finite elements.

When all the structural modelling strategies investigated are compared, the beam element modelling is the easiest to use. This conclusion is mainly justified by the fact that the adoption of dummy bars to prevent possible mechanisms is not required in the beam modelling strategy. Another advantage is the computational model uniformity since all the adopted bars are represented by a single finite element type (3D beam). Despite all the mentioned structural modelling advantages the model final results should be carefully checked. This is due to the fact that in principle, the rigid connections adopted in this strategy can lead to some disturbing and/or spurious effects, especially when the tower critical buckling loads are considered.

Based on an extensive parametric investigation, a modelling strategy combining three-dimensional beam and truss finite elements was proposed. In this methodology the main structure uses 3D beam elements, while the bracing system utilises truss finite elements. This method models the structure as a static determined system discarding the need for dummy bars present in the traditional analysis. The adoption of truss finite elements in the bracing system is explained by two main reasons: a single bolt indicating a hinged behaviour usually makes the bracing system connections to the main structural system. Additionally, the low flexure stiffness values, associated with the bracing elements, imply that no significant moments will be present or transmitted to these structural members.

The use of these two types of 3D finite elements (beam and truss) also eliminates the spurious mechanisms found in the traditional design strategy, disregarding the need for the previously mentioned dummy bars. The authors believe, based on the performed parametric investigations, (Silva et al, 2000, 2002 and 2005), that this mixed modelling strategy can produce more realistic and trustworthy results in respect to the static and dynamic structural analysis, as well as to the tower critical load assessment.

The proposed computational model, developed for the steel tower static and dynamic analysis, adopted the usual mesh refinement techniques present in finite element method simulations implemented in the ANSYS program (ANSYS, 1998). In this computational model, the main structure was represented by three-dimensional beam elements where flexural and torsion effects are considered or truss elements having a uniaxial tension-only (or compression-only) element. The prestressed cables were simulated by spar elements, see Fig. 3.

The ANSYS beam element BEAM44 (ANSYS, 1998), Fig. 3, is a uniaxial element with tension, compression, torsion, and bending capabilities. The element has six degrees of freedom at each node: translations in the nodal x, y, and z directions and rotations about the nodal x, y, and z-axes. The ANSYS spar element LINK10 (ANSYS, 1998), Fig. 3, is a 3-D element having the unique feature of a bilinear stiffness matrix resulting in a uniaxial tension-only (or compression-only) element. With the tension-only option, the stiffness is removed if the element goes into compression (simulating a slack cable or slack chain condition). This feature is useful for static guy-wire applications where the entire guy wire is modelled with a single element.

Static Analysis

Table 1 present linear static analysis results for the investigated guyed towers (50m, 70m and 90m high), according to the three earlier mentioned structural models. Maximum values of stresses and horizontal displacements are presented and compared.

The acting loads considered in the present analysis where self-weight and two wind load cases. In theses cases the horizontal wind loads were applied perpendicular and diagonal to the towers face. The horizontal wind loads were calculated according to the procedures described on the Brazilian code NBR 6123 (NBR 6123, 1988) and applied to the guyed tower nodes.

The prestress cable loads at the lateral anchoring foundation region are normally defined as 10% of the cable nominal strength capacity. It is relevant to observe that prestress cable load values situated between limits of 8% and 15% are allowed by the Canadian Standard (CSA S37-94, 1994). The present analysis adopted values approximately equal to 14%. 13% and 11% for the prestress cable loads of the towers with 50m 70m and 90m height, respectively (Menin, 2002).

The largest differences between the maximum stress values obtained for the simple truss model (Strategy I) and for combined beam and truss element model (Strategy III) are 76.5% (50m high tower), 83.1% (70m high tower) and 79.9% (90m high tower) when compared to the values obtained from the third modelling. When a quantitative analysis of the data was performed it was possible to confirm that the maximum stress values were significantly modified, for the three investigated towers, Table 1. The maximum stress points are depicted in Fig. 4 for the mixed beam and truss element model considering the perpendicular wind load case. The maximum stresses, caused mainly by bending effects, were associated, in all cases studied, to the towers base members.

Table 1. High steel towers maximum stresses and horizontal displacements.

Modelling Strategies: I - Truss Element II - Beam Element III - Combined Beam and Truss Element					
Models	Tower Height (m)	Perpendicular Wind		Diagonal Wind Direction	
		$\sigma_{max.}$ (MPa)	$u_{max.}$ (mm)	$\sigma_{max.}$ (MPa)	$u_{max.}$ (mm)
I	50	83.8	0.049	78.6	0.025
II		344.7	0.049	318.4	0.026
III		357.0	0.049	330.3	0.024
Difference (I and III)		76.5%	None	76.2%	3.8%
Difference (II and III)		3.4%	None	3.6%	7.7%
I	70	74.7	0.089	66.4	0.039
II		411.6	0.093	378.9	0.044
III		425.7	0.093	392.2	0.044
Difference (I and III)		82.5%	None	83.1%	None
Difference (II and III)		3.3%	None	3.4%	None
I	90	83.4	0.090	74.2	0.041
II		388.8	0.099	360.7	0.049
III		398.5	0.099	369.9	0.049
Difference (I and III)		79.1%	9.1%	79.9%	16.3%
Difference (II and III)		2.4%	None	2.5%	None

Table 2. High steel towers natural frequencies.

