ON THE DYNAMIC RESPONSE OF A 190M-HIGH TRANSMISSION LINE STEEL TOWER SUBJECTED TO CABLE RUPTURE

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***Abstract**. The design of a double circuit 500kV transmission line (TL) in the brazilian Amazon region is presently in progress. In addition to the length (over 1400 km) within the rain forest, the design has to cope with large river crossings and very severe environmental constrains. Some of these crossings demand single spans exceeding 2000 meters. The crossing of the Trombetas River is one of the most important, having a total length of more than 5100 meters. The proposed design for crossing the river has located a suspension tower on an island. Each one of the two main spans is approximately 1900m long and demands two 190m high suspension towers and a third 120m high tower. These towers need to be erected over foundations 10m above ground level due to the annual flooding of the river bed. In this context, the present article reports the structural analysis of the special TL towers for this crossing. The focus is on the dynamic response of the 190m-high structure subjected to cable rupture. The entire crossing section is modeled, including the two highest towers and all other elements: foundations, conductor cables, shield wires and insulator strings. The loading resulting from a cable rupture is applied to the system and member responses are computed as a function of the time, by means of explicit numerical integration of the equations of motion. Peak values of the simulated dynamic response are finally compared with responses obtained by standard design methods.*

***Keywords**: dynamic analysis, steel tower, transmission line structures, cable rupture, finite central differences.*

1. INTRODUCTION

The 500kV Transmission Line (TL) from the Tucuruí hydroelectric power plant to Macapá and Manaus, within the brazilian amazonic region, presents great engineering challenges, such as large river crossings and very severe environmental constrains. In addition, during the design stage the available information on the foundation soil and local geology presented large uncertainties. In this context it was decided to resort to towers about 190m high in order to attain spans as long as 2000m. Obviously these structures demanded a detailed assessment, such as the specification of the wind load in a region with scarce meteorological data and the resulting structural response of the towers.

The present paper aims at describing the evaluation of the dynamic response of the preliminary design of the main steel tower for the Trombetas River crossing for cable rupture, which is one of the loading cases considered in design. The studies were carried out through the analysis of an entire section of the transmission line (towers, cables and insulator strings), representing cables and structures by means of truss elements and solving the resulting equations of motion by direct explicit numerical integration. This methodology is programmed through software developed at LDEC/UFRGS – Laboratory of Structural Dynamics and Reliability of Federal University of Rio Grande do Sul, Brazil.

In summary, the paper describes in detail the determination of the dynamic response of tower GTS 01 subjected to cable rupture, *i.e.*, the evolution with time of displacements at the top of the tower and axial forces in structural elements. Peak values are compared with the response obtained through conventional TL design methods.

2. DESCRIPTION OF THE CROSSING AND THE STRUCTURAL SYSTEM

2.1. Crossing over the Trombetas River

The crossing TL over the Trombetas River is of the A-S-S-S-A type, in other words, it is composed of anchor towers at both ends (GTA 00 and GTA 01) and a central section with three suspension towers (GTS 00, GTS 01 and GTS 02), as shown in Fig. 1. The profile of the crossing with the identification of the towers may be seen in Fig.2.

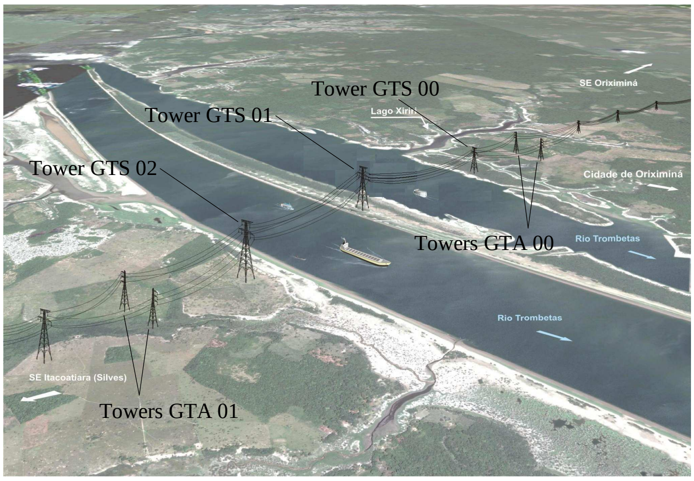

Figure 1. View of the crossing over the Trombetas River.

