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Experimental and analytical study of guyed mast with variable cable tensions

Estudio analítico-experimental de una torre atirantada con tensiones variables en los cables

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Guyed masts are a key component of the telecommunications infrastructure in several countries. Due to their characteristics, these structures are particularly susceptible to dynamic loading originated from strong winds, causing several collapses in recent decades. These structural failures evidence the need to improve the computational models currently used in the design process. The design process is generally based on simplified computational models of the structure, assuming simplified characteristics of its geometry or constructive design values which generally differ from the real characteristics. In the present work, a comparison is made between the modal characteristics and the structural response obtained for two computational models of an existing tower. In the first case, an idealized model of the structure is used, while in the second case, an updated model based on a field study is used. The results obtained show that the modal displacements are the modal parameter most sensitive to structural variations, whereas the study of the structural response of the tower reveals that the updated model presents efforts up to 50% larger on the structural elements for the analyzed wind loads, due to its initial configuration influenced by the asymmetrical prestressing of the cables.

Keywords: guyed masts, dynamic analysis, prestress forces, wind loads

Las torres atirantadas son un componente clave de la infraestructura de telecomunicaciones en varios países. Debido a sus características, estas estructuras son particularmente susceptibles a las cargas dinámicas originadas por fuertes vientos, causando varios colapsos en las últimas décadas. Estas fallas estructurales evidencian la necesidad de mejorar los modelos computacionales utilizados en su diseño. El proceso de diseño se basa generalmente en modelos computacionales simplificados de la estructura, asumiendo características simplificadas de su geometría o valores constructivos que generalmente difieren de las características reales. En este trabajo se realiza una comparación de las características modales y la respuesta estructural obtenida para dos modelos de una torre existente. En el primer caso, se utiliza un modelo idealizado de la estructura, mientras que, en el segundo caso, se utiliza un modelo actualizado basado en un estudio de campo. Los resultados obtenidos muestran que los desplazamientos modales son el parámetro modal más sensible a las variaciones estructurales, mientras que el estudio de la respuesta estructural de la torre revela que el modelo actualizado presenta esfuerzos hasta un 50% mayores sobre los elementos estructurales para las cargas de viento analizadas, debido a su configuración inicial influenciada por el pretensado asimétrico de los cables.

Palabras clave: torres atirantadas, análisis dinámico, fuerza de tesado, carga de viento

Introduction

The development of telecommunications in the last decades has woken up the interest for the study of lattice towers, due to their structural advantages. These slender, permeable structures allow for a high resistance with lower use of materials while reducing the forces provoked by the wind over the structure. Guyed masts are part of the national radio and TV broadcasting systems and support communications for both civil and military applications, which are vital services for the communities. The failure of a tower has relevant consequences for the communications network, national security and a high social impact. For this reasons, the evaluation of these structures is beyond a techno-economic analysis.

In recent years, Cuba has experienced the passage of numerous hurricanes and other large-scale meteorological phenomena, causing total or partial failure of several telecommunications towers. The need to prevent failures in these structures, due to their strategic importance in the country, even under hurricane wind loads, has motivated the study of their structural behavior, as well as the factors present in the failures that could lead to increased structural vulnerability. Several studies conducted in Cuba (Elena *et al.*, 2013, 2015, 2019), have shown that within the collapsed structures, guyed masts represent approximately 80% of the total, highlighting the uncertainties still present in their design and calculation process. Among the main factors causing such uncertainties is the discrepancy between the actual structure and the numerical models used for design and calculation. This area has been a field of growing attention in recent decades (Ballaben *et al.*, 2017; Ewins, 2000; Fernández *et al.*, 2018; Reynders *et al.*, 2016), leading to the development and popularization of techniques for identifying the dynamic response of structures and updating their computational models based on the identified modal parameters, typically frequencies or periods of oscillation, modal displacements, and modal damping ratios. Such procedures, such as the Operational Modal Analysis (OMA), are based on estimating these modal parameters from vibration data measured using sensors (accelerometers, strain gauges) placed on the structure.

Additionally, the dynamic behavior of computational models generally differs from the real response of the structure. In the case of guyed towers, this difference is exacerbated due to their nonlinear behavior, as small changes in mass, stiffness, or damping in the structure can generate significant variations in the dynamic response of the system. These changes are not uncommon, as the total quantity and position of antennas installed at each station are frequently modified according to the needs of the national broadcasting plan, generating variations in the mass distribution on the structure. Similarly, cable tension can vary due to cable relaxation over time and structural movement. Furthermore, cables can occasionally be subjected to excessive tension during the assembly process due to inadequate measurement of the tension values specified in the project.

In this work, the influence of geometry, cable tension, and mass distribution on the modal characteristics of an 80-m-tall guyed mast located in Santa Cruz del Norte is studied. For this purpose, a comparative study is conducted using two finite element models (FEM) in the SAP2000 (2018) software. The first FEM is built based on idealized data of the structure, assuming the geometric characteristics and cable tension described in the construction plans, as well as an antenna distribution based on a survey conducted in 2008. The second FEM is built based on data acquired during a recent field study. The updated model of the structure takes into account three relevant aspects of the structure: the spatial distribution and mass of ancillaries, the asymmetry of cable tensions and the vertical asymmetry of the anchorage points of the cables. Both models are used to determine the variation of the modal characteristics of the structure (oscillation frequencies and modal shapes) when using updated data in the construction of the model. Finally, the structural response to wind loading is obtained for both models using a static-equivalent analysis in SAP2000 (2018), and the maximum axial forces in the main elements are compared.

