Structural Behavior of a Guyed Mast

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ABSTRACT: The inherent nonlinearity in the structural behavior of guyed mast leads to difficulties in their structural analysis, and prevents the formulation of a general-purpose design methodology [1]. The current design standard in Cuba [2] uses a static approach to consider the fluctuating wind component, and a high occurrence rate of structural failures, causing the government to lose millions of dollars on repairs, creates the need for the study, optimization and review of existing models of guyed mast. The study of the structural behavior under extreme winds and the number of failures by type of elements was performed.

Two methodologies for calculating the fluctuating wind components were used: NC 285: 2003 (current Cuban code) and the Patch Load proposed by Sparling and adopted by the British Standard [3]. The improvement of the tower design focused on the aerodynamic factor in columns and changes of the width of the cross section of the mast. The results showed that the proposed tubular column sections reduces the maximum axial force by up to 24% and significantly reduces the work of all elements of the mast, cables and supports. The increased width of the transversal section of the shaft caused a detrimental increase to the internal axial forces. Decreasing the cross section, however, produces lower axial forces in cables and columns. In horizontal members there were no significant results and an increase in the compression axial forces occurs for the diagonal member.

From the comparison of the Patch Load method with the static method of the Cuban Standard, it is concluded that the internal forces increase, in all types of elements, with the implementation of Patch Load method.

KEY WORDS: Guyed Mast; Structural Behavior; Wind Load; Spatial Arrangement.

1 INTRODUCTION

The analysis and design of masts and towers requires special knowledge and experience, especially when it concerns guyed mast. The special problems related to these structures are underlined by many collapses during the years [4]. For example, in Cuba, in the 1996-2006 period, 33 telecommunication towers failed due to strong winds associated with hurricanes and severe storms. Likewise, in the hurricane season of 2008, 20 towers were damaged by high-speed winds [5]. The number of collapses of guyed masts is relatively higher than other types of structures [6], due to the nonlinear behavior of guy ropes, which has led to numerous investigations into its structural behavior. Furthermore, with the current knowledge on guyed towers and calculation tools available, the study and revision of existing models established for decades is justified.

The predominant load on self-supporting towers and guyed mast is the wind load, although in some areas the atmospheric icing and seismic loads may have a significant influence in the design. The wind is a dynamic load, and slender structures are sensitive as they have low damping characteristics.

The resistance of a lattice tower dependent on two parameters: the meteorological parameter, which determines the wind speed and the shape parameter, which determines the aerodynamic coefficients. In these structures, the member’s shape, dimensions and spatial distribution, are aspects that significantly influence the wind load calculation. The flat-sided sections, commonly used in Cuba, have a higher wind obstruction than the tubular sections, which leads to high aerodynamic coefficients and therefore high wind pressure on the structure.

The towers fabricated with tubular sections have shown a higher efficiency compared to the flat-sided sections, the latter widely used on existing models in the country. This is because, not only the beneficial aerodynamic properties of tubular cross-section, but also the inertia properties compared to an angle bar member of the same cross-sectional area. Also in the design phase, tubular towers are often less complicated due to the uniform inertia.

Moreover, the width variation of the cross section tower influences on the stability of the whole structure and individual elements.
2 GUYED MAST TOWER

2.1 Mast geometry and cables

The selected model for the study is the MAR 300 tower, which is used as a guyed mast for radio broadcasts. It is distributed throughout the national territory and it is designed and built in Cuba, which defines its widespread use.

A large percentage of international research recommends the three-dimensional modeling of the tower and the discarding of the equivalent beam model in the representation of the space truss, due to diagonal and horizontal members’ contribution to the axial and bending stiffness. Each member (columns, diagonals and horizontals) forming the three-legged latticed mast was modelled as two-node three-dimensional truss elements with 3 degrees-of-freedom at each node.

Cables are distributed in five levels of a total height of 90m (Figure 1), spaced at 120° in three radial directions. The steel mast section is an equilateral triangle (1.02m), except for the first 3m where the section is reduced to become a single point of support (Figure 2 a). Cable diameters are 9mm and 13mm, with a 1X7+0 configuration. Cables were modeled using an elastic catenary formulation that represents its behavior under loads imposed: self-weight load and prestressing load. For the initial deformed shape, a force was imposed equal to 10% of the rupture load. The cables have a nonlinear relationship because of the absence of direct correlation between force and displacement.

Figure 1. Geometry of MAR 300 tower. Elevation. Units: mm.

Figure 2. Geometry of MAR 300 tower. a) 3D view of the tower with the first atypical section. b) Typical face of the tower. Units: mm.
The elements that conform the mast are, in all cases, flat-sided sections. Specific dimensions are given in Table 1 and the cable levels in Table 2 and Figure 3.