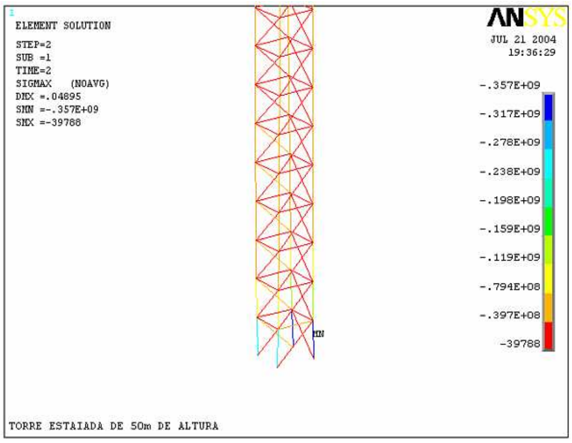
Modelling Strategies:
I - Truss Element
II - Beam Element
III - Combined Beam and Truss Element

Models	Tower Height (m)	Natural Frequencies f_{0i} (Hz)				
		f_{01}	f_{02}	f_{03}	f_{04}	f_{05}
I	50	3.420	3.420	4.203	4.203	5.360
II		4.142	4.142	5.124	5.124	5.504
III		2.609	2.698	2.698	2.731	4.000
I	70	2.616	2.616	3.783	3.783	4.233
II		3.016	3.016	4.225	4.225	4.781
III		3.015	3.015	4.222	4.222	4.779
I	90	2.497	2.497	3.151	3.151	3.420
II		2.903	2.903	3.634	3.634	3.812
III		2.902	2.902	3.633	3.633	3.806

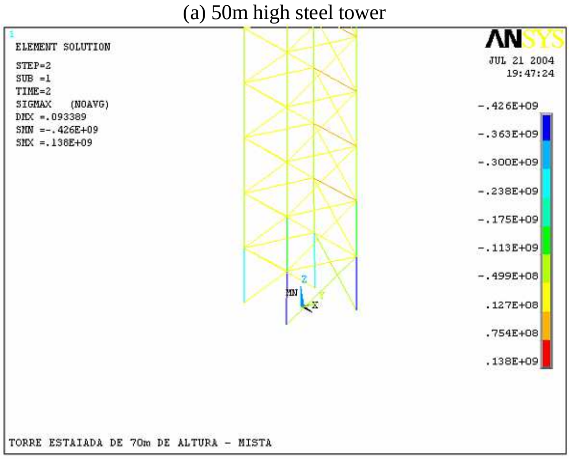
Table 3. High steel towers buckling loads.

Modelling Strategies:
II - Beam Element
III - Combined Beam and Truss Element

Models	Tower Height (m)	Wind Direction					
		Perpendicular			Diagonal		
		Buckling Loads λ_{0i}					
		λ_{01}	λ_{02}	λ_{03}	λ_{01}	λ_{02}	λ_{03}
II	50	10.114	10.306	11.164	10.566	11.063	11.142
III		5.520	5.859	6.118	10.526	10.570	10.630
II	70	14.568	14.810	16.085	15.676	16.079	16.205
III		11.245	11.499	11.648	13.350	13.501	13.522
II	90	11.066	11.121	11.946	11.617	12.354	12.440
III		8.3721	8.4425	8.5999	9.2856	9.3311	9.3617



(a) 50m high steel tower



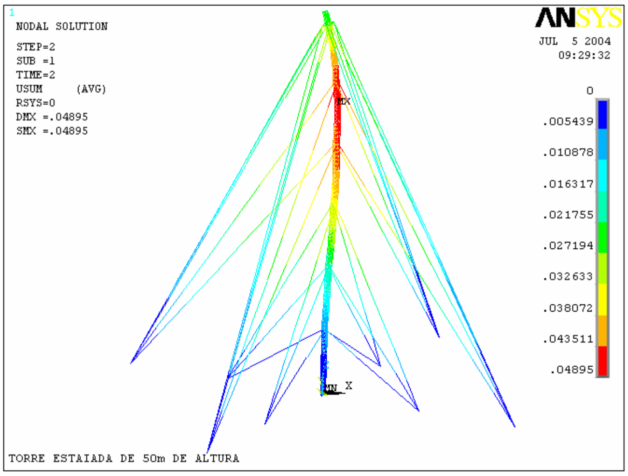
(b) 70m high steel tower

Figure 4. Maximum stress points, perpendicular wind load case.

