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NUMERICAL ROUTINE FOR DYNAMIC ANALYSIS OF TRANSMISSION LINES GUYED TOWERS SUBMITTED TO BROKEN CABLE

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Abstract. *In the design and sizing of guyed lattice metallic structures used in transmission lines (TL) as a way of simplification, static analyzes are usually carried, where the dynamic actions are represented by "equivalent static loads". However, the static response to this type of structure is not the most appropriate, since such structures are lightweight, slender and that are always subject to dynamic nature action, except the own weight. Therefore, a dynamic analysis is critical to get a more precise result in terms of efforts on the bars and nodal displacements. In this context, this paper deals with the development of a computational routine in Fortran, for obtaining the dynamic response of guyed metallic towers of TL, submitted to dynamic action of broken cable. For the solution of this problem is used the Direct Integration Method of the equations of motion, explicitly, whit central finite differences. In the output result files, the axial forces in all the bars are displayed along the time and the nodal displacements are recorded at certain intervals of time during the analysis, in order to facilitate the structural behavior viewing with animation in post-processing programs. The accuracy of the dynamic response is evaluated by analyzing the isolated guyed tower, subjected to an impulsive load, comparing the fundamental frequency, calculated on the top of tower displacements obtained in the time domain, at the same frequency determined by an modal analysis. Knowing that the method is conditionally stable, the stability is evaluated by analyzing two responses of the same model, obtained with two different integration time intervals and smaller than the critical interval, which must be identical*

1 INTRODUCTION

The main actions operating over transmission lines structures (TL) are dynamic in nature, such as wind and cable breakage. The wind is the main action and must be carefully considered in the design phase. The cable breakage, though occurring less frequently, is one of the actions that can take the tower to collapse, including causing the phenomenon known as "ripple effect" in which many towers collapse in sequence.

The rupture of a cable or bundle of cables, generates forces in the longitudinal towers TL, while in the wind action, the forces on the towers can be longitudinal or transverse. In the design of a tower, these actions should be considered as dynamic actions, however, for simplicity, are usually regarded as "equivalent static". Currently, this is no longer justified but, due to the great advances in numerical methods and in the computational area, making it possible to obtain the response of the structure through a dynamic analysis, which would provide the obtainment of more realistic results and consequently the design of a more efficient and economical structure.

The mechanical model usually adopted in TL metal lattice towers project is quite simple, using space frame elements on main legs and spatial lattice in the diagonal bracings, rigid joints (support or bezels) and pinned or fixed connections. The answer is usually obtained through a static and linear analysis. It should be noted that in some types of towers, for example, guyed towers, or freestanding towers with great height the nonlinear geometric analysis is common.

Determining the effects of the dynamic nature actions on TL towers is a complex task, given the range of variables involved and its randomness. The use of methods that result in more proximity to reality is essential to design the towers safely, seeking to maintain the efficiency and economy in their design.

Therefore, a program for performing a dynamic analysis of TL structures, in a simplified and precise manner, becomes necessary. The explicit direct integration method (DIM) of the equations of motion, with central finite differences, which presents a relatively simple formulation, can be used for the response over time in the towers bars, the cable elements and insulators of a TL, also allowing to treat physical and geometrical non-linearities with relative ease, besides the advantage of not requiring the assembly of the global stiffness matrix, since the integration is carried out at element level [1]. It is worth mentioning that the analysis is restricted to structures that can be discretized with the bar elements, in this case, spatial lattice elements.

2 OBJECTIVE

The main objective of this work is to develop a computational routine in FORTRAN, for the analysis of a complete excerpt of TL, including all the components in the model (towers, guy-wires, lightning rod cables, conductor cables and insulators chains) to obtain the dynamic response, in the time domain, using the direct integration method (DIM) of equations of motion explicitly applied to transmission lines with guyed lattice towers and subjected to cable breakage (conductor or lightning rod).

To this end, an adjustment in the routine developed by [2] was made, which allows the analysis of freestanding towers.

Then, the routine is tested in a numerical model of a full TL excerpt, considering all of its components with three guyed towers and four spans, simulating a dynamic loading from the rupture of a cable (conductor or lightning rod) in the model. The response is obtained in terms of displacements of the nodes and effort on the bars of the towers along the analysis time, besides the visualization of the frame moving in the time, from the set of nodal coordinates of the entire TL portion generated at certain time intervals and visualized in post-processing programs.

3 EXPLICIT DIRECT INTEGRATION METHOD

In the solution of dynamic problems the explicit direct integration methods (DIM) have been frequently used, however, in TL structures analysis their application is not very common. The DIM theory is presented next, in a nutshell, explicitly, using central finite differences.

For the solution of the nodal coordinate vector of the structureIn which: $\boldsymbol{q}_{(t)}$ is the vector of nodal coordinates at time "t", $\dot{\boldsymbol{q}}_{(t)}$ is the vector of nodal velocities at time "t", $\ddot{\boldsymbol{q}}_{(t)}$ is the vector of nodal acceleration at time "t", $\boldsymbol{F}_{(t)}$ is the vector of nodal external forces at time "t" and $\boldsymbol{M}$, $\boldsymbol{D}$ and $\boldsymbol{S}$ are the matrices of mass, damping and stiffness of the structure, respectively.

Using the central finite differences to solve the equations of dynamic equilibrium, the equation for obtaining the nodal coordinates of the structure at each instant of time, in the directions x, y and z is summarized as follows:

$$\left[\frac{1}{\Delta t^2}\boldsymbol{M} + \frac{1}{2\Delta t}\boldsymbol{C}\right]\boldsymbol{q}_{(t+\Delta t)} = \boldsymbol{F}\left[\boldsymbol{K} - \frac{2}{\Delta t^2}\boldsymbol{M}\right]\boldsymbol{q}_{(t)} - \left[\frac{1}{\Delta t^2}\boldsymbol{M} - \frac{1}{2\Delta t}\boldsymbol{C}\right]\boldsymbol{q}_{(t-\Delta t)} \tag{2}$$

in which: t is the time (in seconds) and Δt is the integration time interval (in seconds).

After the system state is known at time "(t)" and "$(t-\Delta t)$", it is possible to calculate the second member of the equation (2), and then determine the system state in the next time interval "$(t+\Delta t)$". To begin the process, the initial conditions "$\boldsymbol{q}_{(t)}$" and "$\dot{\boldsymbol{q}}_{(t)}$" must be specified, since:

$$\boldsymbol{q}_{(0-t)} \;=\; \boldsymbol{q}_{(0-t)} \;-\; \Delta t\,\dot{\boldsymbol{q}}_{(0)} \;+\; \frac{\Delta t^2}{2}\,\ddot{\boldsymbol{q}}_{(0)} \tag{3}$$

The acceleration vector "$\ddot{\boldsymbol{q}}_{(0)}$" in the above equation can be calculated from Equation (1), considering the time $t = 0$:

$$\ddot{\boldsymbol{q}}_{(0)} = \boldsymbol{M}^{-1}\left(\boldsymbol{F}_{(0)} - \boldsymbol{C}\cdot\dot{\boldsymbol{q}}_{(0)} - \boldsymbol{K}\cdot\boldsymbol{q}_{(0)}\right) \tag{4}$$