### Table 1. Section properties

<table>
<thead>
<tr>
<th></th>
<th>Columns</th>
<th>Diagonals</th>
<th>Horizontals</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>L 75X8</td>
<td>L 32X4</td>
<td>L 40X4</td>
</tr>
</tbody>
</table>

### Table 2. Cable levels

<table>
<thead>
<tr>
<th>Level</th>
<th>Height (H) [m]</th>
<th>Anchor distance (L) [m]</th>
<th>Diameter (D) [m]</th>
<th>Angle with horizontal (\theta) [°]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17</td>
<td>42</td>
<td>0.009</td>
<td>22.04</td>
</tr>
<tr>
<td>2</td>
<td>33</td>
<td>42</td>
<td>0.009</td>
<td>38.16</td>
</tr>
<tr>
<td>3</td>
<td>50</td>
<td>42</td>
<td>0.009</td>
<td>49.97</td>
</tr>
<tr>
<td>4</td>
<td>68</td>
<td>82</td>
<td>0.013</td>
<td>39.67</td>
</tr>
<tr>
<td>5</td>
<td>86</td>
<td>82</td>
<td>0.013</td>
<td>46.36</td>
</tr>
</tbody>
</table>

![Figure 3. Geometry of MAR 300 tower. Plan view of cable distribution.]

#### 2.2 Support

Support conditions are given for ground anchorages of the cables and the shaft support. In all cases, articulated joints are considered.

#### 2.3 Material

A linear elastic material and constant properties in time was considered. Data of materials used for the shaft elements and cables are given below:

### Table 3. Steel material information (units: kN, m, C)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Mast</th>
<th>Cable</th>
</tr>
</thead>
<tbody>
<tr>
<td>Weight per unit of volume</td>
<td>76.9729</td>
<td>76.9729</td>
</tr>
<tr>
<td>Modulus of elasticity (E)</td>
<td>(1.999 \times 10^8)</td>
<td>(1.965 \times 10^8)</td>
</tr>
<tr>
<td>Poisson coefficient (\nu)</td>
<td>0.3</td>
<td>0</td>
</tr>
<tr>
<td>Coefficient of thermal expansion (A)</td>
<td>(1.17 \times 10^{-5})</td>
<td>(1.17 \times 10^{-5})</td>
</tr>
<tr>
<td>Yield strength (F_y)</td>
<td>248211.28 (250 MPa)</td>
<td>1689905.2 (1600 MPa)</td>
</tr>
<tr>
<td>Rupture strength (F_u)</td>
<td>399896 (400 MPa)</td>
<td>1861584.6 (2000 MPa)</td>
</tr>
</tbody>
</table>
3 LOADS

3.1 Wind load. Historical development.

The study and calculation of towers has changed over the investigations and the growing popularity of computational tools that help resolve mathematical equations. The analysis, required multiple simplifications in the past due to its complexity, today is done with greater precision, corresponding the model to reality and consequently leading to an economically better design.

Wind studies focused on three ways to determine this load: by wind tunnel simulation, by using static approach and the dynamic methods, based on the time-dependent equation of the wind. It is noteworthy that the latter alternative is relatively recent compared to the others.

Theoretical studies applied finite element models for the analysis of guyed towers [7], [8] and insulated cables [9] under the effect of wind and earthquake.

Several researchers have studied the gust factor to simulate the fluctuating nature of wind. In 1967, Davenport [10] took the first steps in determining a gust factor to represent the fluctuating component of the wind. At first, all researchers agreed with the use of a gust factor to take into account the increased of wind pressure, but more recent studies argue for limiting its use for towers up to 150m. In the case of a taller structure, the use of a specific dynamic method or the process developed by Davenport and Sparling called Patch Load is recommended. This method is a simplification to determine the dynamic response of the tower using successive overlapping patches to obtain the final effect of wind loads.

Based on this, the research is focused on comparing the results obtained by the gust factor, the Patch Load method and existing theoretical models so far [11] [12].

The Cuban Wind Standard [2] allows an acceptable design of short self-supporting towers, however tall guyed mast have characteristics that involve the use of a special criteria. To do this, many countries have developed specific standards for calculating towers or exclusively developed sections to deal with these cases.

In Cuba, a deficiency appears in the absence of a specific standard or chapter for towers that allows a structural analysis based on the theory of second-order (geometric nonlinearity) and the effects on the shaft and cables due to the lateral loads.

3.2 Loads considerations.

Self-weight load and wind load were considered in the tower analysis. In case of self-weight load in this research, it was considered only the weight of structural elements, and accessory items were not considered.

