XLI CILAMCE
XLI Ibero-Latin American Congress on Computational Methods in Engineering
16-19 November, 2020 FOZ DO IGUAÇU PARANÁ – BRAZIL

Numerical simulation of full-scale tests in transmission line towers

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Abstract. Transmission line towers are structures designed to carry electrical energy from generating stations to consuming centers. Full-scale tests are traditionally employed in designing verification of transmission line lattice tower structures. Due to the cost of these tests, an alternative is to evaluate their structural behavior using numerical methods. In the present work, a nonlinear analysis method is presented for predicting the ultimate structural response of two steel lattice towers under static load conditions. In the proposed technique, each tower is modeled as an assembly of beam-column elements. The solution scheme consists of an incremental-iterative predictor-corrector strategy based on the arc-length method. Linear, geometric and plastic matrices are used to describe the structure behavior in an updated Lagrangean framework. A lumped plasticity approach coupled with the concept of yield surfaces and plastic hinges is adopted for modelling material nonlinearity. Based on the results, it was verified that the proposed model can produce accurate results to simulate load and failure modes of steel lattice towers. However, some inaccuracies may arise in predicting the collapse load when failure occurs due to elastoplastic buckling of the members.

Keywords: Nonlinear analysis, Transmission line towers, Prototype testing, Steel structures

1 Introduction

Transmission line towers are essential components of electrical power systems and accurate prediction of tower failure is very important for the reliability and safety of the power grid. Most towers collapses are associated with cascades, i.e. progressive failures. When such events occur they can generate losses of millions of dollars for the electric company considering repair, disruption and litigation costs.

One way to verify that the structure has been properly designed is through full-scale tests. These tests are characterized by the assembly of the tower according to the project, being subsequently subjected to the application of loads through cables that simulate the action of the wind and other loads. In order to validate the model design, the towers must withstand 100% of the ultimate design loads. Despite being a common practice, prototype tests are expensive and a cost-effective alternative is to evaluate the structural response using numerical methods.

The usual design practice is to use linear elastic analysis whereby members are assumed to be axially loaded and in several cases pinned conections are also considered. These are major limitations of the analysis model since latticed transmission towers are tall and slender enough to geometric nonlinearity be significant and are constructed using eccentrically connected angle section members that can amplify internal stresses. In addition, several studies show that bending moments in these structures should not be disregarded (e.g. Silva et al. [1]).

This paper presents a nonlinear technique of predicting the transmission tower failure. In order to validate the method, two transmission line towers are analysed and compared to the prototype tests results.

2 Nonlinear analysis

Unlike a linear analysis, in a nonlinear analysis it is necessary to formulate the equilibrium equations for the deformed condition of the structure. As this configuration is initially unknown, an incremental-iterative procedure based on the Newton-Raphson method was adopted. The analysis is divided into several loading steps and at each step, a balance is sought between internal and external forces. In this formulation, the updated Lagrangean framework was used, i.e., the displacements are measured in relation to the last equilibrium configuration obtained in the incremental process. The arc-length control method developed by Ritto-Corrêa & Camotim [2] was incorporated into the procedure in order to compute the nonlinear equilibrium path past the first limit point.

A beam-column element with two nodes and twelve degrees of freedom (Fig. 1) was incorporated into the analysis to model geometric and material nonlinear behaviour exhibited by steel lattice towers. Asymmetric thin-walled open sections, especially equal-legged angle sections, are widely used as tower members. They have relatively low torsional and bending stiffnesses and are connected eccentrically, which causes their formulation to be drastically more complex than double symmetric cross-sections.