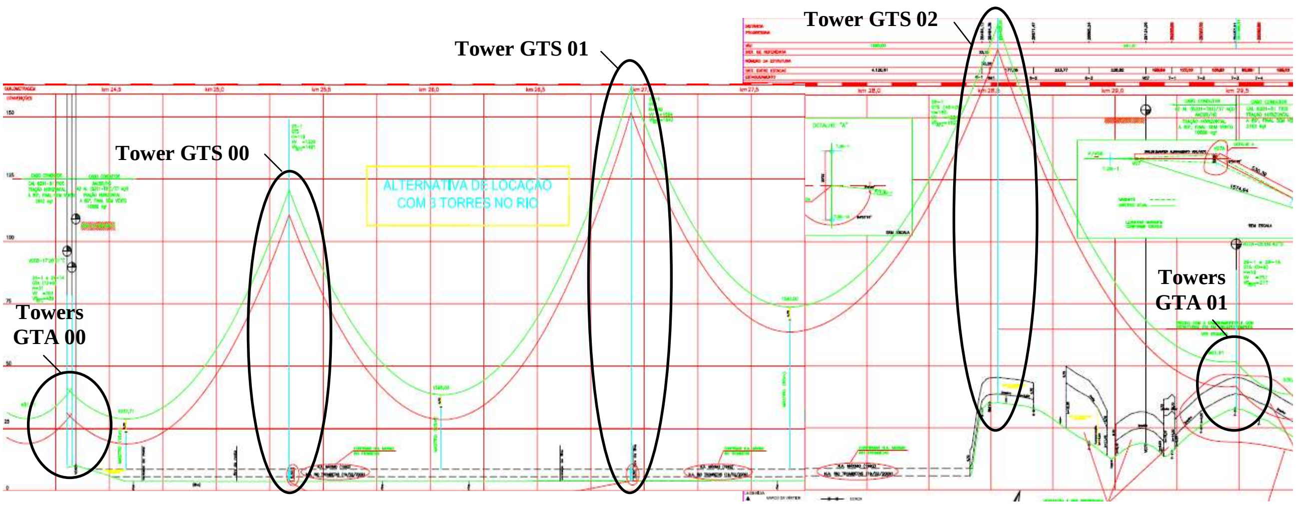

Figure 2. Profile of the crossing over the Trombetas River.

The main spans of the Trombetas River crossing design are 1598m and 1590m long, while the suspension towers should have useful heights equal to 190m and 119m[1]. Two crossing towers (GTS 00 and GTS 01) should have their foundations lifted up about 10m, due to the elevation of the river level during the flooding season.

2.2. General considerations

The mass distribution of a structure plays a fundamental role in its dynamic analysis. Therefore, especial attention was devoted to the correct determination of the masses in the computational model of tower GTS 01. For instance, the masses of main bars are automatically calculated and assigned by the program to the nodes of the model. The mass of secondary bars, which are often introduced just for bracing main bars but do not carry loads in a linear analysis and hence need not be included in the model, were calculated and distributed manually. Additional masses, for example applied loads due to bolts, steel plates, galvanization and equipments, were carefully calculated and lumped at the corresponding nodeal points of the model.

The steel tower GTS 01, was designed to stand on a concrete slab at 10m height above ground level, supported by four concrete tubular section shafts, with 2.50m external diameter and 0.10m thickness. Moreover, the soft soil at the

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[1] Note that in TLs technical literature the height of towers may be referred to ground level or alternatively to the lowest level of tha cables within the span.

site did not allow admitting the usual hypothesis that the structural model is fixed on a rigid base. Penetration tests characterized the soil as extremely soft. In addition, the annual flooding of the area, lasting several months, suggested that the capacity of the soil top layers might periodically decrease to negligible values. Under these conditions, the stiffness of the base of the model was estimated admitting a floating foundation, with the followin values:

Horizontal stiffness in the direction normal to the LT:	$k_x = 2.04 \times 10^6$ N/m;
Horizontal stiffness in the direction of the LT:	$k_z = 2.04 \times 10^6$ N/m;
Vertical stiffness:	$k_y = 2.04 \times 10^7$ N/m.

Regarding structural damping, it is known that energy dissipation in steel lattice towers increases with the vibration amplitude. Limited experimental evidence suggests critical damping ratios around 10% for large response amplitudes (Silva *et al*., 1983). In this paper, the suggested 10% value was adopted.

3. DESIGN PROCEDURES FOR CABLE RUPTURE

Usual design procedures of TL structures consider all acting dynamic loads, such as wind or cable rupture, by means of *equivalent static loads*. Specifically in connection to cable rupture, in usual design practice the load due to cable rupture is applied directly on the tower, in the longitudinal direction of the TL, with an magnitude equal to the residual static load subsequent to the cable failure. For conductor cables, this magnitude is around 80% to 85% of the EDS condition (Every Day Stress).

In the case of the crossing section on the Trombetas River, the conductor cables were designed for a tension equal to 22% of its capacity (UTS - Ultimate Tension Stress). Therefore, the magnitude of the load that should be applied on the GTS 01 tower, in the longitudinal direction, must be around 18% of its UTS, jointly with other relevant loads in the vertical direction due to dead weight of the tower, equipments, conductor cables that did not break and shield wires.