The structure, for wind load, was calculated with the current Cuban Proceedings NC 285: 2003 [2]. Furthermore, the Patch Load method is used to obtain the dynamic response of the tower under extreme wind, according to British Standard Procedures [3]. This method was introduced in 1981 by IASS (International Association for Shell and Spatial Structures) publications [13]. It was recommended for the design and analysis of guyed masts and perfected later in investigations carried out by Davenport and Gerstoft [7] and Sparling [8]. The method uses a series of static load patterns to approximate the effects of turbulent wind [9]. For a number of different existing guyed masts, comparisons have been made with results from a full-dynamic analysis with reasonable agreement [4].

To analyze the structure, a finite element method program is utilized. This program is suitable for modeling guyed towers characteristics, allowing define the cable as a catenary formulation and a nonlinear analysis is performed. Three wind directions were analyzed: 0°, 60° and 90° (Figure 4), recommended by British Standard [3].

![Figure 4. Wind directions. Plan view.](image-url)

The dynamic analysis provided in the Cuban Standard (NC 285:2003) is used when the oscillation period is greater than 1 second \( (f \leq 1 \text{ Hz}) \). In the case of the tower studied, the period \( T = 0.54 \text{ sec} \), therefore this case does not require dynamic analysis.

The Cuban Standard establishes the equation that static wind component is calculated:

\[
q = q_{10} \cdot C_t \cdot C_s \cdot C_h \cdot C_r \cdot C_{ra} \cdot C_f \quad [\text{kN/m}^2]
\]  

(1)

Where:

- \( q: \) Wind static pressure component.
- \( q_{10}: \) Basic wind pressure depending on the region of the country where the tower is located, \( q_{10} = 1.3 \text{ Kn/m}^2 \) (zona 1).
- \( C_t: \) Recurrence period, adopted \( C_t = 1 \) for 50 years.
- \( C_s: \) Site coefficient, adopted \( C_s = 1.10 \) corresponding to exposed site. In our country, it is very common that the towers are placed in exposed sites.
- \( C_h: \) Height coefficient.
- \( C_r: \) Gust factor. The fluctuating nature of wind and its interaction with the structures is taken into account through the gust factor depending on the height of the construction and the terrain type. For a height of 90 m and A terrain type, the \( C_r = 1.06 \).
- \( C_{ra}: \) Coefficient of area reduction. In the case of lattice towers, the exposed areas are relatively small, so it takes \( C_{ra} = 1 \).
- \( C_f: \) Aerodynamic coefficient either drag coefficient. This value takes into account the spatial characteristics of the bodies exposed to the wind, i.e. the coefficient involving the aerodynamic nature of the structure. For spatial lattice towers, this coefficient becomes \( C_{fe} \) (spatial), which is calculated by the following expression:

\[
C_{fe} = C_f(1 + N)
\]  

(3)

Where \( C_f = 1.9 \), for the case of flat-sided section. The value of \( N \) is set to the dependence \( A_{projected}/A_{total} \) and \( b/h \) relationship i.e. the dimensions. A quadruple interpolation was used to pinpoint the value of \( N \).

The cables load is considered uniformly distributed over the entire length. Equation 4 describes the wind load on the cables:

\[
q_{cables} = q_{10} \cdot C_t \cdot C_s \cdot C_h \cdot C_r \cdot C_{ra} \cdot C_{fd} \cdot D \quad [\text{kN/m}]
\]  

(4)

In which \( D \) is the corresponding cable diameter. The coefficients remain the same values described above except that the shape coefficient, defined by:

\[
C_{fd} = C_f \cdot \text{sen}^3 \alpha
\]  

(5)

The \( C_f \) coefficient is taken as equal to 1.2. The value of \( \alpha \) corresponds to the angle between the wind vector and the cable.

3.4 Wind load. Patch Load.

Load Patch is a simplified method that simulates the dynamic response of the tower using an equivalent static method. This method is described by British Standard [3] and is only applicable for the case of guyed towers.

The analysis is divided into two states, the first where the mean component of the wind is calculated and the latter when load patches are superimposed to simulate the fluctuating component.

For the application of this method, three conditions must be met:

1. Cantilever height at the top of the mast must be less than the half distance between the last two cables levels. MAR 300, is a radio tower, and not present a cantilever at the top of the structure.

2. The relationship between the shaft bending stiffness and lateral stiffness of the cables, defined as \( \beta_s \), must be less than unity.

The \( \beta_s \) parameter is given by:

\[
\beta_s = \frac{4E_m I_m / h_t^2}{(1/N_t) \sum_{i=1}^{N_t} S_G_i R_G_i}
\]  

(6)

\( E_m \): Equivalent elastic modulus of the shaft.
\( I_m \): Equivalent inertia of the shaft.