When a discrete mass matrix "*M*" and damping matrix "*C*" proportional to the mass matrix are used, therefore, "*M*" and "*C*" diagonal, the resulting matrix which multiplies "$\boldsymbol{q}_{(t+\Delta t)}$"in Eq. (2) will also result diagonal, and the problem is reduced to *n* algebraic equations whose solution is trivial. This is when the *explicit method* happens. Consequently, as only space truss elements will be used in the model, there will be no need to form the global stiffness matrix *K*, for the integration can be performed in the element level without the need to use a solution process of equations systems to determine the vector "$\boldsymbol{q}_{(t+\Delta t)}$", resulting in a significant reduction of the computational effort compared to implicit methods. Thus, the equation of central finite differences to calculate the displacement at any node of the structure in the directions *x, y* and *z* at time $(t+\Delta t)$, is given by:

$$q_{(t+\Delta t)} \;=\; \frac{1}{c_1}\left[\frac{\Delta t^2}{m}\,f_{(t)} \;+\; 2\,q_{(t)} \;-\; c_2\,q_{(t-\Delta t)}\right] \tag{5}$$

in which: $f_{(t)}$ is the component of the resulting nodal force in the corresponding direction (in Newton) at time t, $q_{(t)}$ is the coordinated of linear displacement of the structure node in the direction *x, y* or *z* (meters) at time t and *m* is the nodal mass (in kg). Being:

$$c_1 = \left(1 + \frac{c_m\,\Delta t}{2}\right) \tag{6}$$

$$c_2 = \left(1 - \frac{c_m\,\Delta t}{2}\right) \tag{7}$$

where: c_m is the damping coefficient proportional to the mass or proportionality constant ($c_m = c/m$) and c is the structural damping coefficient (in N.s/m). Substituting Equations (6) and (7) in Equation (5), finally the central finite difference equation results in:

$$q_{(t+\Delta t)} = \frac{1}{\left(1 + \frac{c_m\,\Delta t}{2}\right)} \left[\frac{\Delta t^2}{m}\, f_{(t)} + 2\, q_{(t)} - \left(1 - \frac{c_m\,\Delta t}{2}\right) q_{(t-\Delta t)}\right] \tag{8}$$

The resulting nodal force component $f_{(t)}$ is composed by the gravitational forces $f_{g(t)}$ (Own weight and external nodal forces) and the axial forces $f_{a(t)}$ in the lattice elements, due to axial deformation. The proportionality constant c_m must be adjusted so that the ratio of critical damping (ζ) is the same as the structure analyzed. For each integration step, that is, in the evaluation of Eq. (8) for all nodes in all directions, the updated nodal coordinates lead to axial deformation of the elements, which react with axial forces which oppose to the displacement. For an axial stiffness element "$E\,A$" and length "L" the axial force at a time t is given by:

$$f_{a(t)} = E\,A \left[\frac{L_{(t)} - L_{(0)}}{L_{(0)}}\right] \tag{9}$$

where: E is the longitudinal elastic modulus of the lattice material, A is the transverse section of the element, $L_{(0)}$ is the lattice element length at time $t = 0$ and $L_{(t)}$ is the lattice element length in a given time t.

Therefore, for obtaining the resulting nodal force components $f_{(t)}$, the axial force $f_{a(t)}$ must be multiplied by director cosines of the element axis in the deformed state, and such components added with the respective gravitational force $f_{g(t)}$ in the directions *x, y* and *z*, acting on the node under consideration at time t. It should be noted that, since the nodal coordinates are updated at each integration step, the geometric nonlinearity is always considered.

According to [3], the convergence and accuracy of the solution depend primarily on the integration time interval $\Delta\mathrm{t}$ adopted, therefore the method is conditionally stable. Then, according to [4], for this stability to be ensured, it is necessary that:

$$\Delta t \le \Delta t_{crit} = \frac{2}{\omega_n} = \frac{2}{2\,\pi\, f_n} = \frac{T_n}{\pi} \tag{10}$$

where: Δt_{crit} is the critical integration time interval, f_n is the corresponding structure natural frequency of vibration with n degrees of freedom (in Hz), ω_n is the BIGGEST angular frequency of vibration of the structure with n degrees of freedom (in rad/s) and T_n It is SMALLEST structure natural period of vibration with n degrees of freedom (in seconds). In determining "Δt" through Eq. (10) the difficulty consists in calculating "T_n", which corresponds to the way of vibration associated with the largest eigenvalue of the structure "λ_n" (Or higher natural vibration frequency "f_n"). However, according to [5], for the case of structures formed by lattice components, the critical time interval "Δt_{crit}" can be given, approximately, by the following equation:

$$\Delta t_{crit} = \frac{L_{min(0)}}{\sqrt{\frac{E}{\rho}}} \quad (11)$$

in which: $L_{\min(0)}$ is the initial length of the shortest lattice bar (meters) at time $t = 0$, E is the longitudinal elastic modulus of the material (in N/m²), ρ is the specific mass of the material (in kg/m³).

According to Eq. (11), the integration time interval "Δt" that should be adopted for a structure of the lattice type, basically depends on the initial length of the shortest lattice bar and the speed of sound in the material used in the bar. It is important to note that Eq. (11) serves only to give an approximate idea of the value of "Δt". In order that the accuracy of the results is proved, without calculating the largest eigenvalue of the structure, the obtainment of at least two identical responses with two different values of "$\Delta t \leq \Delta t_{crit}$" is necessary. In the analysis of metal lattice towers according to [6] in order that the method stability is ensured, the time interval between integration steps (Δt) must be of the order of 10^{-5} to 10^{-6} seconds, which requires between 100,000 and 1,000,000 integration steps for each second of analysis of the structure.

4 METHODOLOGY

The dynamic response of the structure in the time domain is obtained through a numerical routine developed in FORTRAN and adapted from [3] to allow analysis of guyed TL metal lattice towers. The routine uses the direct integration method (DIM) of the equations of motion, explicit, using central finite differences. The application routine is performed in three stages, called: pre-processing, processing and post-processing.

In the pre-processing stage, the input data of the complete excerpt of TL are provided, including the towers, the insulator chains, conductor cables, lightning rods and guy-wires, with their discretizations informing the geometric properties of every element, the mechanical properties of the materials and the cable element selected to break.

In the second step, called processing, from the input data informed, the structure is analyzed, and the dynamic response in the time domain is obtained.

In the final step called post-processing, the results can be viewed, from the output file generated in the previous step, in terms of displacements at the top of the towers and effort in elements, in addition to the structure movement visualization over time of analysis with the aid of a post-processing program.

4.1 Cable breakage simulation

The total time set for the analysis of the complete excerpt of TL is 40 seconds. The own weight loads of the towers and the cable elements are applied gradually from 0 to 100% of their value for a 5 second time interval (t = 0 a 5 s). At this same time interval, the initial deformation is applied to give the desired prestressing to the guy-wires, also gradually. The time interval of 5 to 20 seconds is used to dampen any vibration induced in the numerical model. The rupture of the selected cable element occurs at time 20 seconds (t = 20 s) and the remaining 20 seconds of analyzes are spent in obtaining the structure dynamic response over the time. In Fig. 1 the conductor cable elements are shown (breakage hypothesis 1) and of lightning rod (breakage hypothesis 2) selected to break in the model

The cable elements which must break, both the conductor cable and the lightning rod, are situated near tower 2, the line span 2, between towers 1 and 2, specifically the element 2422 for the conductor cable 1 and the 2022 for the lightning rod 1, as shown in Fig. 1.

The definition of span 2 for the cable breakage, near tower 2 was made with the objective of minimizing the return of the conductor cables and lightning rods vibrations that can occur close to the extremity nodes of the model, when the cable breakage happens, since these nodes are defined as immovable. The tower 2 is the one that is farthest away from the extremities, and therefore, selected for the analysis of efforts and displacements.

The breakage of a cable in the TL full numerical model is introduced into routine nullifying the axial force ($f_a(t)$) acting on the selected element to break in a moment of time, specified, in the case of this analysis at t = 20 s.