Description of the structure

The selected structure for the study is a Babiney model guyed tower located in the region of Santa Cruz del Norte, Cuba, as shown in Figure 1. This structure was chosen considering various geometric, spatial, and geolocation characteristics, as well as its accessibility, with the aim of extrapolating the study results to the majority of existing towers in Cuba. The height of the tower was limited by the operators' ability to transport equipment to the top during measurements. Other factors taken into account for the selection were the level placement of the cable anchors on the ground, the presence of anti-torsion devices that allow studying their influence on the modal identification procedure of the structure, and the spatial location of the tower on a hill.

Figure 1: (a) General view of the guyed mast in Santa Cruz del Norte, and (b) base of the tower.

General characteristics of the structure

The selected tower consists of a 75.5 m high shaft, on top of which a tubular mast measuring 4.5 m in height is located, resulting in a total height of 80 m. The lateral stiffness of the structure is ensured by 21 guyed cables, oriented in 3 directions and spaced approximately 120º apart in plan view. The cables are distributed in 5 levels per vertex, with one cable per level, except for levels 2 and 4, which have two cables due to the presence of anti-torsion systems on the shaft of the structure. The geometric characteristics of the tower and cables (according to construction plans) are detailed in Figure 2.

All elements of the shaft are made of equal-sided angle profiles and ASTM A-36 structural steel. The columns are formed by two back-to-back L750 x 80 mm profiles, bolted together using evenly spaced steel plates every meter and forming an interior angle of 60º. The shaft is braced by horizontal struts spaced every meter and cross-bracing on each face, both formed by L500 x 50 mm profiles. The cross-sectional shape of the shaft is further reinforced by interior struts formed by L400 x 40 mm profiles, as shown in Figure 2. As for the cables, the first four levels consist of 1 x 7+0 braided steel cables with a diameter of $\varphi = 13$ mm (ultimate load $P_u = 162$ kN), while the last level of cables is formed by 1 x 19+0 braided steel cables with a diameter of $\varphi = 16$ mm (ultimate load $P_u = 235$ kN). High yield strength structural steel is used for all cables. The material properties are shown in Table 1 and were considered constant for the study.

Table 1: Characteristics of the structural steel of the tower.

Parameter	Shaft elements	Cables
Material density ρ, kg/m^3	76.97	76.97
Elastic modulus E, MPa	1.99 x 10^5	1.99 x 10^5
Poisson´s ratio v	0.3	0.3
Coeficcient of thermal expansion α	1.17 x 10^{-5}	1.17 x 10^{-5}
Yield stress F_y, MPa	250	1600
Ultimate stress F_u, MPa	400	2000

Loads

The loads considered in the analysis are: a) the self-weight of the elements, b) the pre-tensioning load on the cables, c) the weight of the antennas, anemometer supports, and electrical panels on the structure, and d) the loads due to wind action on the structure. The self-weight of other auxiliary elements, such as stairs and support grids, is not considered in the finite element models. The masses of the antennas are assigned to the finite element model at the intersection nodes of the columns and horizontal struts in order to simplify the analysis of the structure. For the idealized model, the quantity and position of the antennas are determined based on pre-existing data from a study conducted in 2008, while for the updated model, the information collected during a field study conducted in March, 2022 is used. The data for the mass and distribution of the antennas considered in each model are shown in Table 2.

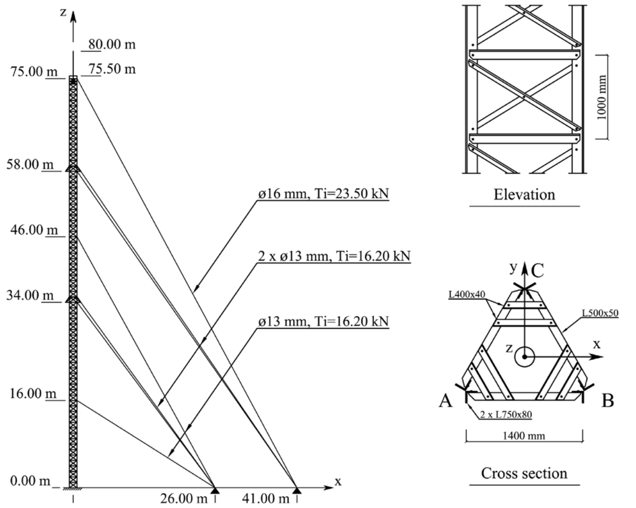


Figure 2: Geometric characteristics of the tower located in Santa Cruz del Norte, according to as-built plans.