The equivalent inertia and the equivalent elastic modulus was calculated according to Kalha [14] (Figure 5), because the shaft of the tower is fabricated with elements of different cross sections.
\[ E_m \cdot I_m = \frac{Ea^2}{2} \left[ A_c + \frac{A_d A_d \cos^3 \theta}{2(A_s + 2A_d \sin \theta)} \right] \]  

(7)

Where:

- \( E \): Elasticity modulus of the material, \( E = 1.999 \times 10^8 \text{kN/m}^2 \).
- \( a \): Width of the shaft cross section, \( a = 1.02\text{m} \).
- \( A_c \): Column cross-sectional area, \( A_c = 1.136 \times 10^{-3} \text{m}^2 \).
- \( A_s \): Twice the horizontal cross-sectional area, \( A_s = 6.08 \times 10^{-4} \text{m}^2 \).
- \( A_d \): Diagonal cross-sectional area, \( A_d = 2.4 \times 10^{-4} \text{m}^2 \).
- \( \theta \): Angle formed between the diagonal and the column, \( \theta = 45.567^\circ \).

\[ \therefore E_m = 1.999 \times 10^8 \text{kN/m}^2, \quad I_m = 6.0728 \times 10^{-4} \text{m}^4. \]

Figure 5. Tower spatial representation for the calculation of the equivalent inertia and equivalent elasticity modulus, according to Kalha [14].

- \( h_s \): Mean distance between levels of cables, \( h_s = 17.25\text{m} \)
- \( N_l \): Number of levels of cables, \( N_l = 5 \).
- \( H_{Gi} \): Height from the base of the tower to the level \( i \) of the corresponding cable.

The parameter \( K_{Gi} \) is:

\[ K_{Gi} = 0.5 \cdot N_l \cdot A_{Gi} \cdot E_{Gi} \cdot \cos^2 \alpha_{Gi} / l_{Gi} \]  

(8)

Where:

- \( N_l \): Number of cables at level \( i \).
- \( A_{Gi} \): Cable cross-area at level \( i \).
- \( E_{Gi} \): Modulus of elasticity of the cable at level \( i \).
- \( \alpha_{Gi} \): Tilt angle of the cable with the horizontal at level \( i \).
- \( l_{Gi} \): Cable length at level \( i \).

Table 4 and 5 summarizes the calculation of \( K_{Gi} \):

<table>
<thead>
<tr>
<th>Cables Level</th>
<th>Height ( H_{Gi} ) [m]</th>
<th>Anchor distance ( L ) [m]</th>
<th>( N_l )</th>
<th>( A_{Gi} ) [m²]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>17</td>
<td>42</td>
<td>3</td>
<td>6.362\times10^{-5}</td>
</tr>
<tr>
<td>2</td>
<td>33</td>
<td>42</td>
<td>3</td>
<td>6.362\times10^{-5}</td>
</tr>
<tr>
<td>3</td>
<td>50</td>
<td>42</td>
<td>3</td>
<td>6.362\times10^{-5}</td>
</tr>
<tr>
<td>4</td>
<td>68</td>
<td>82</td>
<td>3</td>
<td>1.327\times10^{-4}</td>
</tr>
<tr>
<td>5</td>
<td>86</td>
<td>82</td>
<td>3</td>
<td>1.327\times10^{-4}</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Cables Level</th>
<th>( E_{Gi} ) [kN/m²]</th>
<th>( l_{Gi} ) [m]</th>
<th>( \alpha_{Gi} ) [°]</th>
<th>( K_{Gi} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1.965\times10^8</td>
<td>44.7650</td>
<td>22.0362</td>
<td>359.930</td>
</tr>
<tr>
<td>2</td>
<td>1.965\times10^8</td>
<td>52.9517</td>
<td>38.1572</td>
<td>218.961</td>
</tr>
<tr>
<td>3</td>
<td>1.965\times10^8</td>
<td>64.9221</td>
<td>49.9697</td>
<td>119.495</td>
</tr>
<tr>
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<td>1.965\times10^8</td>
<td>106.0740</td>
<td>39.6678</td>
<td>218.485</td>
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<tr>
<td>5</td>
<td>1.965\times10^8</td>
<td>118.4220</td>
<td>46.3639</td>
<td>157.284</td>
</tr>
</tbody>
</table>

The obtained value \( \beta_s = 0.1711 \) validates the use of the Patch load method, since it is less than 1.
The inertial resistance parameter $Q$ must be less than 1. This value takes into account the relationship between inertial forces and damping properties on the tower shaft:

$$Q = \frac{1}{30} \sqrt{\frac{H V_H}{b a V}} \left( \frac{m a V}{H R w} \right)$$

Where:

- $H$: Shaft height including the cantilever (m), $H = 90 m$.
- $V_H$: Average wind speed for one hour interval, at the top of the shaft (m/s), $V_H = 55 m/s$
- $b a V$: Average width of the shaft (m), $b a V = 1.02 m$
- $m a V$: Average mass per length unit of the shaft including the accessory elements (kg/m), $m a V = 5 kg/m$.
- $\sum R w$: Average value of net areas and aerodynamic coefficients for the shaft and accessories (m$^2$/m), $\sum R_w = 0.82$.

