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Reliability assessment of existing transmission line towers considering mechanical model uncertainties

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Abstract. Recent studies have shown the importance of the bolt slippage effect on the structural behavior of transmission line (TL) towers (CIGRE-187 (2009), Ramalingam e Jayachandran (2016), Jiang et al. (2017), just to name a few). However, these effects are not included among the studies regarding the structural reliability of these structures considering model uncertainties. Additionally, the bolt slippage effect has also been disregarded in the structural design, which is carried out through commercial computational packages that employ a linear or geometrically nonlinear elastic analysis. Moreover, the magnitude of these effects is highly dependent on the tower topology, a definition influenced by the engineer experience. Thus, neglecting those effects can lead to a wrong choice of topology configuration. In these cases, the tower elements that are originally designed to resist to axial forces determined through structural analysis procedures that do not include the bolt slippage effect, indeed, can be actually subjected to higher values due to this inaccurate modeling (i.e., they present smaller cross sections that they should have). This fact is even more relevant when old structures are considered, which were designed when studies on this subject were not well established. Consequently, these structures are especially prone to present topologies which increases the influence of the bolt slippage effect, resulting on a structural behavior further away from a linear elastic hypothesis, originally employed on its design. Therefore, given the high number of old towers still operating in Brazil and its relevance for the entire electric system, the main goal of this paper is to assess the structural reliability of existing transmission towers considering mechanical model uncertainties, on a design region (for pre-collapse loads). For this, the behavior of the connections and modeling assumptions are be considered as uncertainties. Thus, allowing, for example, to analyze the reliability of these structures considering different levels of modeling complexity.

Keywords: Transmission line towers, bolt slippage effect, reliability

1 Introduction

TLTs have been widely regarded as one of the most difficult lattice structures to analyze, especially because they are composed of asymmetric thin-walled angle section members eccentrically connected [1]. Thus, several efforts have been conducted to better understand their structural behavior ([2, 3], for instance). Some of these studies point to discrepancies between the design model results and the actual tower behavior. In the context of structural engineering, these uncertainties have been called as *model uncertainties* [4]. According to [5], the main sources of discrepancies in TLT design are the variation in material properties, connections eccentricities, joint slippage (also called bolt slippage), tolerances in manufacturing processes and erection practices.

Nevertheless, except for the bolt slippage effect, it has been observed ([5, 6]) that these divergences occur in similar levels regardless the structural topology, when following the same industrial practices. In fact, the connections slippage influence is highly dependent on the tower structural configuration, which is a definition given by the engineer based on his experience. Even so, these effects are currently ignored in design practice of TLTs, and the commercial packages employ a linear elastic analysis (or geometrically nonlinear).

Therefore, several towers in Brazil have been built with topologies that can increase the impact of the connections' slippage. However, because this effect has not been considered properly on the mechanical model, these structures may be operating with a lower reliability in comparison to what they were originally intended to.

Indeed, reliability methods and bolt slippage modeling on TLTs are reasonable well established in the literature. However, studies merging these two fields (i.e. evaluating the reliability of existing TLTs considering mechanical model uncertainties with particular emphasis on the joint slippage) can not be found, to the best of authors knowledge. Even so, it is possible to find some studies focusing on the structural reliability of TLTs, as the following.

[7] studies the influence of physical and model uncertainties in the reliability assessment of a TLT. Randomness in material parameters, geometry and loads are accounted for with a proper consideration of their spatial correlation. The results show that model uncertainty is a representative factor and should not be neglected in the reliability assessment of TLTs. [8] assesses the structural system reliability of TLs systems. Then, an approximate relationship between the reliability of a fully utilized supporting structure in a transmission line and the reliability of the complete line is developed. [9] conducts an attempt to implement reliability methods to TLTs. For this purpose, a 230kV double circuit is modeled employing 3D truss elements and generic random variables for load and resistance. To identify the most-likely-to-occur failure modes (and their sequences), a replacement technique is applied when the code capacity of some member is exceeded. The process is repeated until the singularity of the stiffness matrix is achieved. Employing the reliability approach proposed by [9], [10] studies a reliability-based optimization procedure for TLTs. [11] describes an approximate method to conduct reliability analysis of nonlinear TLTs, in which the transfer functions from external loading to internal load effects are estimated by regression. Discussions about the modelling of TLTs as series or parallel systems are carried out. [4] evaluates the response of a TL segment subjected to the dynamic loads caused by sudden cable rupture. For this purpose, a TL segment composed by several 138 kV double circuit self-supporting towers, cables and insulator strings is modeled with distinct degrees of details. The authors point out that model uncertainties may significantly influence the outcome of reliability assessment. [12] presents a simulation-based methodology for the optimal span length of a TL which considers uncertainties in both environmental loads and structural resistance. The tower failure is assumed to occur if the combination of vertical and horizontal tower loads lies above an ultimate capacity curve, which is approximately defined a-priori employing a mechanical model that includes material and geometric [13] employs a surrogate model to assess the structural reliability analysis of a TL segment. The load and resistance of structural elements are taken as random variables. The fail is assumed once any of its structural elements has its [14] capacity exceeded. [15] estimates the capacity of TLTs, subjected to uncertain wind loads. Random samples of material properties and section dimensions are generated and non-linear buckling analysis are conducted using ANSYS. By means of a similar approach, [16] compares the results with experimental data of a full scale test.