Table 2: Data of the ancillaries

No.	Antenna type	Height, m	Weight, kg	Vertex	Area, m^2
Idealized model (2008)					
1	Dish antenna	17	9.0	A	1.54
2	Dish antenna	18	5.5	B	1.13
3	FM antenna	23.5 - 26.5	6.0	B-C	-*
4	Electrical panel	31.0 - 32.5	38.7	A, B, C	0.72
5	Close dipole antenna (4u)	36.5 – 41.0	5.0	A	-*
6	Open dipole antenna (4u)	46 - 55	4.5	A	-*
7	UHF panel (2u)	60, 61	13.6	A, C	2.9
8	Dipole antenna Band III (2u)	63.5 - 66.5	25.0	A, B, C, D	0.6
9	Dipole antenna Band III (4u)	69.5 - 72.5	85.0	A-B, B-C	1.89
10	UHF panel (24u)	75.5 – 80.0	15.65	6 x 4	2.9
	Total weight, kg		890		
Updated model (field study March 2022)					
1	Anemometer support	9	62	A-C	0.16
2	Yagi-Uda directional antenna	11	2.3	C	-*
3	Yagi-Uda directional antenna	13	2.3	C	-*
4	Yagi-Uda directional antenna	13	1.7	B	-*
5	Dish antenna (φ=90 cm)	15	18	C	0.64
6	Dish antenna (φ=90 cm)	16	18	B	0.64
7	Dish antenna (φ=120 cm)	20	18	A	1.13
8	Dipole antenna	23	5.3	A	-*
9	Yagi-Uda directional antenna	24.5 - 27.5	74	A	0.6
10	VHF antenna	25.5	2.5	B	0.8
11	Anemometer support	29	62	A-C	0.16
12	Yagi-Uda directional antenna (UHF)	32	7	C	0.65
13	Dipole antenna FM	36	13	B	-*
14	UHF single panel (3u)	42 - 45	30	A, B, C	
15	Dipole antenna (4u)	47 - 56.5	58	B	-*
16	Tubular antenna	51	8.1	A	-*
17	Anemometer support	56	62	A-C	0.16
18	Anemometer support	57	62	A-C	0.16
19	UHF modular bay 4 panels (2u)	59.5 - 61.5	84	A, B	1.2
20	VHF antenna 4 dipoles (4u)	63 - 66	160	A, B, C	0.43
21	VHF antenna 4 dipoles (1u)	67 - 70	56	A-B	0.20
22	VHF antenna 2 dipoles (3u)	70 - 73	148	A-B, B-C, C-D	0.32
23	UHF single panel (2u)	77 - 78	26	A-B, C-A	0.6
24	UHF single panel (3u)	78 - 79	39	A-B, B-C, C-D	0.9
25	UHF single panel (3u)	79 - 80	39	A-B, B-C, C-D	0.9
	Total weight, kg		1058		

*: The exposed area to the wind is considered negligible because is less than 0.1 m^2

The effect of cable pre-tensioning is taken into account in the computational models through a nonlinear analysis. The pre-tension load on the cables is considered as a target force applied at the cable's end that is connected to the ground. The target force load is a special type of load where a specified deformation is iteratively imposed on the cable until the target force in the cable is achieved. This iterative nonlinear analysis provides the initial equilibrium state of the model, which takes into account the self-weight of the elements and the stiffening of the structure due to the force applied on the cables. In the case of the idealized model, the target force for each cable, T_i, is selected as 10% of the cable's breaking load, as specified by the design codes (ANSI/TIA-222-G, 2006). For the updated model, the actual force acting on each cable was determined based on the average of three consecutive measurements, taken using a Dillon Quick-check tension meter. The tension forces used in each model are shown in Table 3. The notation used in the table for the cables takes the form $\alpha - \beta\gamma$, where α is the vertex on which the cable's ground anchor is located, β indicates the anchor number, and γ indicates the cable's anchor level on the tower shaft. Figure 3 illustrates the notation used for vertex A.

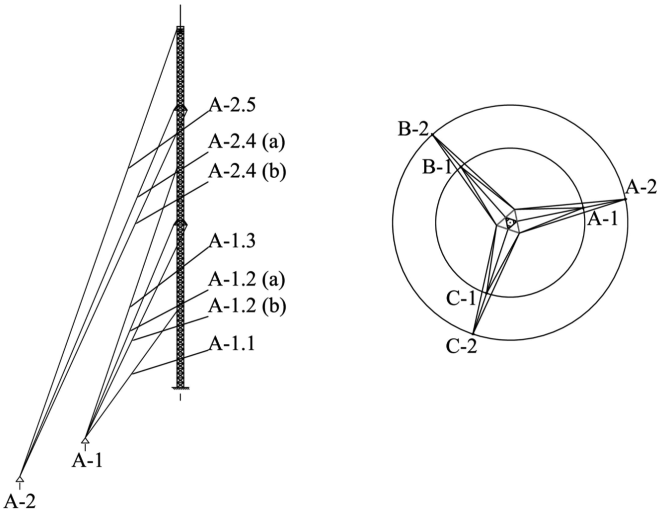


Figure 3: Example of cable notation on vertex A.