The value of parameter $Q$ is 0.147.

The design peak response $r$ may be expressed as:

$$r = \bar{r} \pm \hat{r}_{PL}$$

Where:

- $\bar{r}$: Mean response component.
- $\hat{r}_{PL}$: Peak fluctuating response component.

The mean component was estimated by the coefficients established by the Cuban Standard, as discussed in section 3.3. For the Patch Load method, the basic pressure corresponding to the mean speed interval of an hour is established, so that, the value taken from Cuban Standard was 0.95 kN/m$^2$, and the gust coefficient was not considered.

For the mean component a nonlinear analysis is performed while the patches of the fluctuating component were linearly applied to the structure.

The fluctuating response $\hat{r}_{PL}$ was obtained as a series of static analysis for different stages of loading, taking in consideration the stiffness properties of the system calculated at the equilibrium position. These static load patterns were applied to the mast in succession, with the position to the scheme on figure 6:

![Figure 6. Scheme applying the load patches, according to [3].](image)

These patches are combined as the root sum of the squares:

$$r_{PL} = \sqrt{\sum_{i=1}^{n} r_{PLi}^2}$$

Where:

- $r_{PL}$: Resultant patch load response.
- $r_{PLi}$: Response due to the patch load $i$.
- $n$: Total number of patch loads.

The total fluctuating wind load component is determined by the equation:
\[
\hat{r}_{PL} = r_{PL} \cdot \lambda_B \cdot \lambda_R \cdot \lambda_{TL} \cdot g
\]  
(12)

Where:

\(r_{PL}\): Resultant patch load response.

\(\lambda_B\): The background-scaling factor.

\(\lambda_R\): The resonant magnification factor.

\(\lambda_{TL}\): Turbulent length scale factor.

\(g\): Statistical peak factor.

Using conservative values (\(\lambda_B = 0.75\), \(\lambda_R = 1.2\), \(\lambda_{TL} = 1.05\) and \(g = 4\)), equation 12 reduces to:

\[
\hat{r}_{PL} = 3.78 \cdot r_{PL}
\]  
(13)

4  NONLINEAL ANALYSIS STATES

The analysis is performed in two steps: the Initial State and the Final State. Because of the inherent behavior of tall-guyed towers, geometric non-linearity is included in analysis procedure. In the Initial State, the self-weight loads in addition to the initial prestressing in cables, were applied and the equilibrium was achieved. Then, in the Final State, wind loads were applied. In this step, geometric non-linearity resulting from the difference between the stiffness matrix of the reference structure and the deformed structure under a load increment, was considered.

In the Final State, a static response (NC 285:2003) and a dynamic response (Patch Load) is performed for each wind direction.

5  CASE STUDIES

The first modification proposed, was the replacement of the columns flat-sided section in the typical tower by tubular elements (Figure 7). To this, an equivalent angle-section to circular section is required, not involving modifications in the weight of the tower. The tubular section chosen for the legs, after a trial and error process, corresponds to HSS3X216 (AISC catalog), which has a diameter \(D=0.0762\) m and a thickness \(t=0.005486\) m.

Figure 8 describes the second change performed in the width shaft of the tower, from the typical dimension of 1.02 m. Proposed widths were 1.42 m and 0.8 m to increase and decrease the original design.

![Figure 7. Change proposed in tower geometry](image)

![Figure 8. Change proposed in width of the cross section mast, units: mm.](image)


6 RESULTS

For comparison purposes, the most stressed members were analyzed. The results in the legs, diagonals, horizontal members, cables, anchors reactions and base tower reactions were studied. Finite element model (FEM) analysis was performed using SAP 2000 v.12 in order to obtain the internal forces for each case. For the processing of the results, the shaft is divided into columns A, B and C. The horizontals and the diagonals were divided according to the faces of the tower in AB, BC and CA. The naming of the cables consider the corresponding column and horizontal distance between column and anchorage point.

Columns, diagonals, horizontals and cables are subjected to axial forces, compression or tension. Moment values, shear and torsion are very small compared to the resulting axial forces, so they were negligible in the analysis.

The base of the tower and anchor of the cables were processed to take into account differences in models in the maximum design values of the corresponding foundation. The resulting force is divided into three axes “X”, “Y” and “Z”, corresponding the “Z” to the vertical component and is the most important to the overall analysis.