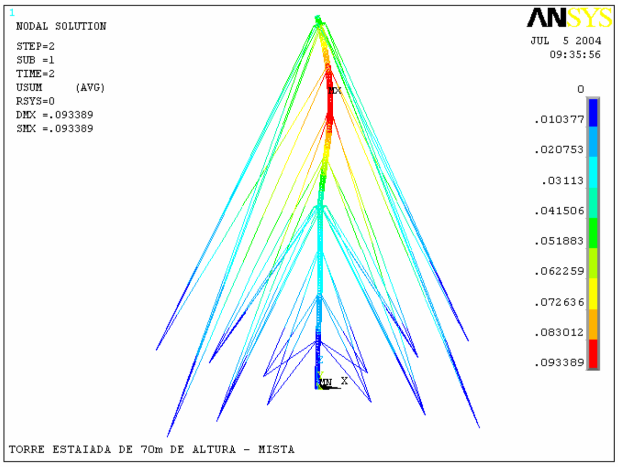
(c) 90m high steel tower

Figure 4. (Continued)..

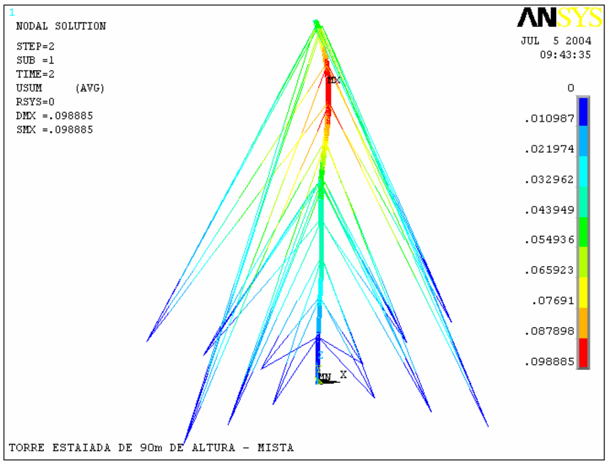
On the other hand, the lateral displacements values were generally not significantly changed when the simple truss model (Strategy I), the beam model (Strategy II) or the combined beam and truss model (Strategy III) were considered, Tables 1 to 3. Figure 5 depicts the deformed shape of the three guyed towers for diagonal wind load case considering the mixed beam and truss element types.



(a) 50m high steel tower



(b) 70m high steel tower

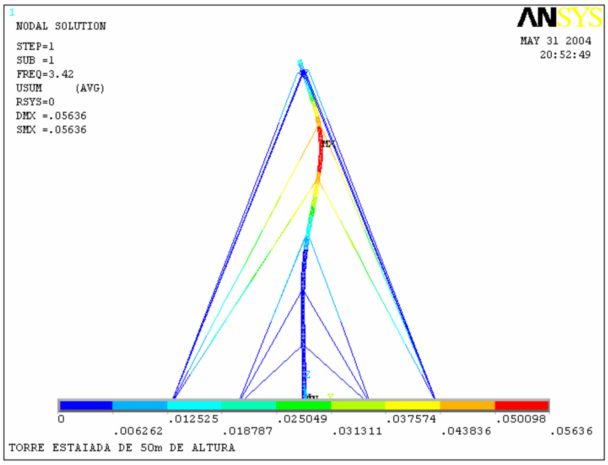


(c) 90m high steel tower

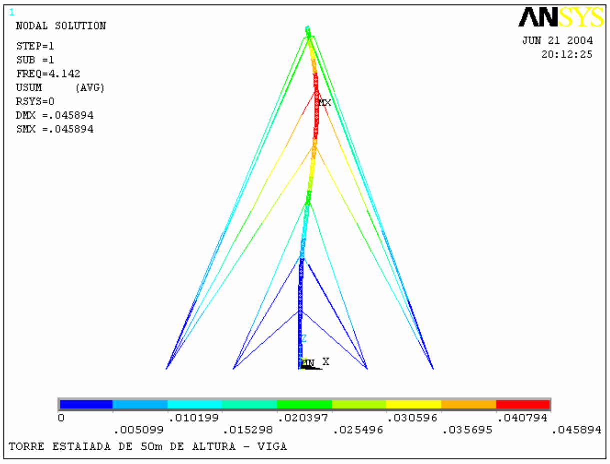
Figure 5. Deformed shapes, perpendicular wind load case.