In this context, the main aim of the present paper is to assess the structural reliability of existing transmission line towers considering mechanical model uncertainties. For this purpose, the failure probability of two real TLTs built in Brazil are investigated employing different levels of modeling complexity. The supports present slightly distinct topology configurations and they had been designed with an important time difference. The first tower was designed 40 years ago and, since then, several TLs are operating in Brazil with such solution. The second is a newer one, with design of 15 years. The study considers since a simple linear elastic model similar to what was originally consider for the towers' design, passing to other model uncertainties relevant for TLT design, such as the definition of redundant members and profile cross-section central axis orientation, until the one including the connections' slippage.

2 Remarks on mechanical model uncertainties and Nonlinear structural analysis

The sources of uncertainties in a structural design may be classified as intrinsic (also called physical uncertainties) or epistemic [17]. The first is associated with the random nature of loads, material properties and so on. In modern codes, it is accounted for by means of partial safety factors, in the so-called Load and Resistance Factor Design (LRFD). On the other hand, the epistemic uncertainties are associated with inaccuracies in our prediction and estimation of reality, in which the most important example is the model uncertainty. In contrast with the physical uncertainty, the ones associated with structural modeling remains virtually ignored in structural design codes [18].

As previously mentioned [5, 19] showed that, with the exception of the joint slippage, all discrepancies occur in similar levels regardless the structural topology when following the same industrial practices. Hence, a proper consideration of the bolt slippage in structural analysis would allow designing supports with more accurate axial forces (when compared to the elastic linear design models), whichever is the topology chosen by the designer (i.e., the divergences would be independent of the tower topology). For further details on this subject (the prototypes constructed and the modeling premises), the reader is referred to [19], [5], [6] and [20] and the references therein.

Based on these consideration, in the present study the physical uncertainties are employed to evaluate failure probabilities, while the model uncertainty is assessed by employing distinct modeling assumptions on a group of predetermined scenarios. Through this procedure it is possible to determine a set of failure probabilities for a specific structure. In its more general version, the model has the ability to consider the presence of redundant members, pinned in-plane rotations for single bolted members, profile cross-section orientations and the nonlinear bolt slippage. When the latter is included, an incremental-iterative solution scheme based on a updated Lagrangian formulation with arc-length control is employed. To capture the bolt slippage effect the connections are modeled employing nonlinear springs. The force displacement relationships are determined based on [21]. Additional details of the modeling can be found in [6, 22]. All the numerical solutions described in this section are implemented in an in-house Matlab code.

The random variables employed are presented in Table 1. Note that for the variables w and t different parameters are considered for each different profile, based on the experimental results of [23]. Furthermore, the parameter for the wind speed are based on [24], for the location of each line.

Table 1. Mean values, c.v. and distribution types for the adopted random variables.