Table 3: Prestress forces in the cables for the idealized (F_{dis}) and updated model (F_{meas})

Cable	Design force, kN	Measured force, kN				Δ, kN
	F_{dis}	F_1	F_2	F_3	F_{meas}	
A-1.1	16.20	22.40	22.20	22.00	22.20	6.00
A-1.2 (a)	16.20	17.00	17.00	17.00	17.00	0.80
A-1.2 (b)	16.20	16.40	16.20	16.40	16.33	0.13
A-1.3	16.20	25.80	25.80	25.60	25.73	9.53
A-2.4(a)	16.20	18.40	18.40	18.40	18.40	2.20
A-2.4(b)	16.20	20.60	20.60	20.60	20.60	4.40
A-2.5	23.50	31.40	30.80	31.00	31.07	7.57
B-1.1	16.20	20.40	20.20	20.00	20.20	4.00
B-1.2 (a)	16.20	13.20	13.00	12.80	13.00	-3.20
B-1.2 (b)	16.20	10.00	10.20	10.00	10.07	-6.13
B-1.3	16.20	16.40	16.00	15.80	16.07	-0.13
B-2.4(a)	16.20	27.20	27.40	27.00	27.20	11.00
B-2.4(b)	16.20	22.00	22.40	22.60	22.33	6.13
B-2.5	23.50	27.00	27.80	27.40	27.40	3.90
C-1.1	16.20	16.00	16.00	16.00	16.00	-0.20
C-1.2 (a)	16.20	14.60	14.60	14.20	14.47	-1.73
C-1.2 (b)	16.20	18.80	18.60	18.60	18.67	2.47
C-1.3	16.20	15.20	14.60	15.00	14.93	-1.27
C-2.4(a)	16.20	20.20	20.20	20.20	20.20	4.00
C-2.4(b)	16.20	22.20	22.80	22.40	22.47	6.27
C-2.5	23.50	30.20	29.80	29.20	29.73	6.23

Wind load

To determine the wind load, the Patch Load method proposed in the Eurocode (EN 1993, 2006) is applied. The Patch Load method uses a series of static load segments that are applied to the shaft and used to estimate the fluctuating component. The results of these segment loads, individually applied to the shaft, are combined and added to the mean component to obtain the dynamic response of the structure. This method is known as the Patch Load method and is used in specific tower standards for telecommunications (Eurocode EN 1993, 2006; US standard ANSI/TIA-222-G, 2006). The method was introduced in 1981 and subsequently refined based on research conducted by Gerstoft and Davenport (1986) and Sparling *et al.* (1996). The basic wind speed used was 33 m/s for a 10 min averaging interval, as proposed in the updated Cuban wind standard NC285 (2003). For the analysis, four main wind directions were considered: 0°, 60° and 90°, as recommended by design standards for symmetric masts (ANSI/TIA-222-G, 2006; EN 1993, 2006; NC285, 2003) and an additional wind direction of 120°, which was included due to the asymmetry of cable tensions in the mast. The wind directions considered are shown in Figure 4.

The wind load on the cables was considered uniformly distributed. The value was calculated taking into account the basic wind speed and the coefficients corresponding to half the height between the cable's anchor point and its attachment level on the shaft. The shape coefficient for all cables was taken according to NC285 (2003). The force was applied in the direction of the wind, considering the angle between the wind vector and the cable, depending on the specific analysis case.

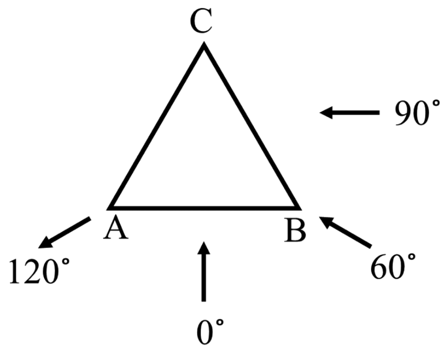


Figure 4: Wind directions considered in the structural analysis.

The wind load on the antennas was calculated considering the basic wind speed, and the shape coefficients were determined based on the type of antenna using values proposed in the manufacturers' catalogs.

Load combinations

For the case of telecommunication towers, the combination of loads is established based on the permanent load and the wind load according to the NC450 (2006). The combinations used are: 0.9 CP + 1.4 CV and 1.2 CP + 1.4 CV, where CP represents the permanent load and CV represents the extreme wind load. In order to account for the prestressing effect of cable tensions, which is obtained through an initial non-linear analysis (target force), the load combinations were defined as additional load cases, and the stiffness matrix used for each load case was the modified stiffness matrix obtained at the end of the target force analysis.

Finite element models

This section describes the two finite element models used in the comparative study. Both models were constructed in SAP2000 (2018). This software was chosen because it allows the use of non-linear structural elements to model the behavior of cables, as well as modules for conducting non-linear analysis, which take into account the variation in stiffness of the structure due to cable tensioning. In both cases, non-linear CABLE type elements with 16 nodes were used to model the cables, and linear FRAME type elements were used to model the shaft bars. Both finite element models consist of a total of 2910 elements (FRAMES + CABLES) and 1472 nodes.