Dynamic Analysis

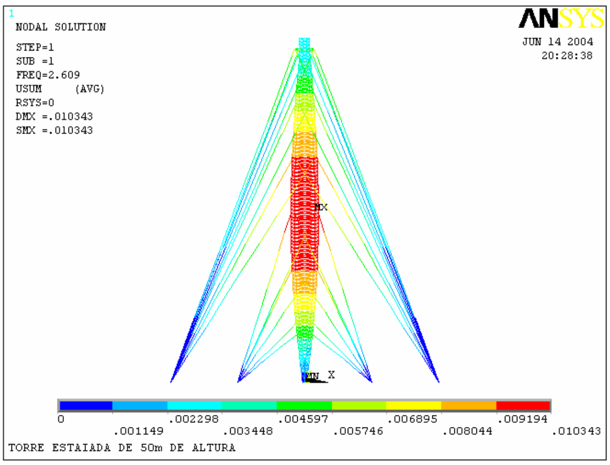
The dynamical analysis performed in the present investigation is based on a free vibration strategy. The study was concentrated in evaluating natural frequencies and corresponding vibration modes for the 50m, 70m and 90m high towers considering all the three finite element modelling strategies previously described. The main objective of this assessment was to investigate if the choice of finite element modelling strategy could significantly affect the tower natural frequencies.



(a) Truss element modelling

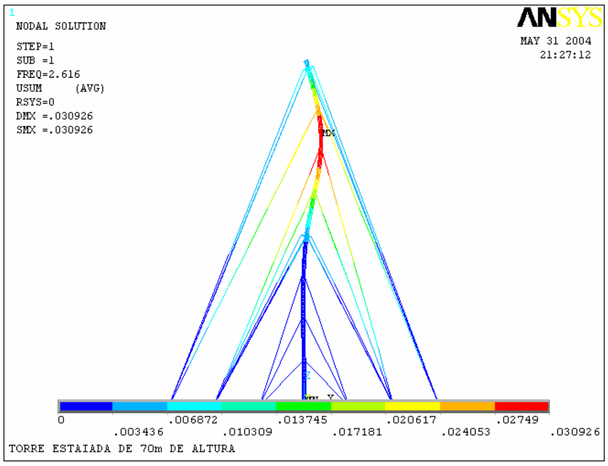


(b) Beam element modelling

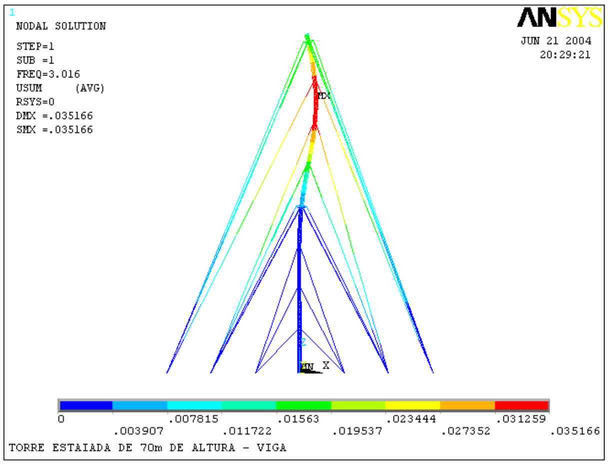


(c) Mixed beam and truss element modelling

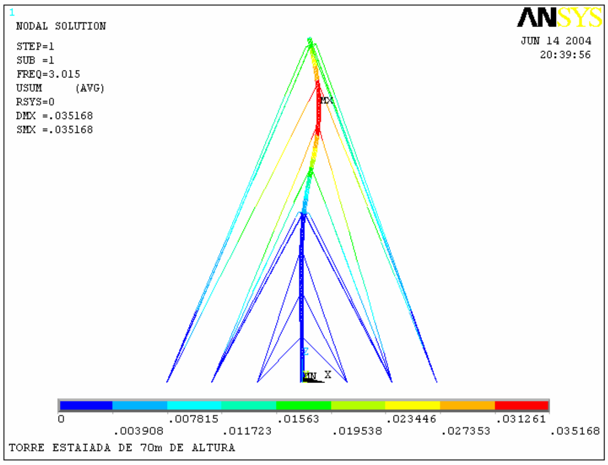
Figure 6. 50m tower models, first mode shape, lateral view.