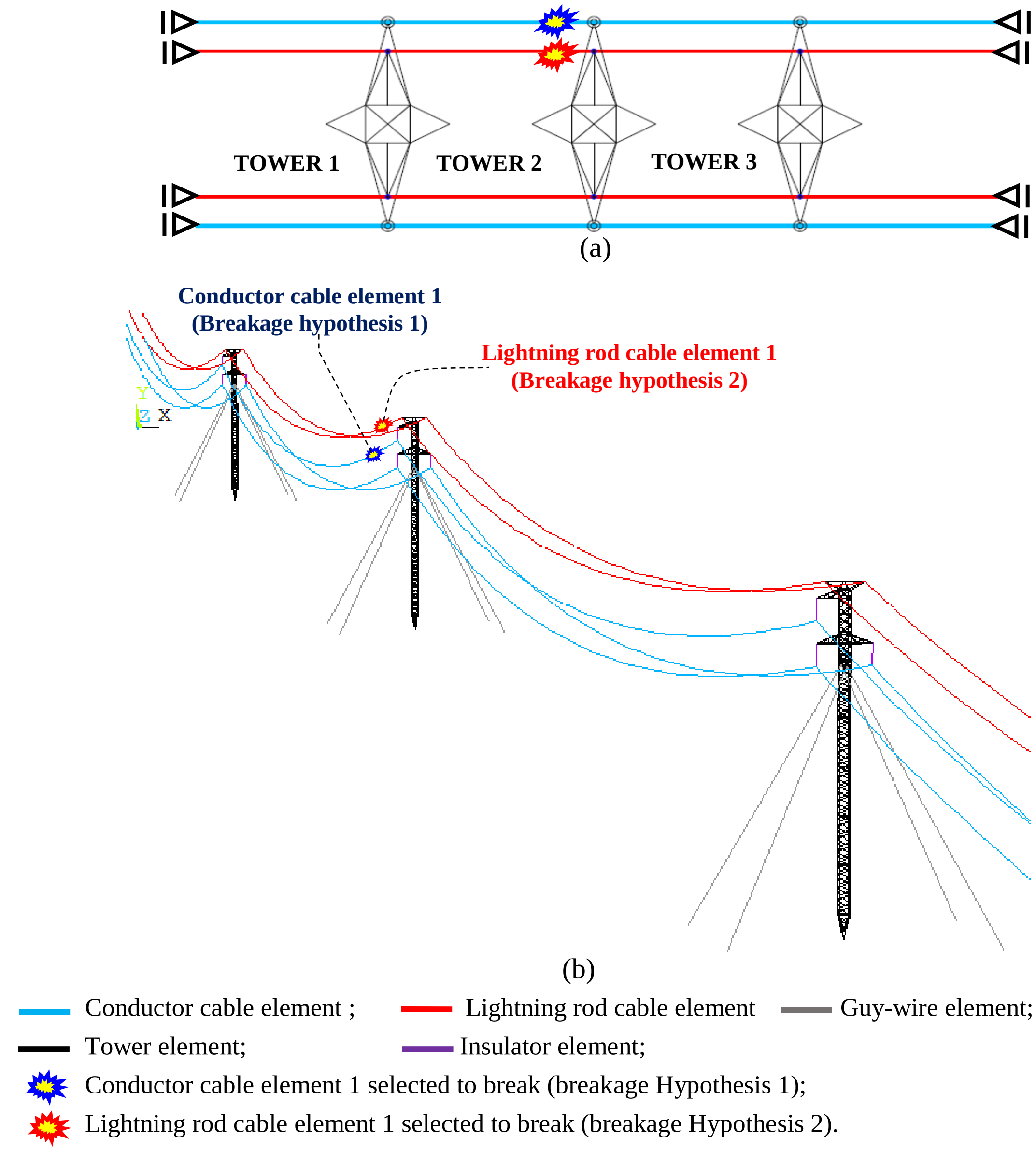

Figure 1: Selected items to break in the dynamic analysis: (a) top view of the full TL model (b) model view in 3D perspective.

For the viscous damping (proportional to the nodal mass) defined by the constant cm (constant that relates the damping with the nodal mass), present in the equations (6) and (7), the values $\mathbf{c_m = 3}$ and $\mathbf{c_m = 2}$ were adopted for the elements of the towers and insulators, respectively. For cable elements, where the damping has a more significant role in the dynamic response, the value of $\mathbf{c_m = 1}$ was adopted. The values of the constant $\mathbf{c_m}$, for each node of the structure, have been defined based on a parametric study by [7]. It should be noted that the constant $\mathbf{c_m}$ must be adjusted so that the critical damping ratio (ζ) is close to the one of the actual structure, in the case of metal lattice towers it varies from 3% to over 10%. Table 1 shows the main input data of the numerical routine for the excerpt in question.

Gravitational acceleration	9,81 m/s²
Constant $\mathbf{c_m}$ applied to the tower nodes	3,0
Constant $\mathbf{c_m}$ applied to the insulators nodes	2,0
Constant $\mathbf{c_m}$ applied to the cables nodes	1,0
Total analysis time	40 s
Time interval for application of the own weight and prestressing of stays	0 to 5 s
Time interval for the damping of induced vibration	5 to 20 s
Instant of time set for the cable breakage	20 s
Time interval for the structure response analyses	20 to 40 s
Total number of nodes in the model	1836
Total number of elements in the model	4012
Integration time intervals used in the analyses	2,5E-06 and 4,0E-06 s
Initial deformation in the stays 1 and 3 (Pair 1)	1,452840E-03 m/m
Initial deformation in the stays 2 and 4 (Pair 2)	1,452083E-03 m/m

Table 1: Basic input data in the routine in FORTRAN for the dynamic analysis by explicit direct integration in the full TL excerpt.

For the application of DIM, the developed program makes it possible to discretize the structure only with space truss elements, which is considered as a limitation, since lattice towers when modeled with only space truss elements may have an interior hypostatization. This problem can be circumvented with the use of dummy rods with an axial stiffness such as to avoid instability of the structure and to not significantly alter the results. The calibration and evaluation process of the dummy bar inserted in the numerical model is described in [8].

The evaluation of the response accuracy and stability of the numerical method is done in two ways, namely: comparison of the fundamental frequency of a model with only one tower, without cables or insulators, with the modal analysis of the same model and of the model dynamic response verification using two different time intervals of integration (Δt_1 and Δt_2), smaller than Δt_{crit}, which should result identical.

4.2 Evaluation of the method accuracy

To evaluate the accuracy of the numerical method, a model with only one tower, without cables or insulators, is subjected to an impulsive horizontal load on top, in order to obtain the horizontal displacement of the structure in the direction of the force applied over time analysis. Thus, it is possible to determine the frequency of vibration associated with this mode and compare it with that obtained in a modal analysis with the same model of isolated tower. This evaluation is done in two horizontal directions, X and Z.

The application of the impulsive load is performed as follows: the bearing of the structure own weight is applied gradually from, 0 to 100%, in a time interval from 0 to 5 seconds. At the same time, the initial deformation is imposed for the stays prestressing, also gradually. After that, at time t = 5 seconds, the application of horizontal load in the direction X starts, increasingly, during 5 seconds, reaching its maximum value in t = 10 s. Then, the load value is kept constant for 5 more seconds and stopped abruptly in t = 15 s, putting the structure in free vibration for 5 more seconds, until it reaches t = 20 s, that is the final time of this analysis. With the displacements at the top of the tower in the direction X, over time in the last 5 seconds of analysis it is possible to obtain the vibration frequency of the isolated tower associated with this mode. In order to obtain the frequency of vibration of the tower in the other horizontal direction the same procedure is performed, however, with the impulsive load applied in the direction Z.

Table 2 shows the results of vibration frequencies found in the modal analysis and the values found in the dynamic analysis by the DIM with impulsive loads.