6.1 Comparison of methods for calculating the wind load

The results of the internal forces in the elements of the tower presented significant differences for the two proposed methods: by the Cuban Standard (NC 285:2003) and the Patch Load calculation in the 0° direction.

In Figure 9, for example, the increase in maximum axial force in each column is analyzed. An increase of 40% in these maximum axial forces is observed in Column A and B, while the increase is 50% in Column C.

Figure 9. Maximum axial force in columns according to the Cuban Standard NC 285:2003 and the Patch Load method.

In short, for all elements of the tower and the support reactions, increases occur in the axial force resulting from the application of Patch Load method compared with the methodology provided by the Cuban Standard, in the 0° direction.

6.2 Structural behavior of the typical MAR 300

The columns of the tower are subjected to axial compression forces. The maximum value is recorded at a height of 57m, corresponding to the Column C. For columns at vertex A, the maximum stress occurs at 16m, while the value for Column B is recorded at 59m.

For the A and C columns, the most unfavorable state is the 90° direction by the Patch Load method. For the same procedure, the worst-case direction is found to be 60° for Column B.

If maximum compression forces of the columns for the most unfavorable condition (Figure 10a) are compared, it is seen that Column C suffers the highest loading, except in the section between 75 and 87m where higher values correspond to the Column B.

The horizontal members are working mainly in tension for the critical condition. The variation of internal forces occurs in at the heights where the cables are connected (Figure 10b). In these zones, axial values increase significantly. This requires, in many cases, more resistance to meet the entire height of the tower design. That is why in some structures, different sections of horizontal members are positioned, with higher resistance in the well-defined zones by the presence of cables, allowing an economic saving. This is not the case being analyzed, which has the same cross-section area throughout its height.

Cable levels were established in the tower at 17, 33, 50, 68 and 86m and for each of these heights, the horizontals show a resulting increase in axial force. This phenomenon occurs because the tension from the cables, which at the Final State involves the horizontal as a transversal stabilizing element.

Furthermore, at a height of 3m, the value of the axial force increases in horizontals, as a consequence of the change in the width of the shaft (figure 10b). The maximum value of axial force for all the horizontals is located at the height of 68m.
members, the most unfavorable working condition occurs for the $90^\circ$ wind direction. For horizontals AB and BC, the peak axial force is found at 68m, while for CA, this happens at 3m from the ground.

The diagonals are subjected to compression and tensions forces. However, compressive values are dominating the design of these elements and for which the diagonals show a more vulnerable state.

The position of the cable levels in the tower determines an increase in the axial force of the diagonals. Similarly, to what occurs in the horizontals, the diagonal members of the structure are affected by the vibratory movements of the cable due to their stabilizing function.

The horizontals are only affected on a well-defined point at the height of each cable level. By contrast, the axial forces on the diagonals increase over a larger height, which comprises of the elements found in the upper and lower sections of this level.

If the diagonals of the three faces of the tower are compared (Figure 11a), it is observed that no group has a noticeable predominance over another, as was the case with the columns.

The cables that hold the shaft of the tower are subjected to axial tension force. This value increases with height to the point where it is critical at 68m and then, begins to decline again (Figure 11b).

The internal forces in the B cable dominate over all the others, and making this element the most critical of the entire structure (Figure 11b).
6.3 Structural revision of the typical MAR 300

The structural revision of the typical tower was based on maximum internal axial forces of the shaft and cables and compared with the corresponding resistant capabilities. The standard used for the revision was the LRFD-99 [15].

Wind loads calculated for extreme conditions generate many failures in structural elements. A failure occurs when the acting axial force (demand) is greater the resistant axial force (capacity).

Table 6 and Figure 12 summarize the quantity of failure elements in each analysis. In all cases, the diagonals and the columns were the most vulnerable elements in the shaft. The horizontals, meanwhile, did not exceed its ultimate resistance. The table also gives the weight and length of the elements, providing a measure of the economic cost involved in the analysis.