(a) Truss element modelling

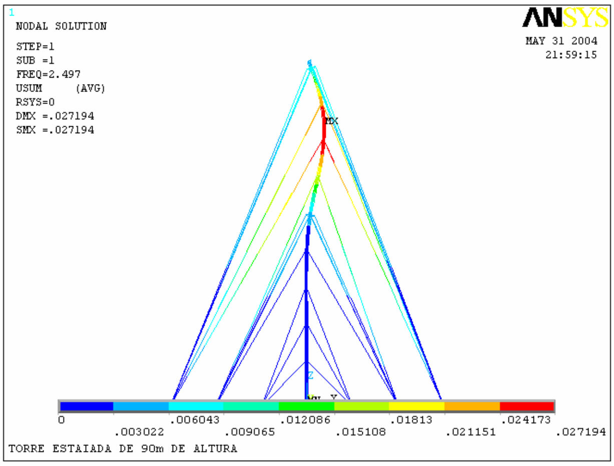


(b) Beam element modelling

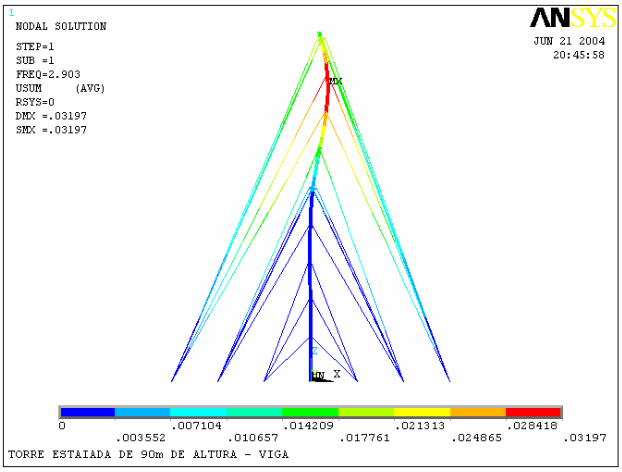


(c) Mixed beam and truss element modelling

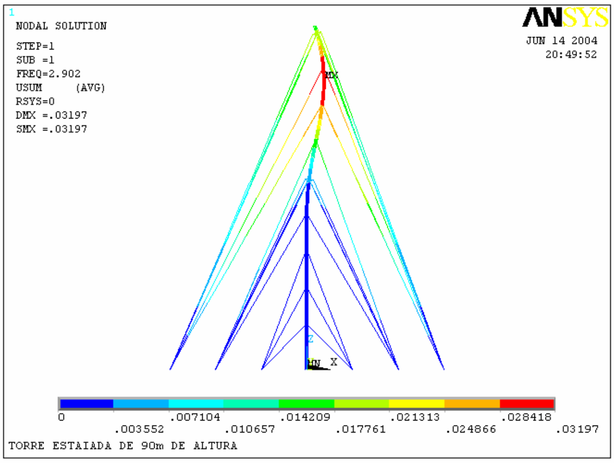
Figure 7. 70m tower models, first mode shape, lateral view.



(a) Truss element modelling



(b) Beam element modelling

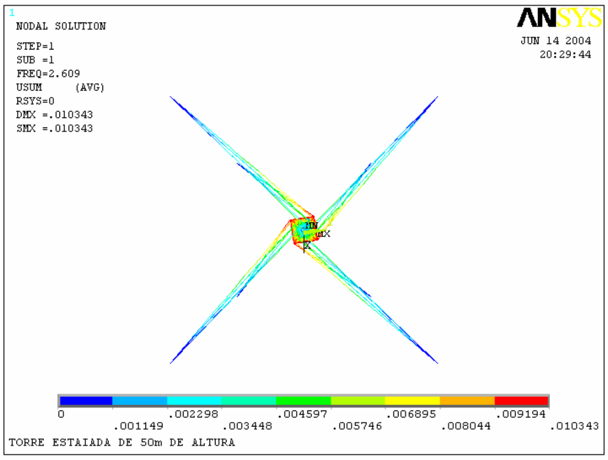


(c) Mixed beam and truss element modelling

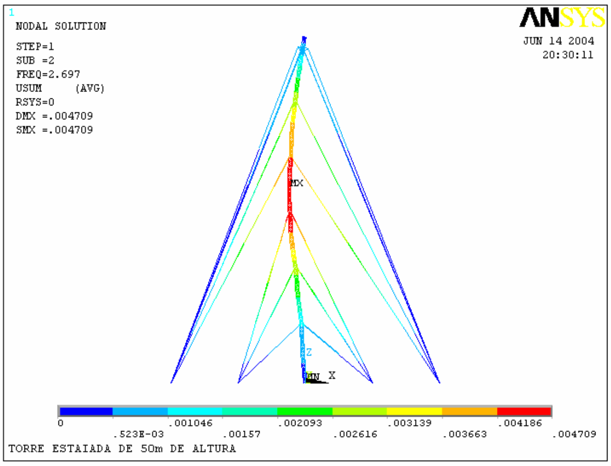
Figure 8. 90m tower models, first mode shape, lateral view.

The tower natural frequencies knowledge is a fundamental issue regarding the tower structural design. This is explained by the fact that if the excitation frequencies are similar to the natural tower frequencies the structure could, in extreme cases, be jeopardised by the resonance or fatigue related phenomena.