The connections between the members of the shaft were considered hinged in all cases except for the columns and the mast located at the top. The columns were modeled continuously from the base to the top of the shaft because the connection between them is made through rigid double-sided plates. In the case of the tubular mast, it was considered continuous because it is composed of a solid steel tube 6 m long. The supports at the base of the columns were considered fixed, restricting the 6 degrees of freedom (DOF) in space, while the connection of the cables to the shaft and to the ground was considered hinged in both cases due to the inability of the cables to take moment.

Idealized model

The idealized model was developed based on the data specified in the tower's construction plans and diagrams. The cable tension forces specified by design F_{dis} were considered, which are shown in Table 3 and are symmetrical with respect to the global axes (x, y, z) considered. Regarding the geometry of the cables, it was considered for this model that the ground anchors of the 7 levels of cables are all at the same level as the base of the shaft, ensuring the symmetrical arrangement of the cables' geometry. The masses of antennas and auxiliary elements assigned to the model were determined from available information obtained from a technical survey conducted in 2008. The idealized finite element model is shown in Figure 5(a).

Model updated based on field study

For the update of the finite element model, a field study was conducted to obtain the actual characteristics of the structure on site. A topographic station (Figure 6(a)) was used to determine the level of the cable ground anchors with respect to the base of the shaft, as well as the distance between them and the base of the shaft. Additionally, the verticality of the structure was verified. The cable tension force was measured on site using a DILLON Quick-Check tension meter (Figure 6(b)). A total of 3 readings were taken on each cable (Figure 6(c)), and the acting force F_{meas} was determined from the average of the recorded values. The values recorded in each reading and the average force obtained are shown in Table 3.

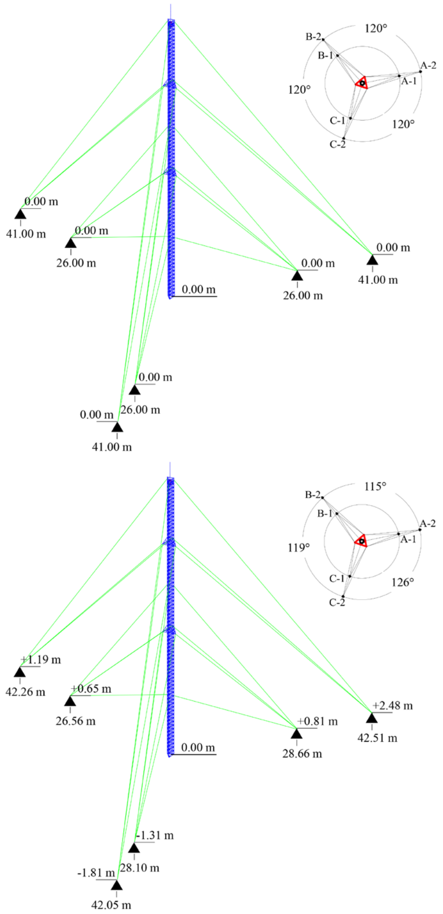


Figure 5: (a) Idealized FEM in SAP2000 and (b) updated FEM in SAP2000, from field study

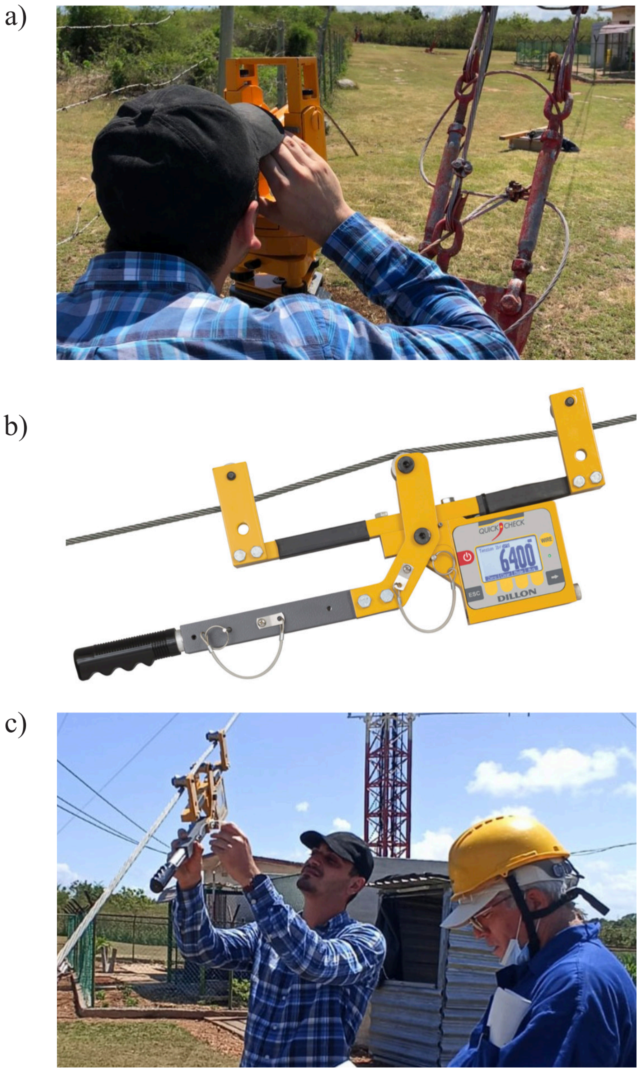


Figure 6: (a) Topographic station employed for the obtention of ground level, (b) DILLON Quick-Check tension meter, and (c) measurement of prestress forces in the cables