Table 6. Failure elements

<table>
<thead>
<tr>
<th>Case</th>
<th>Quantity</th>
<th>Total weight [kN]</th>
<th>Disaggregated by elements</th>
<th>Total length [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>NC 285:2003</td>
<td>127</td>
<td>5.71</td>
<td>126 Diagonals</td>
<td>179.98</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1 Column</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>284 Diagonals</td>
<td>403.80</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>80 Columns</td>
<td>80.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>220 Diagonals</td>
<td>314.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>31 Columns</td>
<td>31.00</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>390 Diagonals</td>
<td>522.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>105 Columns</td>
<td>105.00</td>
</tr>
<tr>
<td>Patch Load 0°</td>
<td>364</td>
<td>14.46</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Patch Load 60°</td>
<td>251</td>
<td>8.53</td>
<td>220 Diagonals</td>
<td>314.96</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>31 Columns</td>
<td>31.00</td>
</tr>
<tr>
<td>Patch Load 90°</td>
<td>495</td>
<td>18.83</td>
<td>390 Diagonals</td>
<td>522.18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>105 Columns</td>
<td>105.00</td>
</tr>
</tbody>
</table>

The most critical condition for the tower proved to be the 90° direction, where failures occurred in 495 elements of the array. Figure 12 shows the number of columns, diagonals and horizontals belonging to the tower, and the proportion of these elements that failed the critical condition. In the columns, the number of resistant elements outnumber those that failed. On the diagonals, unlike the columns, the number of elements in failure is much greater than the resistant ones under extreme wind loading. In the horizontals, no losses occurred.

Figure 12. Proportion of failed elements for Patch Load 90°.

Due to the loads imposed on the structure, three failures occurred in the cables that hold the shaft. These cables are located at the second and fourth levels of Column B, which are at heights of 33m and 68m. The other failure is located at the Column A and a height of 33m. Despite this result, the cable is considered to be working at 100% capacity, however, some regulations such as TIA/EIA [16] establish that only 60% of the ultimate load should be considered for greater safety of these elements. Considering this, the amount of cables in failure will increase significantly.

Table 7 shows the maximum axial force registered and the resistance capacity of each element. The demand/capacity ratio is a measure of each member’s performance. A closer relationship to 0 indicates that the element works conveniently and sufficient to withstand an increase in loads. For values close to 1, the element is at the limit capacity and a value exceeded, the structural failure will occur.

From the results shown in Table 7, it is concluded that axial forces exceeding the ultimate strength limit were recorded in all elements leading to failure, except for the horizontal members. In addition, it is observed that in the columns, demand/capacity ratio is greater than the other elements, indicating that that loading has been imposed over its capacity.
Table 7. Demand and capacity of each element tower.

<table>
<thead>
<tr>
<th>Element</th>
<th>Max axial force registered [kN]</th>
<th>Demand</th>
<th>Max resistant axial force [kN]</th>
<th>Capacity</th>
<th>Wind direction</th>
<th>Demand/Capacity ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Cables 9mm</td>
<td>82.16</td>
<td>69.00</td>
<td></td>
<td>90°</td>
<td>1.19</td>
<td></td>
</tr>
<tr>
<td>Cables 13mm</td>
<td>133.23</td>
<td></td>
<td>120.00</td>
<td>90°</td>
<td>1.11</td>
<td></td>
</tr>
<tr>
<td>Columns</td>
<td>-299.09</td>
<td>-199.04</td>
<td></td>
<td>90°</td>
<td>1.50</td>
<td></td>
</tr>
<tr>
<td>Horizontals</td>
<td>42.14</td>
<td>67.91</td>
<td></td>
<td>90°</td>
<td>0.62</td>
<td></td>
</tr>
<tr>
<td>Diagonals</td>
<td>-22.28</td>
<td>-18.55</td>
<td></td>
<td>90°</td>
<td>1.20</td>
<td></td>
</tr>
</tbody>
</table>

6.4 Comparison of changes in column geometry in the MAR 300.

Figure 13 shows the results of the wind load on the model with tubular sections in comparison to the original flat-sided model, over the entire height. The behavior shown in the figure is similar for all other wind directions.

![Figure 13. Wind load in models with different section geometry.](image)

In Figure 14, the maximum axial forces obtained for each column are compared for the two different models. In addition, highlighted in red borders are the absolute maximums obtained. In the Column A, the axial force is superior in tubular section model, while for the remaining columns, the axial force decreases for tubular section model.

The increase in percentage terms for Column A is 7%. For Columns B and C, the reduction of the axial force on the tubular sections model is 18% and 24% respectively.

In Column A, the maximum internal force of the tubular section model is produced while this value corresponds to the Column C of the original model. If these two maximum values of both models are compared, the difference is 24%, and shows a better performance in tubular section model which have reduced internal forces.

![Figure 14. Maximum axial force obtained for each column.](image)

Figure 15 shows the difference of the axial forces in horizontals of the different faces of the models studied. Can be observed that having columns with tubular section decreases the values obtained in axial forces. Decreases in percentages are respectively 42%, 3% and 21% for each side of the tower AB, BC and CA. The maximum the axial force was recorded for the typical model.
in the AB horizontals. In the model of tubular sections, contrary to the above, the maximum value is located in the BC horizontals. If these two peaks of forces are compared, the resulting difference is 10%.

Figure 15. Maximum axial force obtained for each horizontals.