Modal Form	Frequency (Hz)		Variation (%)
	Modal An. *	DIM**	
Bending in the direction X	2,739	2,611	4,57
Bending in the direction Z	3,662	3,649	0,19

* Frequencies obtained with modal analysis for the isolated tower model;

** Frequencies obtained with dynamic analysis by the DIM for the model with the isolated tower and impulsive forces.

Table 2: Comparison of vibration frequencies of guyed tower S1E2 in the directions X and Z, obtained in modal analysis and by the DIM with impulsive forces

From the results shown in Table 2 for the model with the isolated guyed tower, it can be seen that the DIM is quite accurate.

4.3 Evaluation of the method stability

To evaluate the direct integration method stability, two different integration time intervals were used (Δt_1 e Δt_2) for the same model of isolated tower, smaller than the critical time integration (Δt_{crit}) of the structure.

The values of the integration time interval (Δt), adopted in the analyses of the isolated tower, are shown in Table 3.

Δt_1	Δt_2
2,50E-06 s	4,00E-06 s

Table 3. integration time intervals adopted for the dynamic analysis of the isolated tower

The results of the model analyzed were identical for both Δt values used, both for the vibration frequencies in the direction X and in the direction Z, confirming the method stability.

5 NUMERICAL EXAMPLE

The model used as an example for application of the DIM consists of a complete excerpt of TL, consisting of three guyed towers, called S1E2, with their respective chains of insulators and stay cable, four spans of conductor cables and lightning rods with 500 m long, with a total excerpt of 2000 m. The towers are on the same level, that is, the cables suspension points are positioned at the same height. At the extremities of the TL excerpts (in $x = 0$ m and $x = 2000$ m), cables are fixed on fixed nodes, that is, with all degrees of freedom to the translation (X, Y, Z) restricted. The same occurs in the anchor points of the stays and at the feet of the towers. The numerical model consists of 1902 actual tower bar elements, 9 insulator elements, 1012 cable elements, (conductors, lightning rods and stays) and 1089 elements of fictitious tower bars. Thus, the complete excerpt results in a total of 1836 nodes and 4012 elements.

Fig. 2 shows the complete TL model excerpt analyzed, identifying the numbering of the three guyed towers and of the four cables spans.

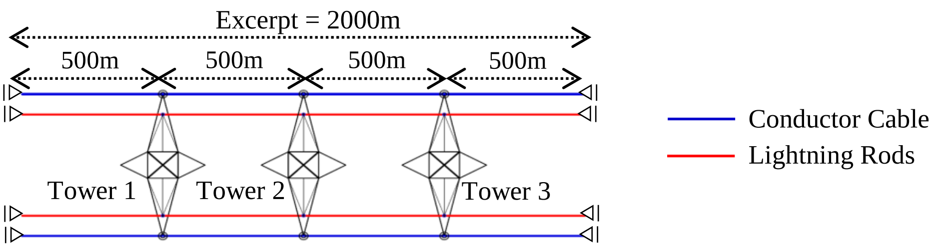

Figure 2: Top view of the TL excerpt analyzed, consists of three towers and four cables spans

The model is subjected to the breaking action of a conductor cable in a first analysis and in a second analysis to the breaking action of a lightning rod.

5.1 Towers and insulators chains

The guyed towers S1E2 are of suspension (tangent), single circuit 230 kV, with arrangement of the conductor cables in an asymmetric triangular shape and two lightning rods. They are metal lattice structures with a total height of 43.5 meters. The central mast has a square cross-section, with 130 x 130 cm. Each tower has four stay cables which ensure the stability of the structure, divided into two pairs, where each pair is fixed on the tower in symmetric points positioned between the trunk and the head of the tower. Each stay cable has a total length of 39.65 m. The silhouette and the spacing between the cable anchor points (foundations of the stay cables) and their connection points in the body of the tower are detailed in Figures 3 and 4.

The stays anchoring happen at 17.0 m from the central mast axis, in the direction X and 15.0 m in the direction Z (model global horizontal axis) as can be seen in Fig. 4.

The insulator chains in the numerical models adopted were considered with a single element, able to withstand only tensile stresses. They are suspended at the extremities of the arms of the conductor cables and are shaped with a length of 2.70 meters.

5.2 Conductor cables, lightning rods and stay cables

Regarding the mechanical performance of the conductor cables and lightning rods used in TL, linear and nonlinear stress-strain relationships can be used. For non-linear relationships, the expressions are given by a polynomial of 4th degree, similar to those employed by [9, 10]. In this work, the behavior of the cables was considered to be linear. Thus, the stress-strain diagram of the conductor cables and lightning rods for a constant temperature is a straight line. For the stay cables the behavior was also considered to be linear.

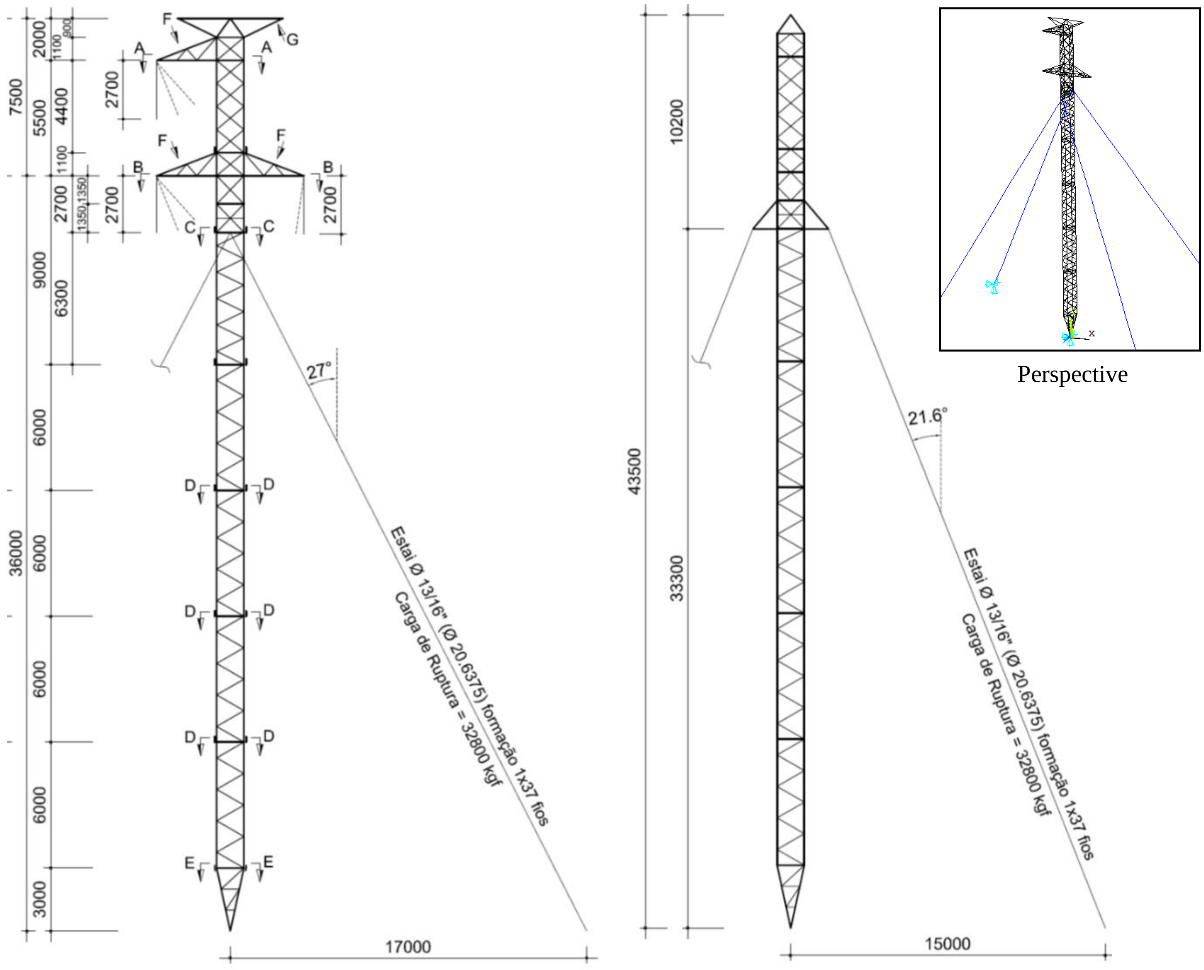

Figure 3: Front, side and perspective view of the guyed tower S1E2 with the structural configuration of the elements and other geometric details (dimensions in millimeters)