4. SOLUTION METHOD

To perform the dynamic analysis, direct explicit numerical integration of the equations of motion in the time domain was adopted, using the central finite differences scheme, because it does not require assembling or updating the system global stiffness matrix. Integration is accomplished at element level, which constitutes an advantage in non-linear problems. When the system mass and damping matrices **M** and **C** are both diagonal, the method becomes explicit and the expression in central finite differences for the displacement at any node in either the x, y or z direction, at time $t + \Delta t$, may be written as:

$$q(t+\Delta t) = \frac{1}{1 + c_m \Delta t/2} \left[\frac{f(t)\ \Delta t^2}{m} + 2\,q(t) - (1 - c_m \Delta t/2)\, q(t - \Delta t) \right] \tag{1}$$

in which q denotes the nodal coordinate in either the x, y or z direction, $f(t)$ the resultant nodal force component in the corresponding direction at time t, $c_m = c/m$ is a constant, m the nodal mass and c the nodal damping coefficient, assumed proportional to mass m. The resultant nodal force $f(t)$ consists of gravitational forces (dead weight and external nodal forces), and axial forces in the truss elements. It is important to quote that geometrical non-linearity is always considered, since the nodal coordinates are updated after each integration step Δt.

Convergence and accuracy of the solution depend basically on the integration time interval Δt. Since the method is only conditionally stable (Bathe, 1996), it is necessary that $\Delta t \leq \Delta t_{crit}$. For latticed structures, the critical time interval Δt_{crit} can be estimated by (Groehs, 2005):

$$\Delta t \leq \Delta t_{crit} = \frac{L_{min\ (0)}}{\sqrt{E/\rho}} \tag{2}$$

in which $L_{min(0)}$ is the initial length (in t = 0) of the smallest truss element, E is the elastic modulus and ρ is the material mass density. Additional detais about the integration method applied to dynamic analysis of TL towers and cables can be found in Kaminski *et al.* (2005), Miguel *et al.* (2005), Kaminski (2007) and Kaminski *et al.* (2008).

5. MECHANICAL MODEL FOR THE DYNAMIC ANALYSIS

5.1. Description of the mechanical model

To evaluate the dynamic response of the GTS 01 tower subjected to cable rupture, a mechanical model with the entire crossing section over the Trombetas River was modeled, including the two highest towers (GTS 01 and GTS 02), conductor cables, shield wires, insulator strings as well as the foundations. Such model with all elements is presented in Fig. 3.

The insulator strings for each conductor cables bundle in the GTS towers are double, as showed in Fig. 4. The length of all insulator strings in the GTS towers is 7.15m.

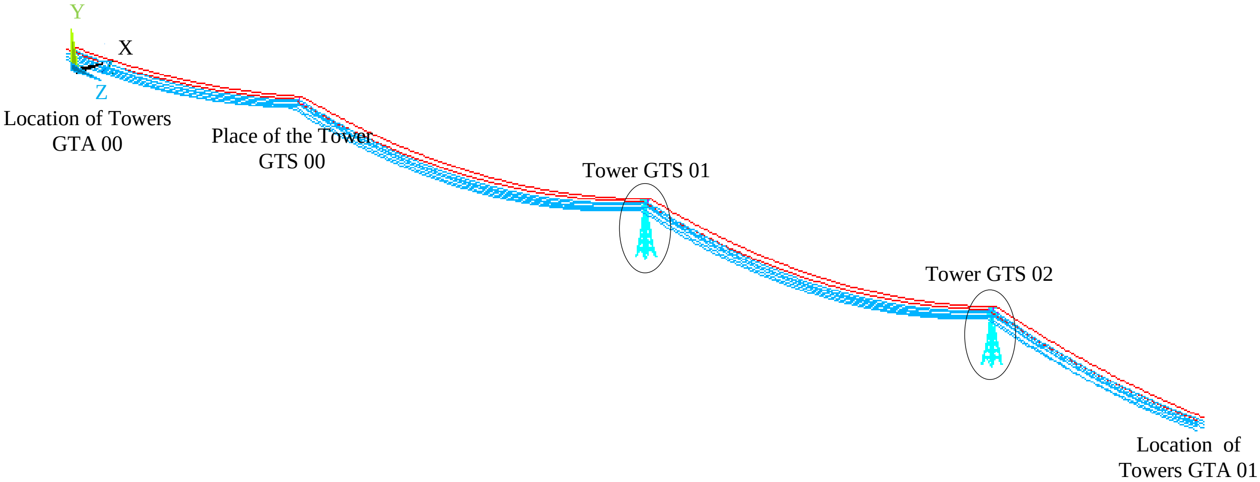

Figure 3. Mechanical model of the crossing section on the Trombetas River.