In Figure 16, the maximum axial forces for the diagonals of the different faces of the models are represented. In all the diagonals can be seen a decrease in compression that are subjected for the case of tubular sections columns. Decreases in percentage terms are 4%, 18% and 27% respectively for the AB, BC and AC sides.

Similar to what happens with the structural elements discussed above, the greater axial force is not produced in the diagonals of one face in both towers. For the model of flat-sided sections, the maximum demand is registered in the CA diagonal and for the tubular sections tower; this value occurs on AB face. The difference established by these two peaks is 11%.

Figure 16. Maximum axial force obtained for each diagonals.

In short, switching to a tubular section in columns significantly improves the maximum axial forces in all elements of the structure. In the reactions of the tower and cables decreasing the vertical force occurs in all supports, except on the C vertex, where is not significant to the corresponding foundation design.

6.5 Comparison of changes in width shaft in the MAR 300.

Figure 17 shows a comparison of wind load along the tower height, in the three models for width variation of the cross section shaft. An increased width of the cross section shaft determines a higher value of the resulting wind load. The referred behavior is similar in all wind directions.

Figure 17. Wind load in models with change in width of the cross section mast
The maximum axial force recorded for each element varies after change in the width of the shaft. The 1.42m model aggravates the compression (Figure 18), in each column of the tower. The 0.8m model, unlike the first, registers a decrease in the resultant axial force in columns. It is therefore an advantage to decrease the width of the cross section shaft.

The maximum axial forces registered in the typical model and 0.8m model correspond to C columns, while for the 1.42m model, the maximum compression is located in B column. If we compare these three peaks, it is appreciated that the 0.8m model decreases the axial force 9% and the 1.42m model increases 7% the axial force, respect to typical model.

Figure 18. Maximum axial force obtained for each columns.

In the horizontals, higher values of axial force produced with an increased width of the cross section of shaft (Figure 19).

The 0.8m model registered a decrease in the values of the maximum axial force in the members of the BC and CA sides. In AB horizontals is observed a slight increase in this value, but is negligible because it's minor than 1%.

The maximum traction was recorded in the AB elements for widths of 1.02m and 0.8m, while for the 1.42m model, it is located at the BC face. If these maximum axial forces in each model are compared, the behavior is unfavorable in both cases relative to the typical model. Increases in percentage terms are 7% for 1.42 m model.

Figure 19. Maximum axial force obtained for each horizontals.

In Figure 20, it can be observed that an increase in width (1.42m) is detrimental to the diagonals of the AB face where the compression is more meaningful. For the remaining faces occurs a decrease in this value over the typical model.

For 0.8m model, the axial forces are increased for each of the faces of the tower in relation to the typical model.

In short, both increased as decrease of the width of the shaft were harmful to the diagonals of the tower, because the maximum values of internal forces increase over the original model. It is noteworthy that these increases are not so significant, considering the values of axial forces. This represents increments between 10 to 11%.
In general, it is concluded that the 1.42m model aggravates the structural behavior of the elements of the tower, because greater axial forces are observed on cables, columns, diagonals and horizontals.

The 0.8m model, in turn, has advantages over the original model in cables and columns elements of the tower. For horizontals and diagonals, increases occur in the maximum axial force.

For the support reactions, increased to 1.42m the width of the shaft aggravates the vertical force generated on the structure base and cables anchors. By contrast, the decrease corresponding to 0.8m model produces lower values of the vertical force in each of the analyzed points and consequently a decrease in the design required to the foundation.

7 CONCLUSIONS

1. The structural revision of the MAR 300 tower indicated that, regardless of the method used for calculating wind load, either the Cuban Proceedings (NC285:2003) or the Patch Load, the tower does not resist the regulation set speed for the Cuba western region.

2. Changing legs to tubular section improves the performance of the MAR 300 tower. The values decrease in axial forces in all the elements of the shaft, the cables and the support reactions. The percentages for each element forces are:
   - 24% in legs
   - 10% in horizontal elements
   - 11% in diagonals
   - 15-18% in cables
   - 22% in the support reactions at the base
   - 11-22% in the anchors of cables

3. Changes in the width of the cross section shaft produce significant changes in the structural behavior of the MAR 300 tower, reflected as follows:
   - The increased in the width of the cross section of the shaft not improve the structural behavior of the tower. Increases occurred both in the interior forces of the elements of the tower and the support reactions.
   - Decreasing the cross section of the shaft produces lower axial forces in the cables in both diameters and legs. In horizontal members there were no significant changes and for the diagonals occurs an increase in the compression axial forces.

4. From the comparison of the Patch Load method with the static method of the Cuban Standard, it is concluded that the internal forces increase in all elements of the implementation of the first method.

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