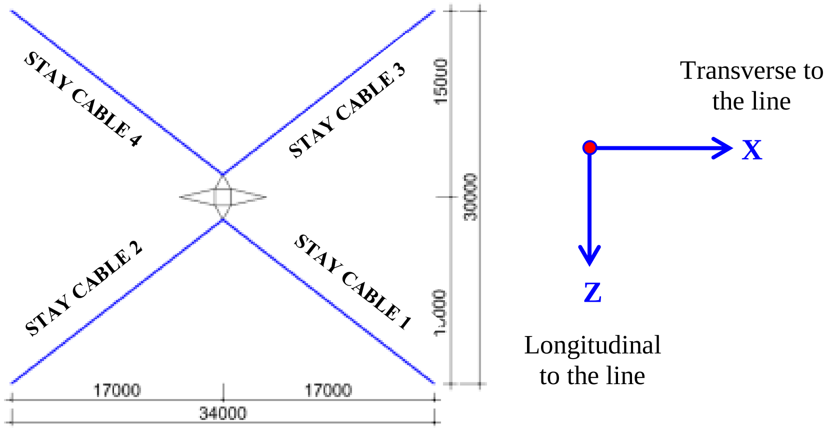

Figure 4: Top view of the guyed tower S1E2, with the distances between the anchoring points of the four stays (dimensions in millimeters).

The conductor cables used in the model are made of aluminum steel-reinforced IBIS type (*CAA/ACSR – Aluminum Conductor Steel Reinforced*) consisting of 26 aluminum wires in the outer layer and beam with 7 steel wires (26/7 cables). The lightning rods used are of steel type Extra High Strength (*EHS - Extra High Strength*) with seven 3/8 inch steel cables. For the stays EHS wire ropes of 37 wires13/16" are used. The mechanical properties of the cables used are detailed in Table 4.

Properties	Conductor cables	Lightning rods	Stay cables
External diameter	19,89 mm	9,14 mm	20,20 mm
Cross-sectional area	234,00 mm²	51,08 mm²	320,00 mm²
Linear weight	7,98 N/m	3,98 N/m	19,42 N/m
Modulus of elasticity	74515 N/mm²	172369 N/mm²	120000 N/mm²
Ultimate tensile strength (T_{rup}) - UTS	72506,0 N	68502,6 N	328000,0 N

Table 4: Properties of IBIS type conductor cables, AAC/ACSR (aluminum cable with steel core), 26/7 wires, lightning rod of EHS type (7-wire) 3/8" and stay cables EHS wire rope type (37 wires) 13/16"

In order that the conductor cables and suspended lightning rods are properly modeled within a TL span, the distorted characteristic of the cables should be taken into account, which in turn has the format of a catenary. The maximum deflection that the cables will reach when subjected to own weight loads and the initial tension should also be considered. The configuration of the catenary, in relation to the center of the wire span (central axis) is given in two ways: symmetrical or asymmetrical. The difference depends on the height of the cable suspension points. The catenary is considered symmetrical when the suspension points are given in the same height, where the vertex is located (center of the span), which is the point where the maximum deflection (f_e) occurs. When the suspension supports are in unequal heights, then the catenary is asymmetrical and the maximum deflection does not occur at the center of the span, as illustrated in Fig. 5. The deflection is due to the span length, the temperature and the tensile level applied to the cable when installed in the TL suspension points.

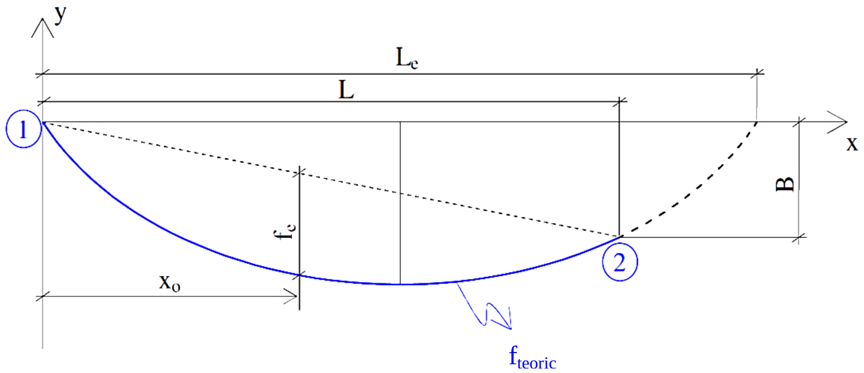

Figure 5: Cable suspended between the supports "1" and "2" with different heights [2]

The suspension points were considered all of the same height (B = 0), that is, all catenaries are symmetrical.

In the case of the suspended conductor cables in transmission lines, their catenary, in the EDS condition (Every Day Stress), is designed for an initial design tension around of 20% of its ultimate capacity of tension (UTS - Ultimate Tension Stress). This value is recommended by [11]. For the lightning rods, the value adopted for the design tensile strength, in the theoretical position, must be calculated for a theoretical catenary deflection "f_{ep}" equal to 90% of the maximum deflection "f_e" of the conductor cables in the span under consideration. It is noteworthy that, according to the [11], the recommended value for the design initial tension in the lightning rods is of the order of 14% of UTS condition.

At the initial time of the analysis ($t = 0$), the weight force is not taken into account in the initial coordinates of the catenary points. Thus, there are two catenaries, one without considering the weight force (own weight), called initial catenary, and another after the application of own weight, called theoretical catenary. In this condition, after the application of the weight force in the structure, the cable must be positioned so that it is subjected to a design tensile strength (T_p), equivalent to a percentage of breaking strength in the cable traction (T_{rup}), with the theoretical catenary ($f_{teórica}$) and maximum deflection (f_e). The value for the design tensile strength of the conductor cables and lightning rods used in the study was calculated by Equations (12) and (13), respectively:

$$T_p = 0.20 \cdot T_{rup} \quad \text{(20\% for the conductor cables)} \tag{12}$$

$$T_{pp} = 0.1175 \cdot T_{rup} \quad \text{(11.75\% for the lightning rods)} \tag{13}$$

The catenary of the conductor cables IBIS AAC/ACSR 26/7 and lightning rods EHS 3/8" adopted in the project (Table 4) in the initial condition (before application of weight force) and theoretical (after the application of weight force) considering a span L = 500 meters and with B = 0, were determined considering the formulations presented in [8]. Additional details can be found in [12]. The results of the catenaries in the theoretical and initial conditions of the conductor cables and lightning rods are shown in Figures 6 and 7, respectively.