The results depicted in Table 2 indicated that the finite element modelling strategy can somehow affect the nominal values of the tower natural frequencies for the initial vibration modes. These results also indicated that the finite element modelling strategy can significantly affect the values of the tower natural frequencies for the 50m tower initial vibration modes. The last two towers (70m and 90m) presented small natural frequencies differences for different modelling strategies. The truss element modelling is associated with the smallest values, indicating a reduced system stiffness. The second and third modelling strategies presented no meaningful differences.



(a) First mode shape, top view



(b) Second mode shape, lateral view

Figure 9. 50m high steel tower layout, mixed beam and truss element modelling.

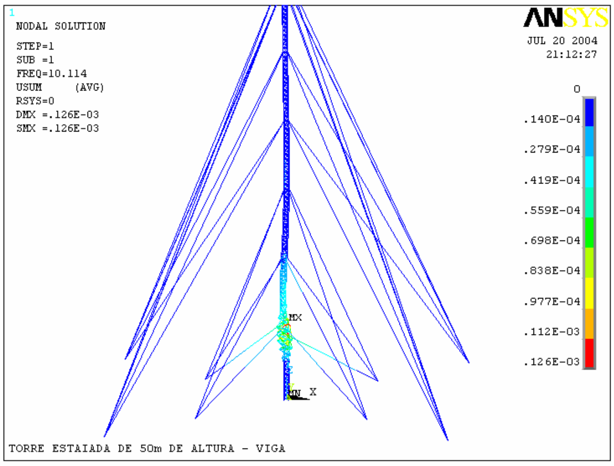
It is also interesting to observe in Table 2 that in the mixed third modelling strategy a slightly lower fundamental frequency occurs when the 50m tower is considered. This is related to the tower base horizontal bracing system finite element modelling. This fundamental frequency is associated with a different vibration mode, see Fig. 6 (Mixed beam and truss element modelling), than the modes present in the other modelling strategies.

Figures 6 to 8 depict the first mode shape associated with all structural models studied in the present paper. All mode shapes, related to the various models, corresponded except in the first structural model (50m high tower). In this case there is no correspondence between the mode shapes extracted by the different modelling strategies. The third modelling strategy presented a spurious mode shape associated with the first natural frequency (Fig. 9). This mode shape describes a torsional mechanism that is not present if the truss or the beam element models were considered. If this first mode shape is neglected, the correspondence between the next two mode shapes associated with the mixed beam and truss element models can be clearly noticed.

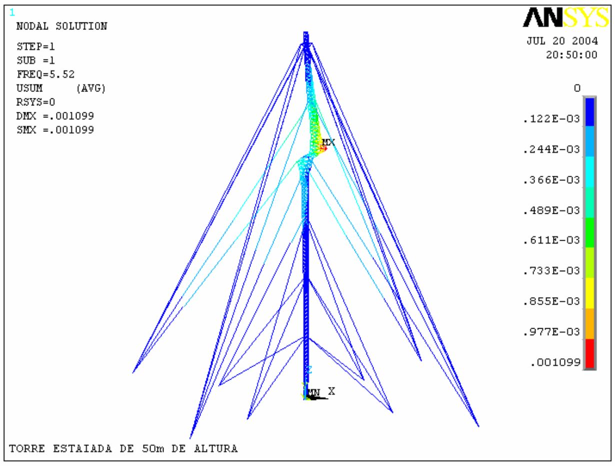
Stability Analysis

Table 3 presents the first three buckling load factor for the investigated tower structures. The stability analysis considered the load actions related to perpendicular and diagonal wind load combinations and the last two already mentioned finite element modelling strategies (beam elements and mixed beam and truss elements). As expected the results clearly indicated the significant influence of the bracing system finite element modelling strategy over the tower critical loads.

Critical loads evaluated according to second methodology (beam elements) are substantially higher than the proposed combined strategy. The lower critical factors are always associated with the perpendicular wind load case, which can be associated with the instability failure control. These buckling loads are not associated with usual design practice and, if adopted, could lead to unsafe structures.