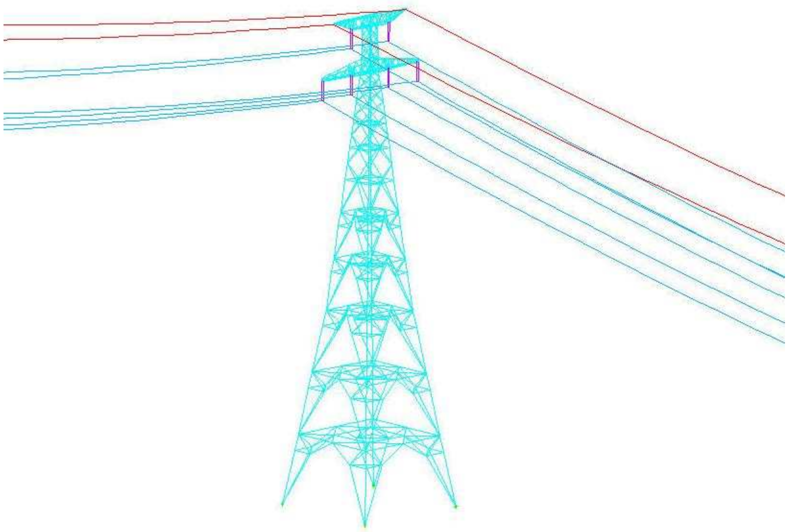

Figure 4. Detail of GTS 01 tower in the mechanical model.

The crossing section on the Trombetas River has a total length exceeding 5100m. The model presents the following spans: 1037.71m between the anchor GTA 00 towers and the GTS 00 suspension tower , 1598.0m between the GTS 00 suspension tower and the GTS 01 suspension tower, 1590.0m between the GTS 01 suspension tower and the GTS 02 suspension tower and finally 961.61m between the GTS 00 suspension tower and the GTA 01 anchor towers . The conductor cables used in the crossing section are bundles with four AACSR 535/240 cables (AACSR - Aluminum Alloy Conductor Steel Reinforced). Each cable has 775.06mm^2 total cross sectional area (aluminum alloy + steel). The shield wires are OPGW type (OPGW - Optical Fiber Composite Overhead Ground Wire) with 349.14mm^2 cross sectional area. Other properties of the conductor cable AACSR 535/240 and of the shield wire OPGW are presented in Table 1 and Table 2, respectively.

Table 1. Properties of the AACSR 535/240 conductor cable.

External diameter of the conductor cable	36.21 mm	0.03621 m
Cross sectional area (aluminum alloy)	535.70 mm^2	535.70×10^{-6} m^2
Cross sectional area (steel)	239.36 mm^2	239.36×10^{-6} m^2
Total cross sectional area (aluminum alloy + steel)	775.06 mm^2	775.06×10^{-6} m^2
Tension capacity of the conductor cable	49950.0 daN	499500 N
Unit weight of the conductor cable	3.464 daN/m	34.64 N/m
Elastic modulus in tension	94.50 daN/mm^2/100	9.45×10^{10} N/m^2

In the mechanical model, the bundles were replaced by a single cable element, with outside diameter, cross section area, tension capacity and unit weight equal to four times the values presented in Tab. 1.

Table 2. Properties of the OPGW shield wire.

External diameter of the shield wire	24.30 mm	0.0243 m
Cross sectional area	349.14 mm^2	349.14×10^{-6} m^2
Tension capacity of the shield wire	39768.76 daN	397687.6 N
Unit weight	2.2563 daN/m	22.563 N/m
Elastic modulus in tension	129.845 daN/mm^2/100	12.9845×10^{10} N/m^2

5.2. Constitutive law of conductor cables and shield wires

Cables are formed by the association of threads, able to carry only tensile forces. In this paper, a linear model is used to calculate cables sags, elongations and tensions, *i.e.*, the cable stress-strain diagram, at constant temperature, is a straight line. The following constitutive law was adopted for conductor cables and shield wires in tension:

$$F_C = E_C\, A_C\, \Delta L_C / L_{OC} \tag{3}$$

in which A_C denotes the cross sectional area of the cable element (m^2), equal for conductor cables to the total area (aluminum alloy + steel) and for shield wires to the steel area; E_C the elastic modulus in tension (N/m^2); F_C the tension force in the cable element (N); ΔL_C the elongation of the element (m) and L_{OC} the unstressed length of the cable element (m).The values used in Eq. (3) to calculate the tension forces in the cable elements are presented in Tables 1 and 2. Suspended cables in TL present the form of a cathenary. In the condition EDS, the conductor cables AACSR 535/240 used in the crossing section were designed for a tension of 22% of its capacity (UTS - Ultimate Tension Stress). The shield wires were designed for maximum sag equal to 90% of the conductor cables maximum sag, resulting in a tension around 20% of the shield wires UTS.