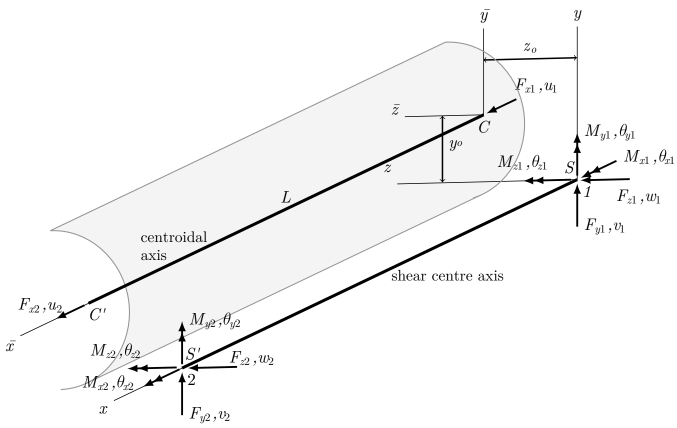


Figure 1. Beam-column element end forces and displacements

The element stiffness matrices are formulated using the local principal generalised coordinates of the element, relating local displacements with the respective local forces. Considering that the element stiffness matrix is formulated in the centroidal and the shear centre axes, which in general do not coincide with the supporting or loading planes, a translational transformation is required to consider excentricities between members. In addition, to relate local coordinate systems to the global coordinate system, a local to global transformation is also required. The details for obtaining these transformation matrices can be found in several books (e.g. Weaver & Gere [3])

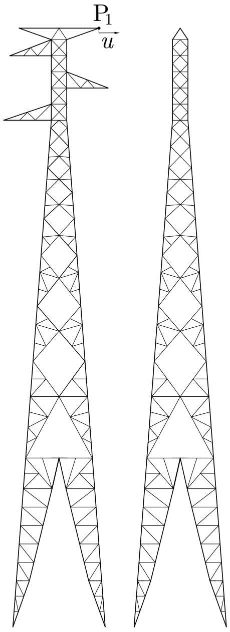
Kitipornchai & Chan [4] developed a geometric stiffness matrix for these sections based on the finite element method which is adopted in this research to handle geometric nonlinearities. This matrix can simulate the large deformation behaviour of space frames in which the influence of the sectorial warping in the member cross section can be neglected. The method is appealing for analysing large scale structures such as transmission towers, as it enables one to determine the structural response in the pre- as well as the post-buckling region, using a reduced number of elements per member.

The assumption of lumped plasticity coupled with the concept of a yield surface in force space is used to treat material nonlinearity of these members, which is an efficient way to include the inelastic behaviour of large assemblies of beam-column members. The whole element cross section is assumed to respond inelastically, in which stress resultants interact with each other to produce yielding for the section. Full plastification, i.e. the formation of plastic hinges, is expected to occur under the action of combined axial force and biaxial moments at the element's ends. The increase of the stress resultants at these locations causes the hinges to yield, triggering a reduction in the element's stiffness accounted for through a plastic reduction matrix. The hinges are assumed to become elastic again upon unloading. The plastification of the cross sections is controlled by means of a yield surface formulated by Alminhana et al. [5] that divides a threedimensional force space into elastic and plastic zones. Steel was considered with behavior elastic perfectly plastic. More details on the method of structural analysis are presented by Roman [6].

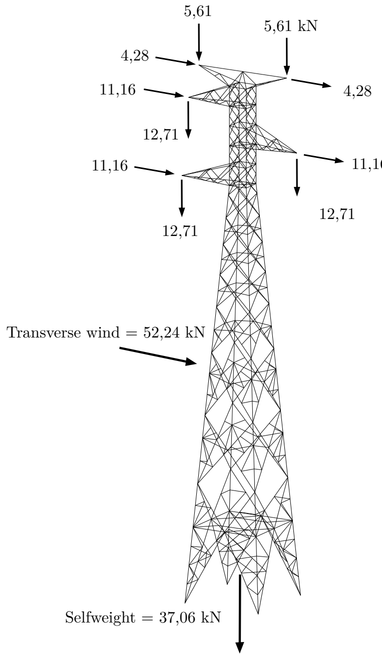
3 Failure prediction of TL towers

3.1 Single circuit 115 kV tower

The first structure analyzed is a 39 m high self-supporting tower with a 6 m x 6 m square base, as shown in Fig. 2a. The support was designed as a tangent suspension structure and has a truncated pyramidal shape, one circuit, triangular (symmetrical) layout of the conductors and staggered bracing at the cage. This structure is part of a 115 kV TL and is formed by steel ASTM A572 grade 50 and grade 60.