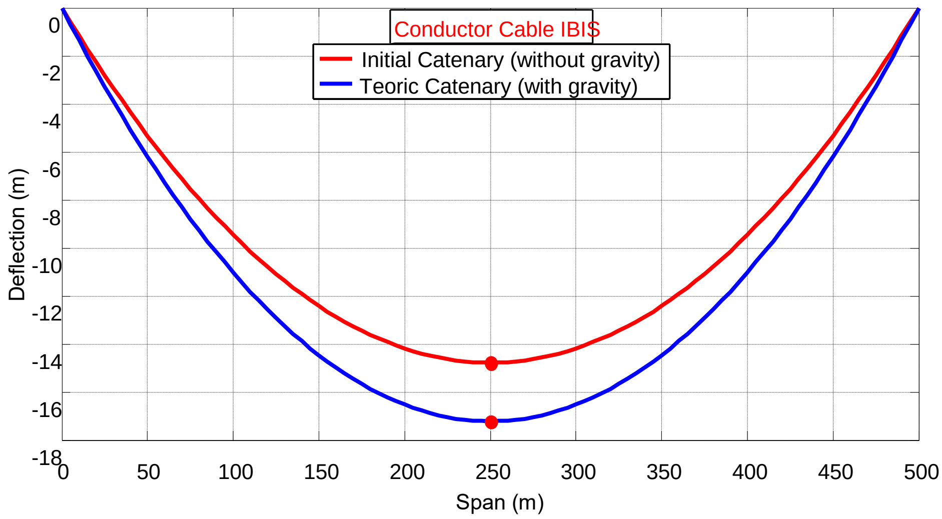

Figure 6: Catenary in initial and theoretical condition for a conductor cable IBIS AAC/ACSR 26/7 considering a span of 500 m between towers and gap between the cables suspension points equal to zero (B = 0)

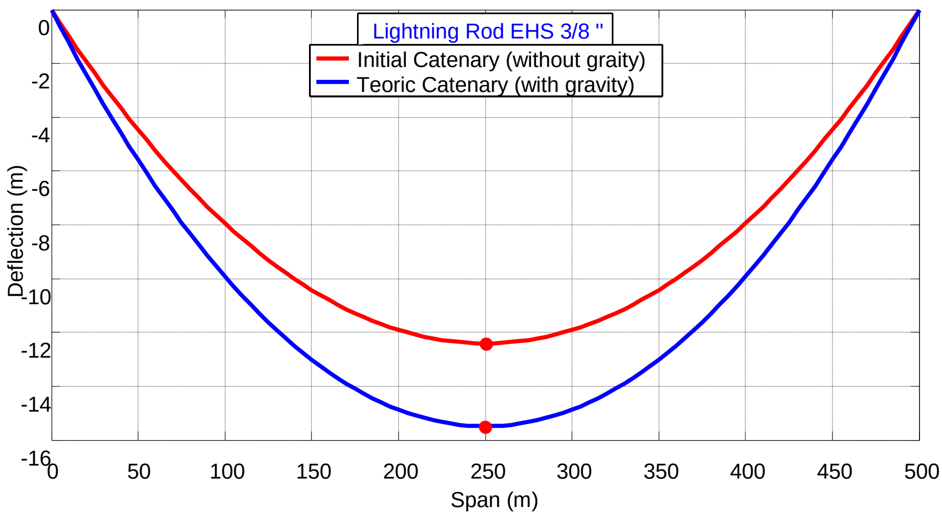

Figure 7: Catenary in initial and theoretical condition for a lightning rod EHS 3/8 ", considering a span of 500 m between towers and gap between the cables suspension points equal to zero (B = 0)

5.3 Stay cables prestressing

The initial prestressing that must be applied to the stay cables of the S1E2 tower, responsible for ensuring the stability of the structure central mast, produces a tensile strength in these cables of approximately 13% of its breaking load, according to recommendations of the [13], which suggests values between 8 and 15% of the cable rated load capacity. In fact, the initial prestressing in the stay cables, applied through a deformation in the cable elements is of the order of 17% of its breaking load, and this value is reduced to 13% after the application of the tower own weight, of the conductor cable and lightning rods and their own prestressing, which causes the central mast to shorten, thereby reducing the initial prestressing.

In the numerical routine, the initial prestressing in stay cable is applied through a constant initial deformation which is added to the deformation of the element when calculating the axial force in the cable element. As the tower has an asymmetrical silhouette, its structure naturally has the center of gravity displaced from the central axis of the mast. Thus, the application of a different prestressing on each side, or each pair of stays is necessary, in order to keep the structure in its upright position.

To determine the amount of prestressing to be applied, a calibration process was done, where the first initial deformation value adopted for the stays was that capable of producing prestressing of 17% of the breaking load on the cables, applied equally on the four stays of the tower. The calibration was made by changing the values of the initial deformation, and consequently of the prestressing in the stay pairs, and by monitoring the horizontal displacement in the direction X of the nodes of the stays arms tip, which should be close to zero, in order to maintain the structure of the central mast vertical. This process was carried out considering only the central mast own weight, so the structure construction process could be represented accurately. The own weight of conductor cables, insulators and lightning rods were applied after calibration of stays. So, as expected, when the own weight of the conductor cables and lightning rods were applied a relief occurred in the prestressing of the stay cables. The relief is generated because of the central mast shortening caused by the own weight of the structure, conductor cables and lightning rods and by the application of prestressing. The values obtained for the initial deformation applied to each pair of stays, so as to ensure that the central mast remains vertical, are shown in Table 5.

Element	Initial Deformation
Pair 1 (stays 1 and 3)	1,452840E-03 m/m
Pair 2 (stays 2 and 4)	1,452083E-03 m/m

Table 5: Initial deformation values applied to the pairs of stays

5.4 Analysis Results

The results obtained for tower 2 analyzed in the numerical routine and subjected to conductor cable and lightning rod breakage in terms of displacement and normal stresses in the bars, are presented below. The results of a static analysis of the isolated model of the same tower, which uses the "equivalent static loads" in the cable breakage simulation, is used as a comparator in terms of maximum displacements at the top and normal stresses on the bars.

5.4.1 Displacements

The results for the displacements at the top of the tower 2 resulting from the dynamics of the conductor cable and the lightning rod breakage in the longitudinal direction of the line (Z) are depicted in Fig. 8 along with the respective displacements obtained in static analysis.

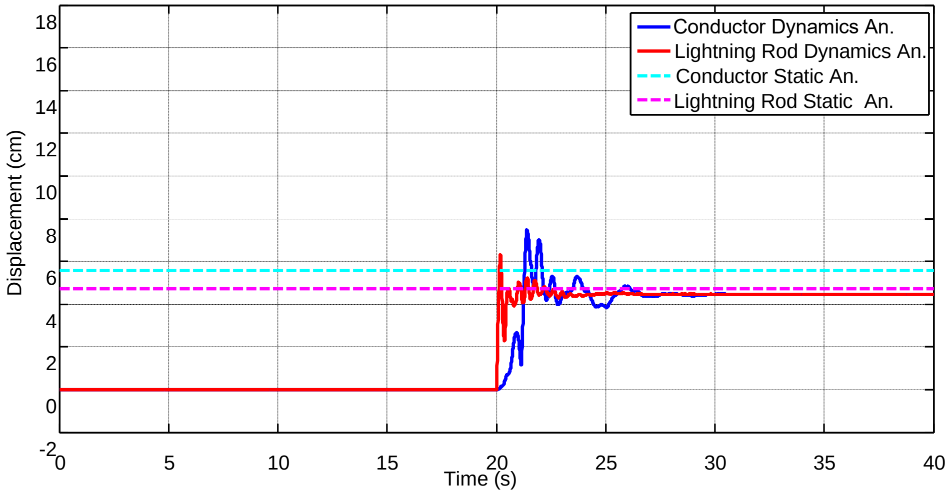

Figure 8: Longitudinal displacements in the node 251 of Tower 2 for the two cases of breakage (conductor cable 1 and lightning rod 1)

5.4.2 Stress in the elements

The normal stresses in stays 1 and 3, for the pair 1 of Tower 2 (adjacent to the breaking point) of the TL excerpt, for the conductor cable breakage hypothesis, are shown in Fig. 9.