When the suspension points of the cable have the same height, the cathenary is symmetrical in relation to the center of the span (central axis), where the vertex is located, *i.e.* the point where the maximum sag occurs. In the case of supports with different heights, the cathenary is not symmetrical and the maximum sag f_e does not occur at the center of the span, as shown in Fig. 5. The sag depends on the span length, on the temperature and on the tension in the cable when it is fixed at the supports.

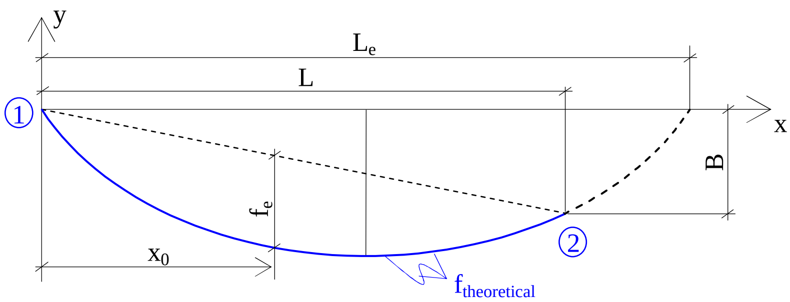

Figure 5. Suspended cable between supports “1” and “2” with different heights ($B \neq 0$).

At the beginning of the analysis (initial condition, t = 0s) the cable should be in a position such that, after the application of dead loads, it is subjected to the design tension force, equivalent to a percentile of the tensile strength of the cable, with the theoretical cathenary ($f_{theoretical}$) and the maximum sag (f_e). The formulation used to determine the theoretical cathenary, the maximum sag, the position of the maximum sag (x_0) and the theoretical length of the cables is described by Kaminski (2007). Additional details are given by Irvine and Caughey (1974).

5.3. Constitutive law of insulator strings

The insulator strings were modeled with elements able to carry only tensile forces. In this paper, a linear model is used to describe the force-displacement behavior of these elements. As mentioned before, all the insulator strings in the GTS towers are double with 7.15m length. The following constitutive law was adopted for insulator strings in tension:

$$F_I = E_I\, A_I\, \Delta L_I / L_{OI} \tag{4}$$

in which A_I denotes the cross section area of the insulator string element (m^2); E_I the elastic modulus in tension of the steel that joins the insulators (N/m^2); F_I the tension force in the insulator string element (N); ΔL_I the elongation of the element (m) and L_{IC} the unstressed length of the element (m). The values used in the Eq. (4) to calculate the tensile forces in the insulator strings are presented in Table 3.

Table 3. Properties of insulator strings.

Cross sectional area of two insulators strings	5.00×10^{-4} m2
Weight for meter of two insulators strings	466.5 N/m
Elastic modulus in tension	2.00×10^{11} N/m^2

5.4. Constitutive law of bars of the towers

Towers GTS 01 and GTS 02 were designed for ASTM A572 steel, with elastic modulus E = 200GPa. The following linear model, both in tension as well as in compression, was adopted to describe the force-displacement behavior of the truss elements:

$$F_B = E_B\, A_B\, \Delta L_B \,/\, L_{OB} \qquad (5)$$

in which A_B denotes the cross sectional area of the truss element (m^2); E_B the elastic modulus of ASTM A572 steel (N/m^2); F_B the tension or compression force in the element (N); ΔL_B the elongation or shortening of the element (m) and L_{OB} the unstressed length of the truss (m).

5.5. Load application

The total duration of the dynamic analysis was limited to 40 s. The dead weight of cables, towers, insulators and additional masses was gradually applied during 5 s, allowing 15 s to damp out induced vibrations. Rupture of the cable occurs 20 s after beginning the integration process. The ensuing 20 s were used for the analysis after rupture. In this period, the evolution with time of axial forces in each truss element and of the displacements at the top of tower GTS 01 were determined, and the maximum values identified. The results are presented in Section 6. It should be underlined that it is assumed that rupture of a conductor *cable bundle* takes place, which implies that the fours cables of the bundle break at the same time. Four analysis of cable rupture were performed, one for rupture of a shield wire and one for each conductor cable bundle of one side of tower GTS 01. The cable elements (conductor cable bundle and shield wire) assumed to break are illustrated in Fig. 6.