Finally, an updated survey of the quantity, type, and position of antennas on the tower was carried out, with the aim of updating the masses of antennas on the structure. The antennas observed in this study, as well as their position on the tower, are shown in Table 2. The major discrepancies found consists of the existence of a significantly higher number of antennas, with a total weight 18.9% higher than that considered in the idealized model, and the asymmetry both in the geometric arrangement of the anchorages and in the tensioning force of the cables. The finite element models obtained for the idealized model and the updated model, as well as the geometrical differences observed are shown in Figures 5(a) and 5(b).

Results and discussion

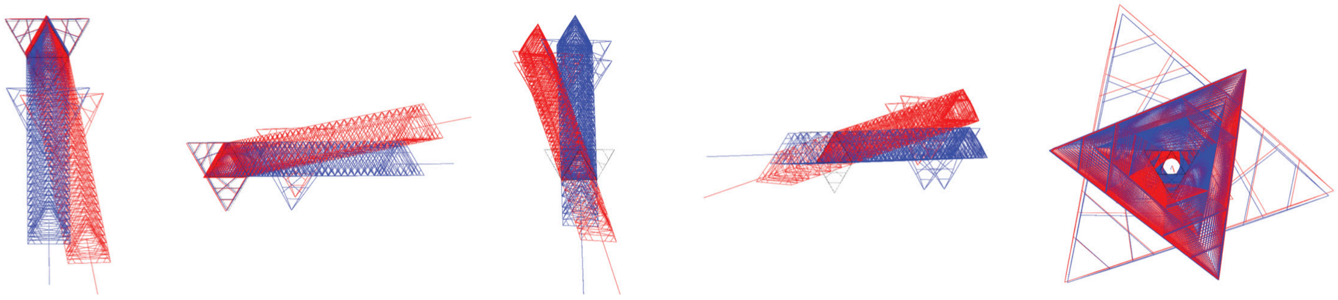
Modal parameters variation

Based on the two FEMs created, a modal analysis was conducted in SAP2000 (2018) to determine the natural frequencies and corresponding modal displacements for the first 12 modes of the structure. First, a nonlinear analysis was performed in SAP2000 to determine the initial configuration state of the structure under self-weight loads, antenna masses, and cable tensioning. Then, the modified stiffness matrix corresponding to the initial state of the structure was then used to determine the modal characteristics. The method used in the modal analysis was the Eigenvectors method, as it allows for the identification of the frequencies and oscillation modes of the structure independently of the applied external loads.

Table 4 shows the natural frequencies obtained for the two models analyzed. It is observed that the frequencies obtained for the updated model do not present significant differences compared to the idealized model, with maximum variations of 6%. However, the idealized model presents even flexural oscillation modes for orthogonal axes, meaning it has very similar frequency values for the *x* and *y* axes. In contrast, in the updated model, the frequencies in even flexural modes differ slightly from each other due to the asymmetry in the position of the anchorages and cable tensions. This behavior is also observed when comparing the modal displacements of the idealized model and the updated model. For the updated model, which exhibits asymmetry in the acting cable tensions, the flexural modes are not predominantly found on the *x* and *y* orthogonal axes, but manifest as a combination of bending around these two axes, with one being slightly more predominant. In the case of torsional modes, there is no notable difference in the modal shapes obtained for both models. This behavior is illustrated in Figure 7, which shows the modal shapes obtained for the first four flexural modes and the first torsional mode.

Table 4: Natural frequencies obtained for the idealized (λ_{ideal}) and updated (λ_{upd}) models of the guyed mast.

	Mode number / type											
	1	2	3	4	5	6	7	8	9	10	11	12
	BY-1	BX-1	BY-2	BX-2	BY-3	BX-3	T-1	BY-4	BX-4	T-2	BY-5	BX-5
λ_{ideal} Hz	1.52	1.52	2.61	2.61	3.76	3.77	4.23	5.62	5.63	7.27	7.59	7.59
λ_{upd} Hz	1.51	1.58	2.60	2.71	3.76	3.85	4.46	5.90	5.95	7.26	8.00	8.05
Δ,%	-0.59	3.94	-0.38	3.52	0.00	2.18	5.44	4.86	5.61	-0.18	5.47	5.99



BY-1 a) BX-1 b) BY-2 c) BX-2 d) T-1 e)

Figure 7: Top view of the first 4 flexural modes and the first torsional mode of the idealized (blue) and updated (red) models.

Structural response under wind load

The axial forces of the studied tower were analyzed for both models considered in the study, considering the four wind load directions mentioned earlier (0˚, 60˚, 90˚ and 120˚), as well as the two previously discussed load combinations (0.9 CP+ 1.4 CV and 1.2 CP+ 1.4 CV). The structural response values considered in the study were the maximum axial load per structural element (columns, braces, diagonals, and cables) and the behavior of the axial force as a function of height for the three columns of the tower.