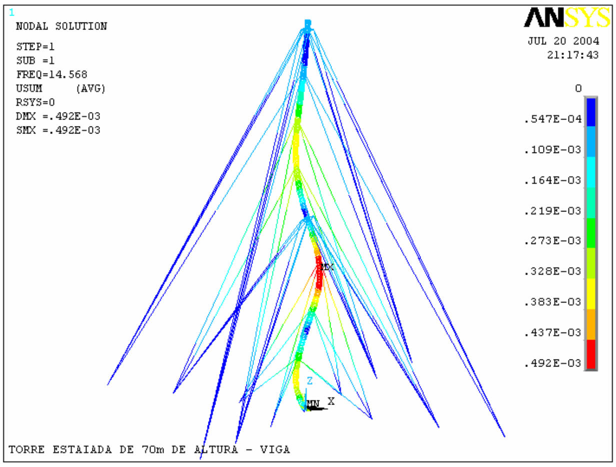
Beam element modelling



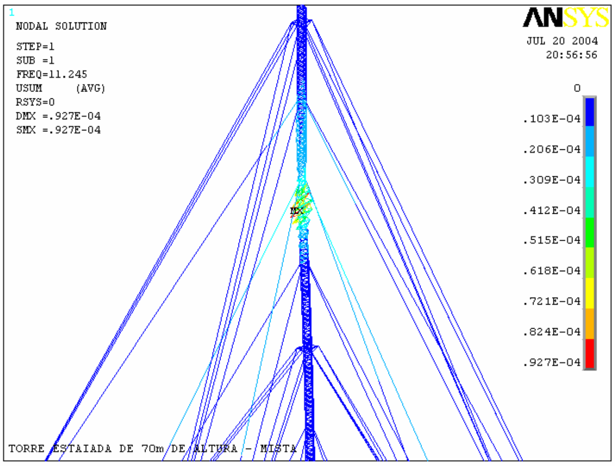
Mixed beam and truss element modelling

Figure 10. 50m high tower first bucking mode, perpendicular wind load case.

Figures 10 to 12 illustrate the towers first buckling modes associated with the perpendicular wind load case, for the last two modelling strategies. It can be noticed that all the instability modes are associated with local effects, except for the 70m high tower, when represented by beam elements. In this case the instability mode is defined by a global structural system behaviour, Fig. 11.



Beam element modelling



Mixed beam and truss element modelling

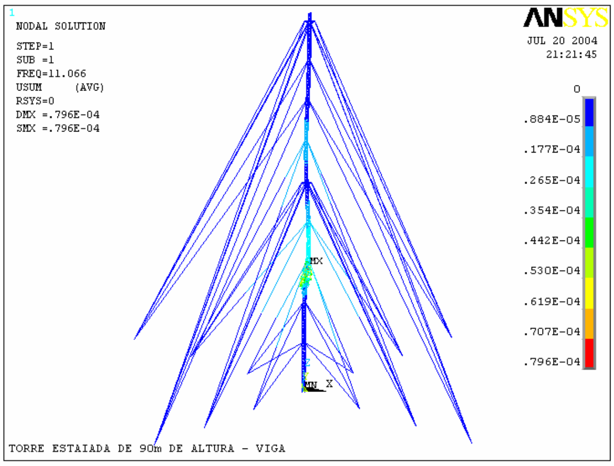
Figure 11. 70m high tower first bucking mode, perpendicular wind load case.

Non-linear Analysis

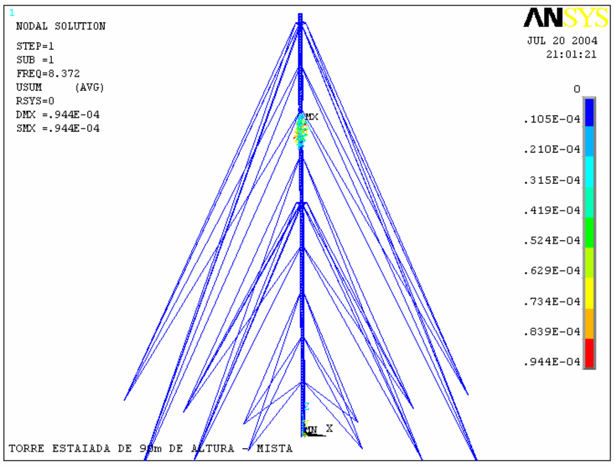
A guyed tower is generally very sensitive to non-linear effects. These effects can produce substantial influence over the structural behaviour indicating that a preliminary investigation should be made before neglecting them. Because of its low stiffness, the tower is subjected to large displacements even under service loading conditions. Other source of non-linearity is the tension-only nature of the guys. The prestressing level of the guys should be enough to keep its status as “in tension” and never allowing a slack configuration.

To investigate the presence of these two above described non-linearities, a large displacement analysis was performed considering the guys as cable (tension-only) elements, as illustrated in Fig. 3, for the 90m high steel tower structure. The updated Lagrangean formulation, based on the Newton-Raphson method, was used to model the geometric non-linearity (ANSYS, 1998).

Three load cases are considered to determine the load level in which the non-linearities can be clearly observed: Case I - Wind loads are applied after gravity loads and guys prestressing; Case II - Wind loads are duplicated and applied after gravity loads and guys prestressing and Case III - Wind loads are quadruplicated and applied after gravity loads and guys prestressing.



Beam element modelling



Mixed beam and truss element modelling

Figure 12. 90m high tower first bucking mode, perpendicular wind load case.