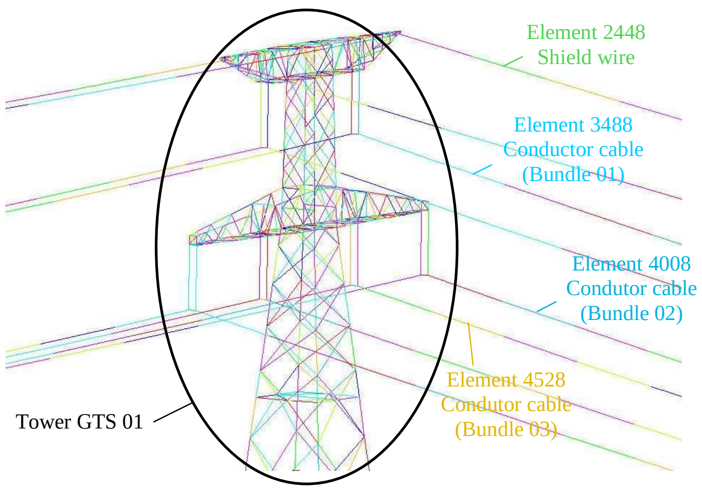

Figure 6. Cable elements assumed to break.

6. RESULTS AND DISCUSSIONS

The dynamic response of tower GTS 01, was obtained using explicit numerical integration. To determine the response envelope, rupture of all bundles was evaluated in sequence. The tower peak response due to cable rupture, by means of equivalent static loads, was also determined using a FEM program. The results are presented below.

6.1. Dynamic response due to rupture of a conductor cable bundle

The variation with time of displacements of four nodes at the top of tower GTS 01, shown in Fig. 11, in the longitudinal direction to TL (axis *z*), due to rupture of conductor cable bundle 01, is shown in Fig. 7. Fig. 8 presents the evolution with time of axial forces in some selected diagonal elements of tower GTS 01, identified in Figs. 10 and 11, due to the rupture of conductor cable bundle 01. Similarly, Fig. 9 shows the axial forces in selected main members of tower GTS 01, also, identified in Figs. 10 and 11, due to rupture of conductor cable bundle 01.

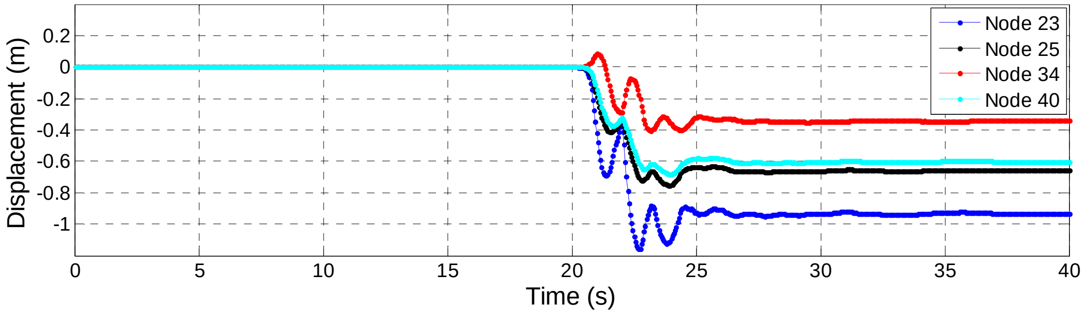

Figure 7. Nodal displacements at top of tower GTS 01, in the direction of the TL, due to the rupture of conductor cable bundle 01.