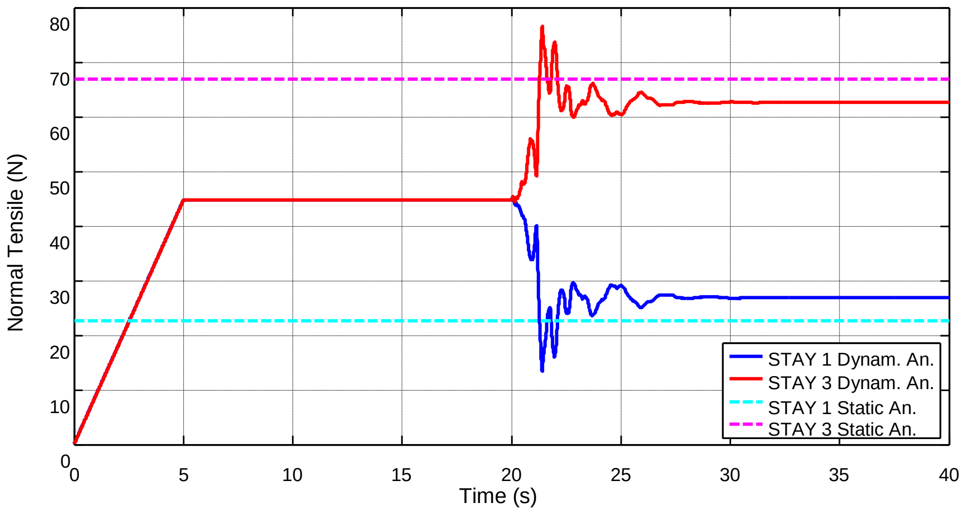

Figure 9: Normal tensile stress in stays 1 and 3 (pair 1) of tower 2 for the conductor cable 1 breakage hypothesis

For the stays 2 and 4, relating to the *pair* 2 of the tower 2 in the TL excerpt, for the conductor cable breakage hypothesis, the values of normal stresses are shown in Fig. 10.

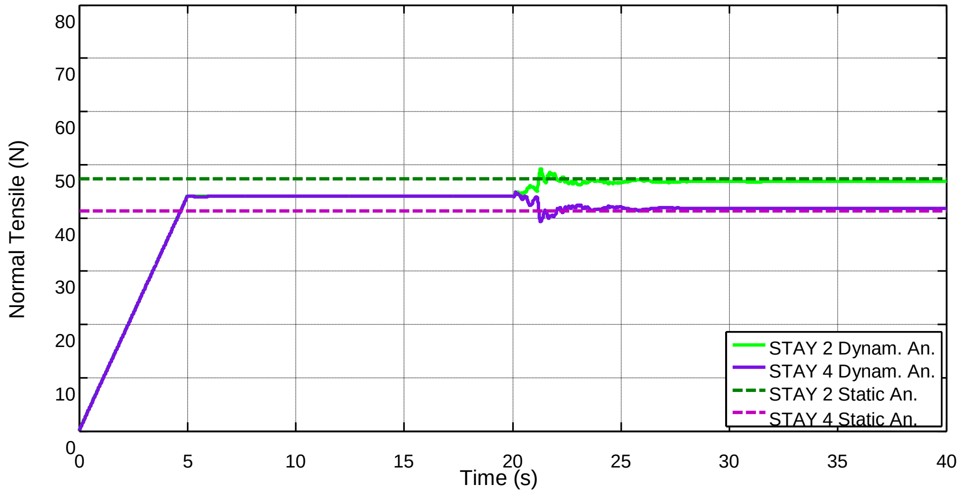

Figure 10: Normal tensile stress in stays 2 and 4 (pair 2) of Tower 2 for the conductor cable 1 breakage hypothesis

Fig. 11 shows the normal stresses on the main leg 173 of the tower 2 positioned below the fixing supports of stays, for the conductor cable breakage hypothesis.

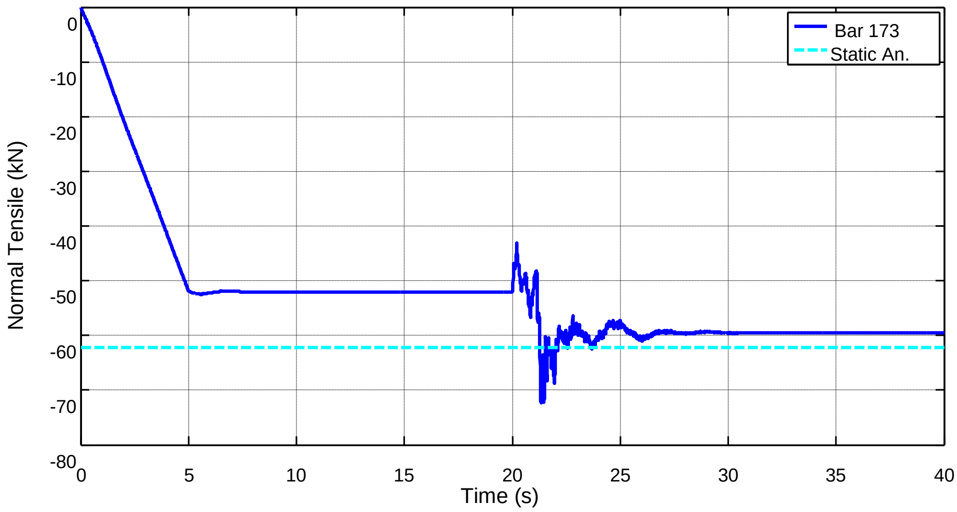

Figure 11: Normal stress on the main leg 173 of tower 2 for the conductor cable 1 breakage hypothesis

In Fig. 12 the stresses of the diagonal bracing 483 of the tower 2 are given, which is situated in the region of the most significant stresses for this type of bar (above the stays fixing corbels) for the conductor cable breakage hypothesis.

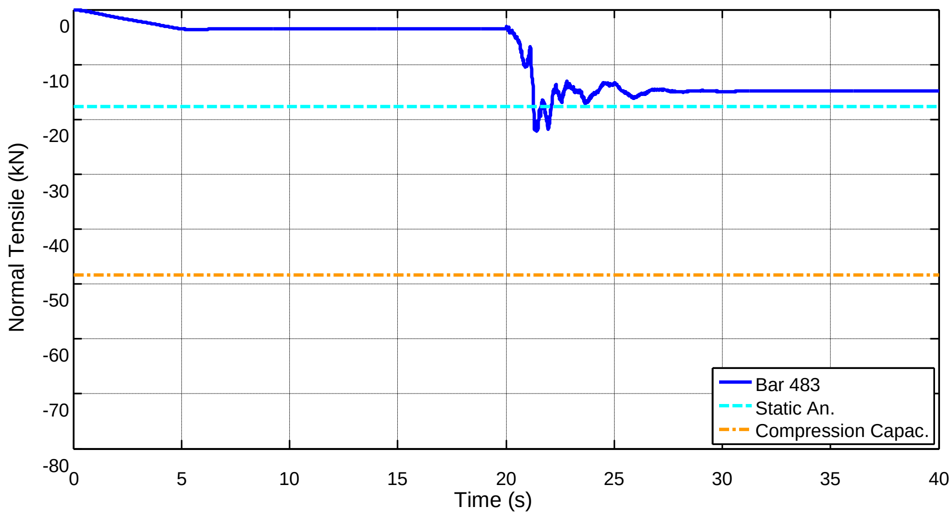

Figure 12: Normal stress in the diagonal bracing 483 of tower 2 for the conductor cable 1 breakage hypothesis

In Figures 13 and 14, the stresses on the bars located in the upper arm that supports the conductor cable which breaks, in tower 2 are illustrated.