(a) Transverse and longitudinal views



(b) Critical load tree

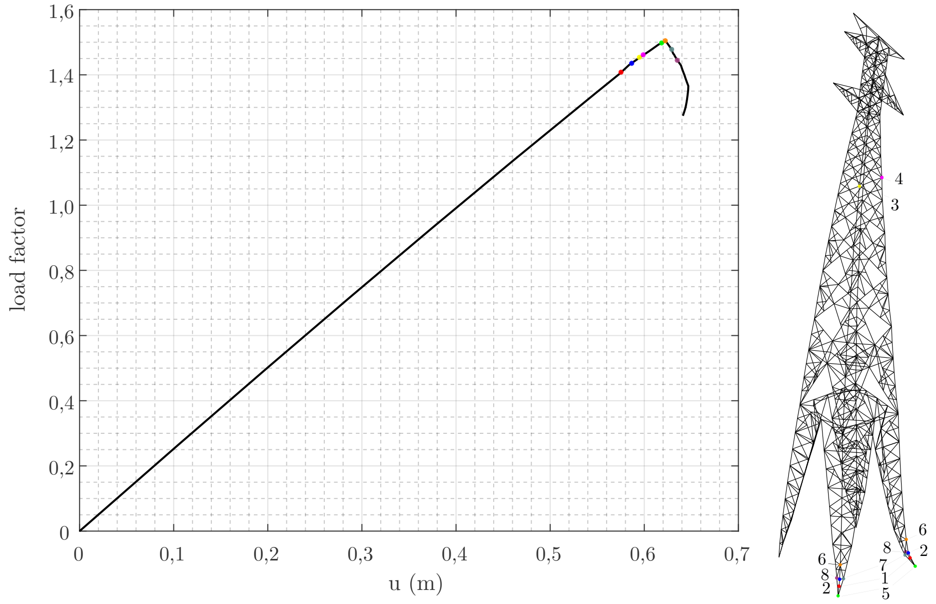
Figure 2. 115 kV tower

The tower was subjected to a full-scale test and the adopted loading tree is shown in Fig. 2b. The collapse of the structure was reached for a load factor, i.e. the ratio of the applied load in the analysis to the specified ultimate design load, of $\lambda = 1.15$ and the failure occurred due to the buckling of the leg. In this case, there was a joint action between the section plasticization and buckling instability in the region of the failure.

The nonlinear analysis method implemented was used to simulate the collapse of the tower. Each member of the structure was discretized into two finite elements, which results in a model composed of 1666 nodes, 2364 elements and a total of 9996 degrees of freedom. This discretization was adopted due to the limitation of RAM available on the computer.

The nonlinear analysis procedure was performed considering the effects of both geometric and material nonlinearities. The equilibrium path for the transverse displacement of the point P_1 (top of the tower) is shown in Fig. 3a. It is noticed that the collapse of the structure was achieved for a load factor $\lambda = 1.51$, which is much higher than the factor of collapse found in the full-scale test. The highlighted points in the graphic represent the formation of plastic hinges. The sections that plasticize during the analysis can be seen in Fig. 3b. The first plastic hinge is formed for a load factor $\lambda = 1,40$ and is located at the point where the failure occurred during the test. Then, new hinges are formed in the compressed leg which considerably reduces the stiffness of the structure in that region until the maximum resistant load is reached. After the load limit point, some hinges are formed in diagonals close to the plasticized legs due to the reallocation of the internal forces on the elements. It is also noticed that two plastic hinges are formed in the tower body. These hinges do not affect the collapse load because the decrease in the stiffness of the structure due to the plasticization of the section has localized effects. Thus, the collapse of the structure will be given by the set of plastic hinges on the legs.

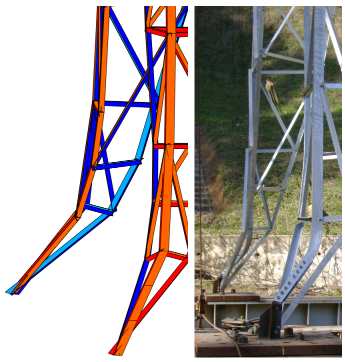
Although the collapse load obtained during the numerical simulation is considerably different from that found during the full-scale test, the failure mode of the structure is similar to that obtained during the load test. This can be verified by comparing the actual structure after the test and the deformed configuration obtained via numerical simulation, shown in Fig. 4. The difference in the collapse load found via numerical analysis and prototype test can



(a) Load path (b) Plastic hinges formation sequence

Figure 3. 115 kV tower

be explained by the consideration of steel having a elastic perfectly plastic behavior. This consideration does not quite represent the reality when the failure occurs due to inelastic buckling and may result in overestimation of the failure load of towers. This statement is based on the expression of axially loaded compression members defined by ASCE 10-97 [7] which provides lower strengths for compressed members that fail due to inelastic buckling (e.g. with slenderness less than 80) compared to the method with elastic perfectly plastic material adoted herein.