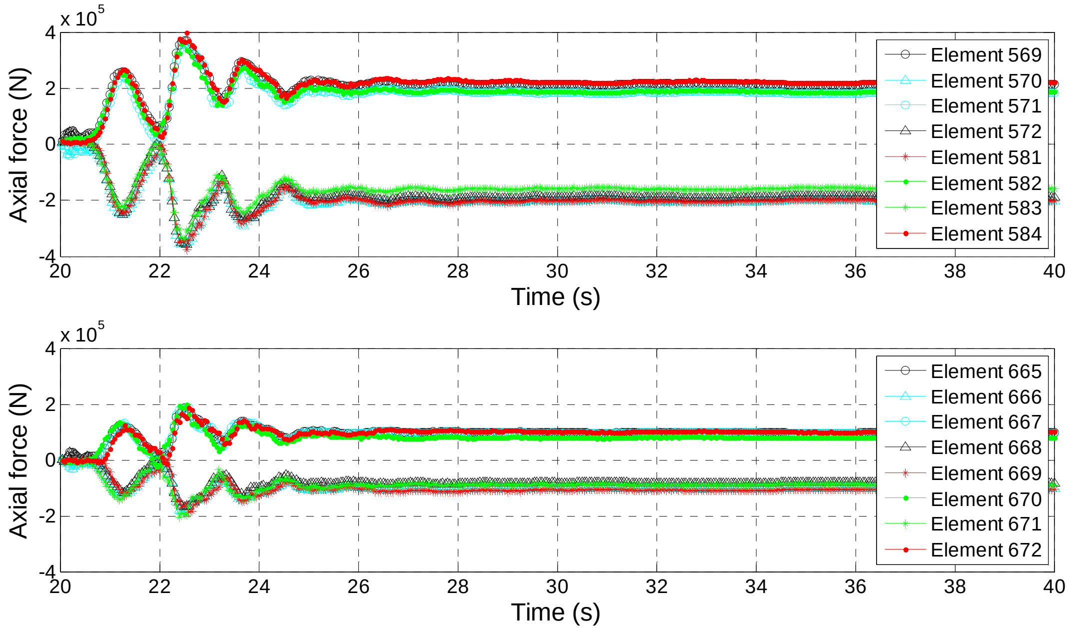

Figure 8. Axial forces in diagonal elements of tower GTS 01, due to rupture of conductor cable bundle 01.

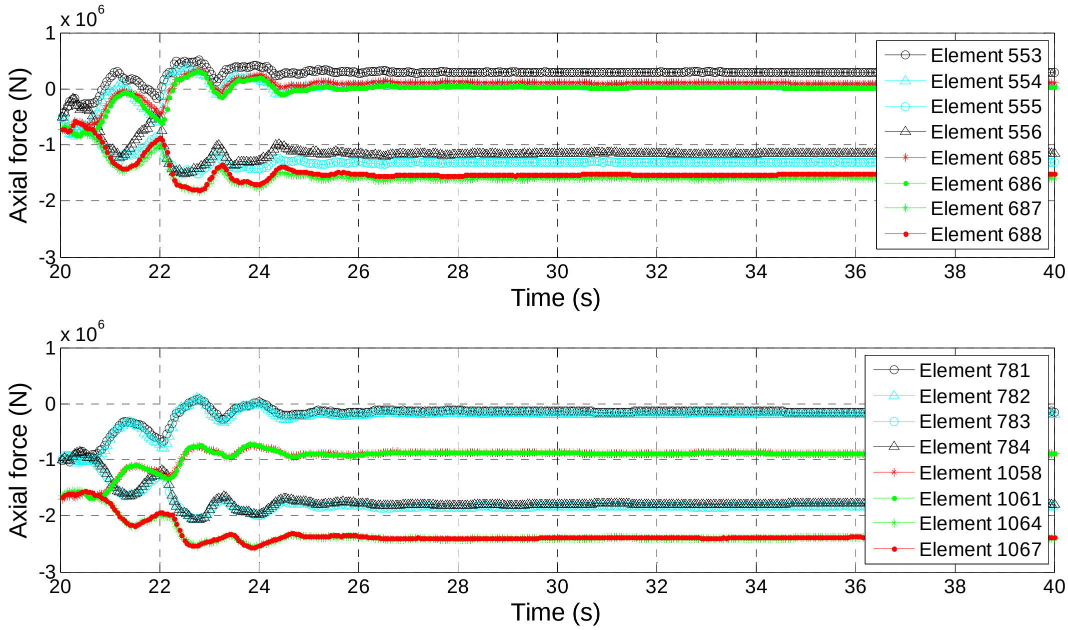

Figure 9. Axial forces in main members of tower GTS 01 due to the rupture of a conductor cable bundle.

Similar results, presenting however smaller amplitudes, were obtained when the rupture of the conductor cable bundles 02, 03 and shield wire were simulated.