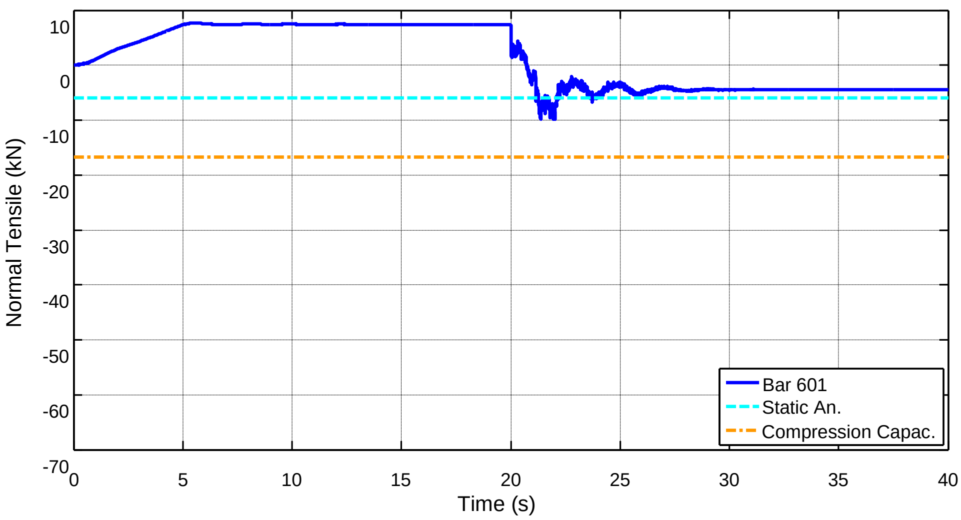

Figure 13: Normal stress in the bar 601 of the arm of conductor cable 1 in tower 2, for the conductor cable 1 breakage hypothesis.

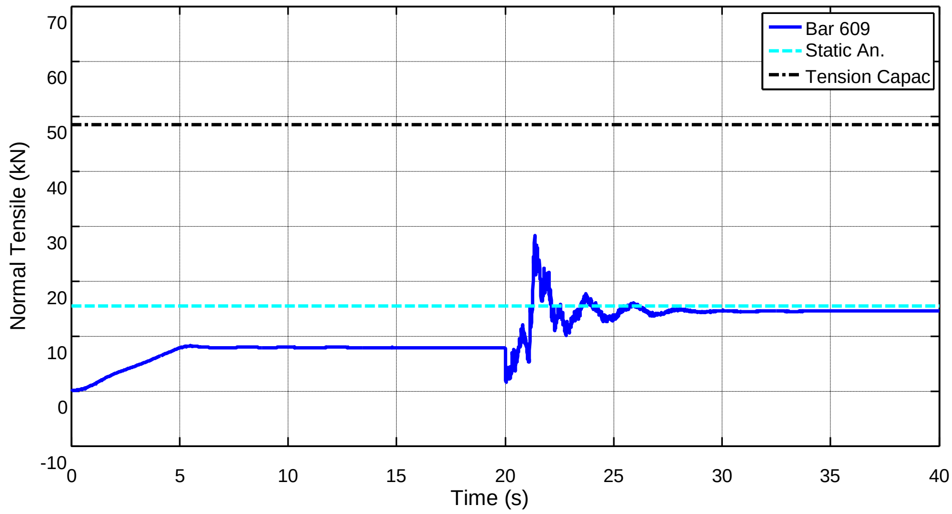

Figure 14. Normal stress in the bar 609 of the arm of conductor cable 1 in tower 2, for the conductor cable 1 breakage hypothesis

The correct functioning of the routine during the gradual application of own weight loads and prestressing on the stay cables (t = 0 to 5 s), time to damp possible vibrations (t = 5 20 s), the instant of breakage of the selected cable element (t = 20 s) and finally, during the interval for obtaining the dynamic response of the structure at the time (t = 20 to 40 s) can be observed by the results presented.

5.4.3 Post-processing

After the results of the dynamic analysis of the complete TL excerpt are obtained, the results output files generated in the routine in FORTRAN in terms of nodal displacements over the time of analysis, can be seen in post-processing programs, making it possible to see the movement of the structure over time. Fig. 15 shows the nodal coordinates of the complete TL excerpt for all time instants determined along the analysis (0 to 40 seconds).

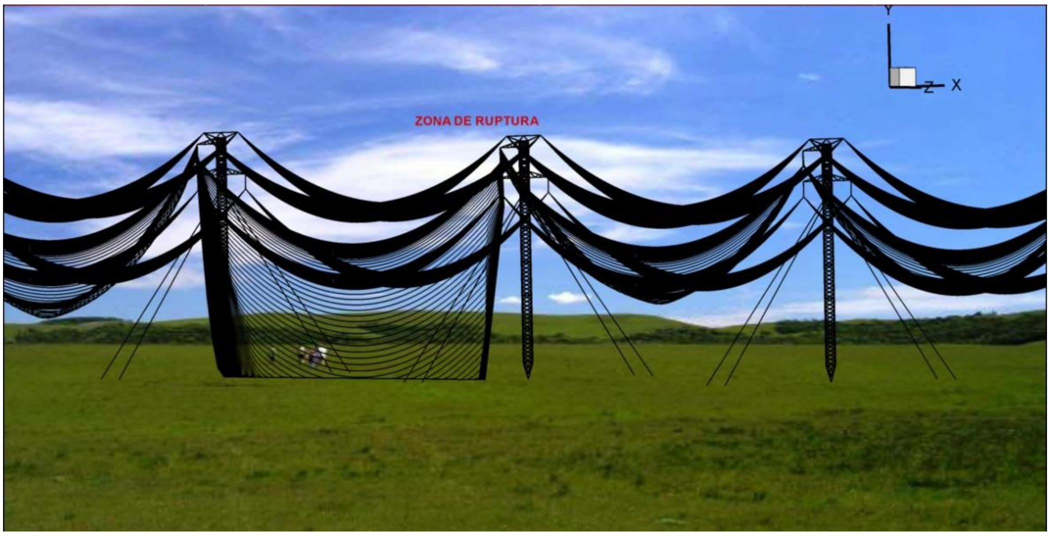

Figure 15: Animation tables of the numerical model of the complete TL excerpt, submitted to conductor cable breakage (illustration background)

From the set of nodal coordinates of all the TL excerpt, generated at certain instants of time, for the conductor cable breakage hypothesis, the post-processing tool also allows the development of video simulation, from start to finish of analysis (0 to 40 s), where the behavior of the tower adjacent to the breaking point can be observed, as well as the deflection variations in the cables adjacent spans.

6 CONCLUSIONS

This work presents a computational routine developed in FORTRAN language able to generate and analyze a complete segment of a TL, consisting of metal guyed towers and including all components subjected to dynamic actions.

The results showed that the routine is very efficient and easy to use, and can become an excellent tool for designers to obtain the dynamic response of TL structures, in order to verify the obtained sizing with a conventional static analysis, especially when new types of towers and cable arrangement is proposed.

In validation processes by vibration frequencies comparison and normal stresses on the elements by impulsive loads, it was found that the numeric routine used for the dynamic solution for the cables breakage problem, using the DIM, is accurate for the analysis, once these processes converged. Moreover, the accuracy of the results was also tested using two different integration times (Δt_1 and Δt_2), which indicated the stability of direct integration method.

Because of the large amount of elements (4012) in the dynamic model of the TL excerpt, with an integration interval (Δt) of 4,0E-06 s, the time invested in the processing of dynamic cable breakage analysis was relatively long, a bit over seven hours, considering a PC with Intel Core i5 and 4GB of RAM processor.

One should take into account the simplifications considered in programming and that their responses come from an approximate numerical solution, since this is an issue with many variables to be further investigated. It is necessary for the program development to be continued so that it can be applied in a more complete program that mainly allows the use of gantry elements plan, simplifying and increasing the accuracy of the models in analysis.

Finally, it is essential that more studies on this subject are conducted with models increasingly realistic and considering other types of guyed towers, besides conducting experimental tests to confirm the accuracy of the results.

7 ACKNOWLEDGMENTS

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