(a) Numerical analysis (b) Prototype test

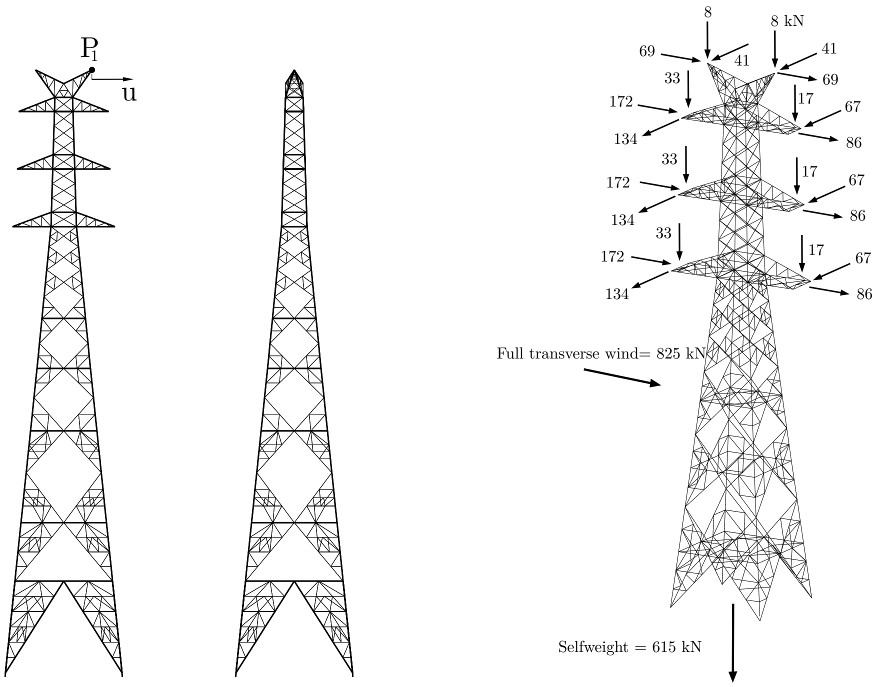
Figure 4. Leg deflected shape after collapse

3.2 Double cicuit 275 kV tower

The second structure analyzed is a self-supporting lattice LT tower, double circuit with a truncated pyramidal shape, designed for a 275 kV transmission line. The tower has a square base of 14 x 14 m and a height of 73 m, as shown in Fig. 5a. All members of the structure are composed of angle sections in steels grades of 250 or 345 MPa.

This tower was initially studied by Albermani et al. [8], who performed the analysis of the structure using a nonlinear analysis procedure in order to predict the collapse of the tower. The purpose of the analysis was to determine the response of the tower, i.e., the nodal displacements and internal forces on the bars, for five static load cases. These represent the loading conditions that could occur during the life of the structure, such as loads induced by broken wires and wind action.

Albermani found during the study that the critical load case corresponded to the condition where the examined tower was subjected to the load tree depicted in Fig. 5b. For this hypothesis, a maximum load factor equal to



(a) Transverse and longitudinal views

(b) Critical load tree

Figure 5. 275 kV tower

$\lambda = 0.96$ was found, which indicates that the collapse of the tower would occur before the full application of the design loads ($\lambda = 1.00$). Albermani reported that the tower failure would be initiated due to the elastic buckling of a hip bracing in the lower part of the tower, which would lead to the buckling of a main diagonal bracing member in the second panel from the ground. As soon as the buckling of this bar occurs, the compressed leg member will also buckle, which will result in the total collapse of the tower. For the other loading cases, the tower behaved properly, with ultimate load factors ranging from $\lambda = 1.06$ to $\lambda = 1.20$. In addition to the numerical simulation, it was reported by Albermani that a full-scale test was carried out and the collapse of the structure occurred during the application of the critical loading hypothesis. The failure was initiated when the applied force was increased from 95% to 100% of the ultimate design load. Later, Alminhana et al. [5] carried out studies with the same structure to assess the load and failure mode through a dynamic analysis method, obtaining results similar to those of Albermani.