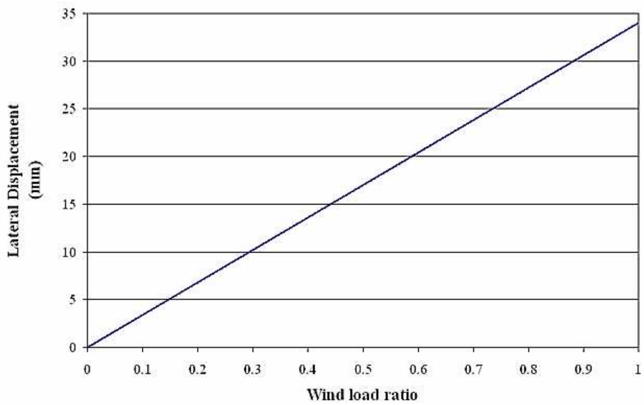


Figure 13. Tower top displacement versus nominal wind load ratio (Load condition I).

It is important to notice that gravity loads and the forces due to guys prestressing are symmetric and do not induce lateral deflections before the wind loads are introduced into the system.

For the first and second load condition, non-linear effects related to the guyed steel tower structure, cannot be observed as depicted in Figs. 13 and 14. Alternatively, when the quadruple of the wind load were applied to the system the non-linear effects can be clearly identified, Fig. 15.

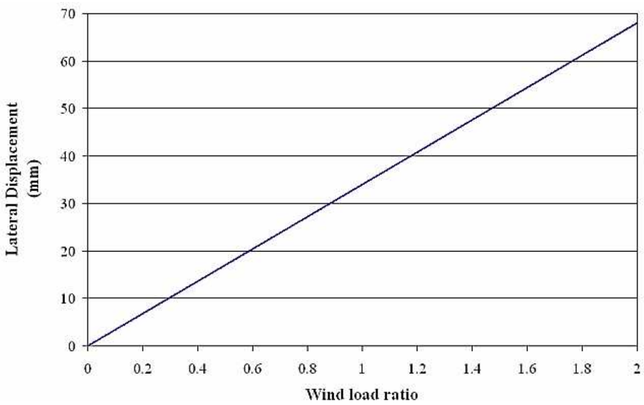


Figure 14. Tower top displacement versus nominal wind load ratio (Load condition II).

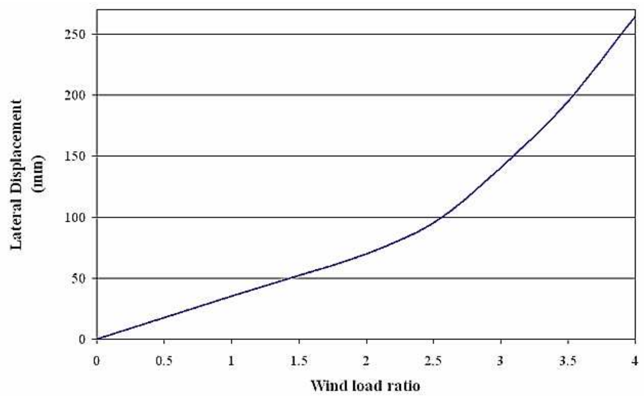


Figure 15. Tower top displacement versus nominal wind load ratio (Load condition III).

Final Remarks

This paper proposes an alternative structural analysis modelling strategy, based on qualitative and quantitative comparisons, for guyed steel towers. The proposed methodology, less conservative than traditional analysis methods, uses a combined solution of three-dimensional beam and truss finite element to model the structural behaviour of 3D tower structures under several loading conditions.

Generally, in all the cases studied, the maximum stress values for the structural tower modelling based on the three investigated methodologies were significantly modified. On the other hand, the lateral displacement values were not significantly changed when the usual truss model, the beam model or the combined beam and truss model were considered.

Based on the difficulties found in the analysed guyed steel towers, present in current engineering design practice, and corroborated by the nature of the 3D truss finite element, an analysis only using this element cannot be indicated. This method also implies in the utilization of a great number of dummy bars to prevent the occurrence of structural mechanisms. This fact increases the amount of work to model the structure and generates a potential error source if the rigidities and/or number of dummy bars were not properly considered.

Despite the numerous advantages related to the use of beam finite elements its adoption for modelling the investigated steel tower should be carefully addressed. The designer should bear in mind that if rigid connections were used in the design model, higher buckling load values would be produced, overestimating the actual values. The effects produced by the adopted modelling strategy were small when the dynamical behaviour of the systems is considered. The correspondence between natural frequencies and mode shapes is dependent of each studied model.

A non-linear analysis was performed and non-linearities were detected only for extreme load conditions. These results can be used to validate the initial assumption of the system linear behaviour for the usual limited load ranges used in design practice.

Finally, based on the obtained results for the investigated tower geometries, the authors would like to suggest the adoption of the third mixed strategy in which the bracing systems are modelled by truss elements.

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