Figures 8(a) to 8(d) show the highest values of axial load per structural element of the tower for the four wind directions and the two load combinations analyzed. It is shown that for columns and diagonals, the most unfavorable angle of attack is 120˚, whereas for the braces the highest axial loads are obtained for 0˚ and for the cable the highest tensions are produced by the 90˚ direction. Another remarkable observation is that the type of load combination does not have a significant influence in the maximum values of axial force obtained, regardless of the structural element, wind direction or type of finite element model considered. The load combination does not generate variations of the maximum axial loads larger than 4%.

Regarding the difference in the maximum values of axial load obtained for the idealized or updated model, the largest differences are found in the columns for the wind directions of 0˚ and 120. For 0˚, the idealized model yields compression forces 80% higher than the updated model, whereas the opposite is noted for a wind direction of 120˚, where the updated model shows values 50% higher than the idealized model. This behavior is explained by the asymmetry of the cable tensions in the mast, whose unbalance causes an initial deformation of the mast in the direction of vertex A. This initial deformation causes the column of vertex A to have a larger initial compressive force, and the columns on vertex B and C a smaller compressive force, due to the beam-like behavior of the

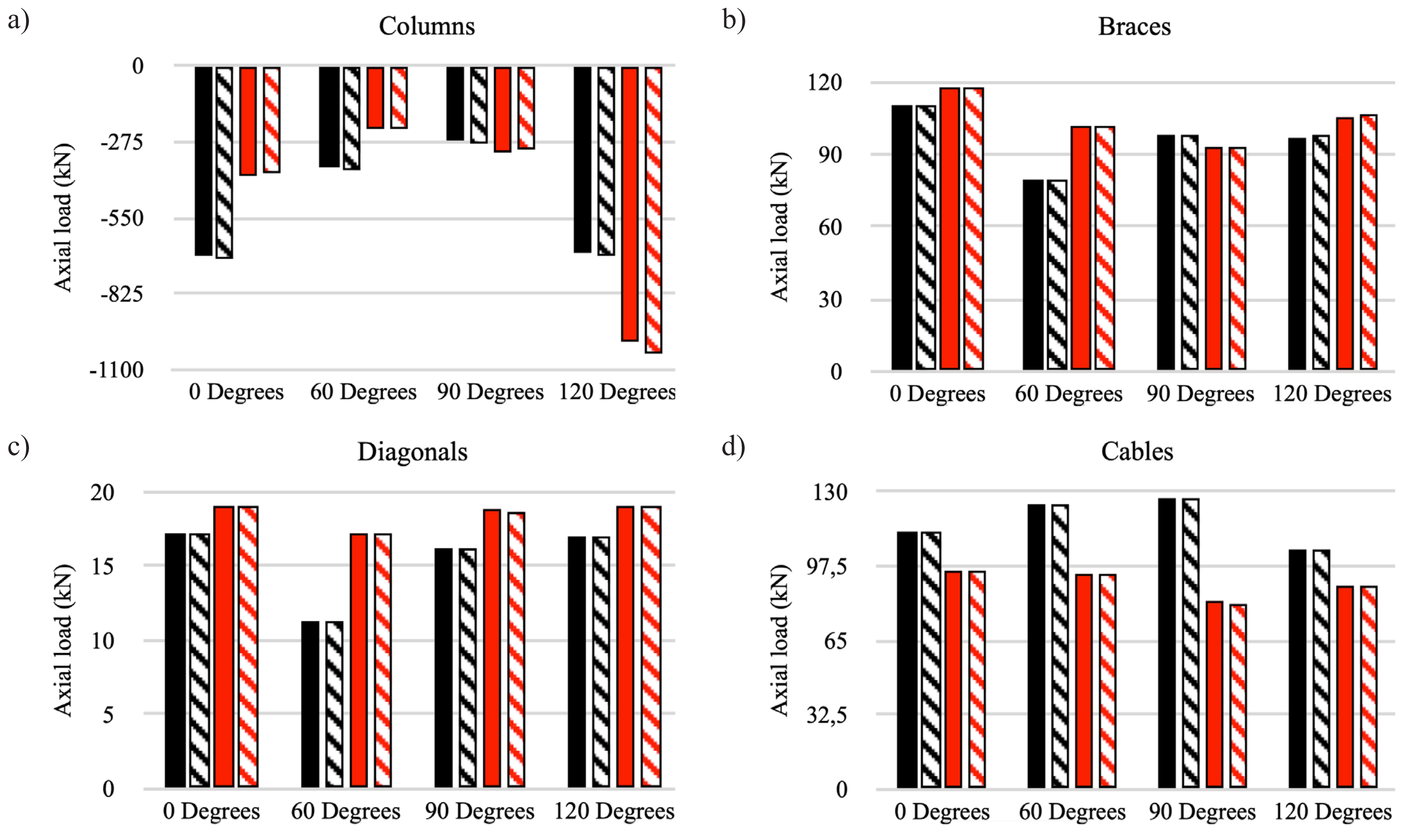


Figure 8: Maximum values of axial load obtained in (a) columns, (b) horizontal braces, (c) diagonals, and (d) cables of the studied tower, for the idealized model (black) and the updated model (red) and for the load combinations 0.9CP+1.4CV (solid) y 1.2CP+1.4CV (hatched).