The nonlinear analysis technique presented in this paper is used to simulate the tower's response to the critical load case. A finite element model was generated in order to contemplate each member of the tower, which results in a total of 5244 degrees of freedom. Each tower bar was discretized into two finite elements due to computational limitations.

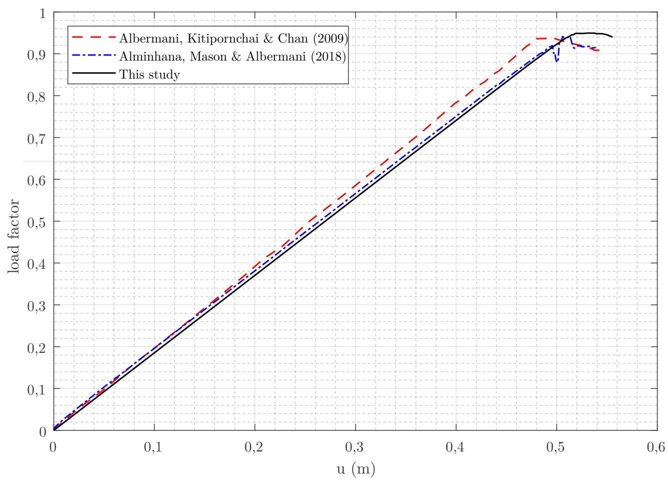
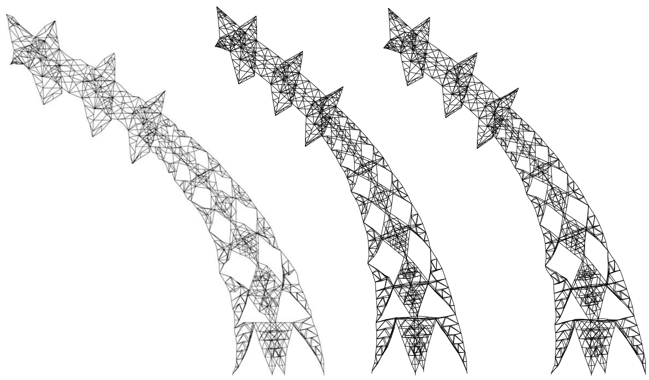


Figure 6. Load-deflection curve at point P_1 in transverse direction

The transverse displacement at the tip of the upper right cross arm, represented by the point P_1 in Fig. 5a, was monitored. The load paths obtained by Albermani, Alminhana and with the present formulation are shown in Fig. 6, with the load defined in terms of the λ load factor. It can be observed that there is a good agreement of the load path obtained in relation to the other curves found in the literature between the three load paths. For the present analysis, the failure load was generated for a load factor $\lambda = 0.95$, which is in agreement with the results of the prototype test. After the formation of some plastic hinges on the compressed legs the structure collapsed, indicating failure due to inelastic buckling. Considering only geometric nonlinearity the failure of the structure occured due to elastic buckling for loads 2.5 times greater than the design load.

According to Albermani, the failure mode found by the full-scale test is similar to that obtained through the simulation. Fig. 6 compares the deformed structures, before the collapse, obtained by Albermani [8], Alminhana [5] and this study.



(a) Albermani et al. (b) Alminhana et al. (c) This study

Figure 7. Magnified tower deflected shape at the time of collapse

4 Conclusions

Two transmission line towers were analyzed, both of them self-supporting lattice structures with pyramidal shape. Compared to the experimental results, the formulation proposed in this article showed accurate results regarding the failure mode in both cases. A good agreement between experimental and numerical collapse loads was found for the 275kV tower. However, the present formulation predicted a collapse load relatively larger than the measured in the full-scale test of the 115kV tower. The reasons of this difference were investigated and it was found out that the consideration of steel as a elastic perfectly plastic material, although a common practice in nonlinear analysis, can produce higher failure loads compared to more accurate stress-strain relationship.

Acknowledgements. This work was developed with the support of Copel Geração e Transmissão S.A. by means of the R&D project 6491-0311/2013.
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