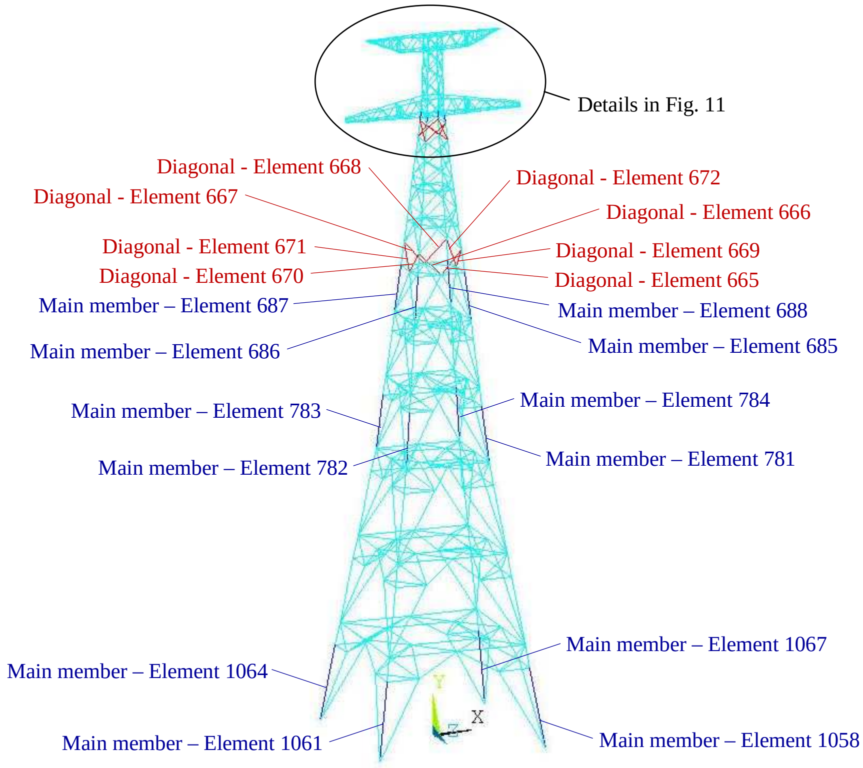

Figure 10. Selected diagonal and main members of the lower part of tower GTS 01.

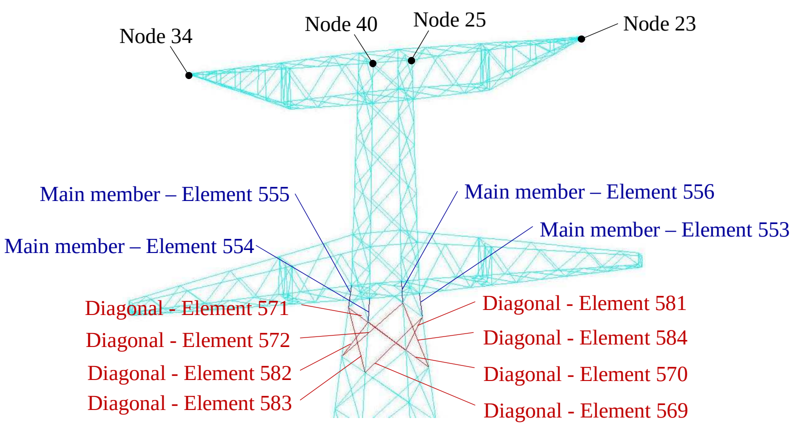

Figure 11. Selected nodes, diagonals and main members of the upper part of tower GTS 01.

6.2. Static response due to rupture of a conductor cable bundle

The nodal displacements at the top of tower GTS 01, identified in Fig. 11, in the direction of the TL (axis *z*), for the standard static analysis of rupture of conductor cable bundle 01, are indicated in Table 5. Table 6 presents the axial forces in selected diagonal and main members of the tower, also identified in Figs. 10 and 11, according to a static analysis of loads due to rupture of conductor cable bundle 01.

Table 5. Displacements (*z* direction) at the top of tower GTS 01 due to rupture of conductor cable bundle 01.

Node	Displacement in the direction of the TL (m)
23	- 0.933
25	- 0.660
34	- 0.350
40	- 0.605

Table 6. Axial force in selected members of tower GTS 01 due to rupture of conductor cable bundle 01.

Main member	Axial force (KN)	Main member	Axial force (KN)	Diagonal member	Axial force (KN)	Diagonal member	Axial force (KN)
553	287.9	781	- 113.5	569	216.6	665	98.4
554	39.7	782	- 159.1	570	- 204.3	666	- 102.3
555	- 1287.9	783	- 1853.8	571	191.5	667	87.9
556	- 1120.6	784	- 1909.5	572	- 183.8	668	- 90.8
685	126.8	1058	- 863.8	581	- 210.3	669	- 111.1
686	49.9	1061	- 865.5.	582	184.3	670	78.2
687	- 1599.0	1064	- 2421.5	583	- 155.8	671	- 88.3
688	- 1531.5	1067	- 2409.9	584	229.1	672	104.5

7. CONCLUSIONS

The paper describes the dynamic analysis of a four spans section of a TL crossing over the Trombetas River, which includes a 190m-high TL steel tower, subjected to cable rupture. The computed dynamic response of tower GTS 01 was then compared with the static response obtained by standard procedures.

Since the latter aim at determining forces and displacements *after rupture has occurrred*, the close correlation of the *final state* in the dynamic analysis with the *standard static* predictions constitutes strong evidence of the robustness of both models. On the other hand, dynamic amplification may approach 50% for main members and significantly exceed that value in case of diagonel members. It is thus concluded that dynamic amplification effects *are not negligible* in TL crossings and may cause failure of the towers if not properly taken into account for design purposes.

8. ACKNOWLEDGEMENTS

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10. RESPONSIBILITY NOTICE

The five authors, Leandro Fleck Fadel Miguel, João Kaminski Junior, Letícia Fleck Fadel Miguel, Jorge Daniel Riera and Ruy Carlos Ramos de Menezes, are the only responsible for the printed material included in this paper.