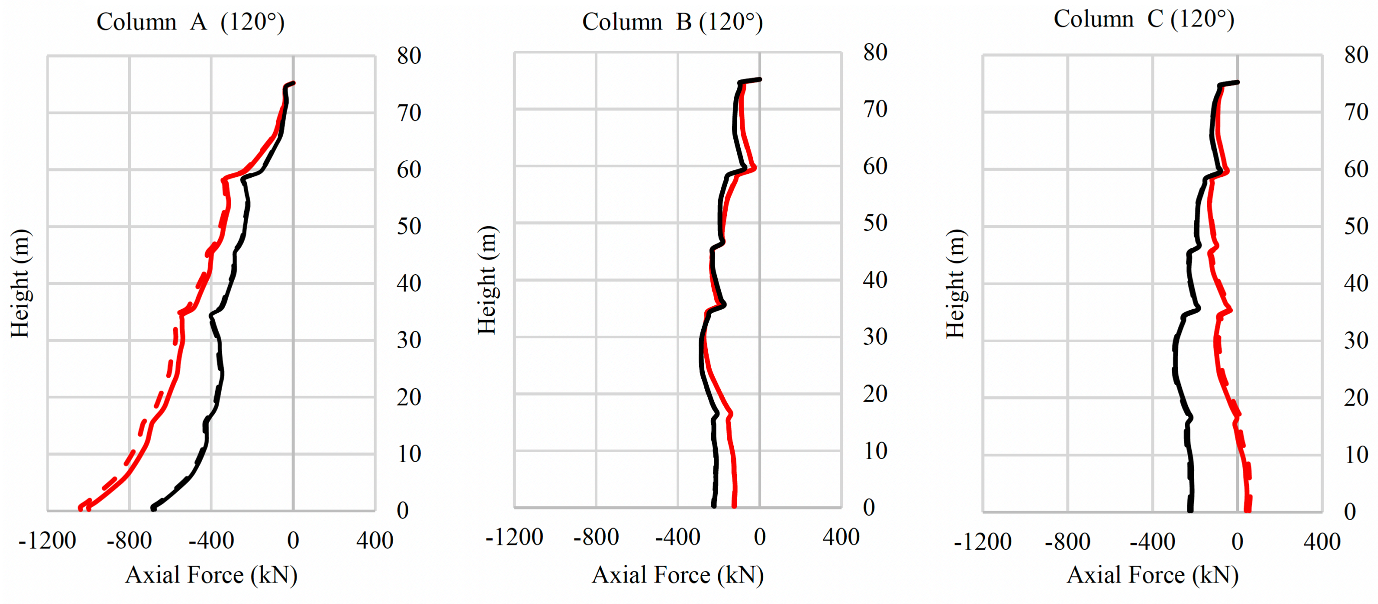


Figure 9: Axial force obtained in the columns of the shank for the 120 degrees direction in the idealized model (black) and the updated model (red) and for the load combinations 0.9CP+1.4CV (continuous line) and 1.2CP+1.4CV (dash line)

mast. For the attack angle of 0˚, the wind load counteracts the initial deformation, and larger compressions are obtained for the idealized model instead, where no initial lateral deformation occurs. The inverse is observed for the wind direction of 120˚, where the initial compressions are worsened by the lateral wind load.

Figure 9 shows the axial force as a function of height for the three columns of the structure, located on vertices A, B and C, respectively, for the critical wind direction of 120˚. In the columns located on vertices B and C, the idealized model has higher values of axial load, reaching percentage differences at the lower heights of the tower of around 45% for vertex B and around 130% for vertex C. At vertex A, the updated model presents higher values of axial load in the columns with differences of around 50% at the base of the tower.

It should be noted that the absolute maximum axial load values are obtained for the updated model and a wind direction of 120˚, indicating the importance of considering the asymmetry of the cable tensions and all unfavorable attack angles of the wind.

Conclusions

The results obtained from the comparative study between the idealized and updated finite element models lead to the following conclusions:

There are no significant differences in the natural frequencies for the updated model, with maximum differences of 6% compared to the idealized model; however, the consideration of asymmetric tensions in the cables has a marked influence on the modal displacements corresponding to the flexural modes, causing them to tilt with respect to the geometric axes of symmetry.

The highest axial loads in the structural elements of the shaft are obtained for the updated model, with values between 6% and 10% higher in the case of diagonals and braces; and up to 50% higher for the case of the columns. For the cables, the maximum tensions are obtained for the idealized model.

While the wind direction 90° proved to be the most unfavorable in the case of the cable tensions, the highest internal forces in columns and diagonals were obtained for the additionally considered wind direction of 120°, indicating the importance of considering additional wind directions when there is asymmetry in the geometry and cable tensions.

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