		Example 1		**Example 2**		**Distribution**
		Mean	**c.v.**	**Mean**	**c.v.**	**type**
	V	20.98 m/s	17.97%	15.95 m/s	17.94%	Gumbel
	ϕ_c	26.68 mm	1.00%	27.03 mm	1.00%	Normal
	ϕ_{gw}	9.52 mm	1.00%	9.52 mm	1.00%	Normal
	Fu_b	1.21	4.13%	1.21	4.13%	Lognormal
A36	Fy	355.6 MPa	11.10%	*	*	Frechett
A36	Fu	501.5 MPa	7.90%	*	*	Frechett
A572-G50	Fy	400.7 MPa	4.18%	400.7 MPa	4.18%	Shifted Rayleigh
A572-G50	Fu	531.7 MPa	4.37%	531.7 MPa	4.37%	Gumbel
	E	200,000 MPa	3.00%	200,000 MPa	3.00%	Lognormal
	w	*	*	*	*	Normal
	t	*	*	*	*	Normal
	VA	1	12.09%	1	12.09%	Normal
	VK_1	1	20.01%	1	20.01%	Normal
	VP	1	34.36%	1	34.36%	Normal
	VK_3	1	15.02%	1	15.02%	Normal

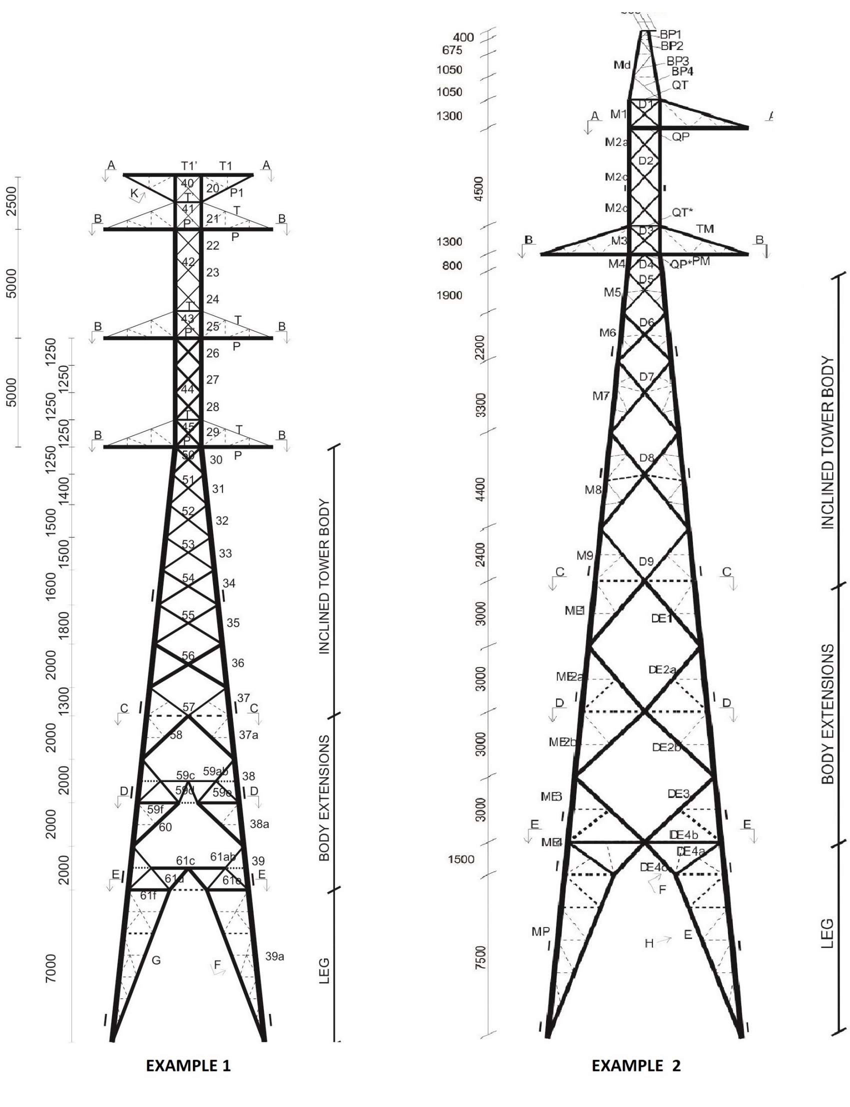
3 Case studies

This section assesses the structural reliability of two existing transmission line towers considering mechanical model uncertainties. For this purpose, the failure probability of two real TLTs built in Brazil are investigated employing different levels of modeling complexity. The supports present slightly distinct topology configurations and they had been designed with an important time difference. The first tower was designed 40 years ago and, since then, several TLs are operating in Brazil with such solution. The second is a newer one, with design of 15 years. The failure is defined when the axial force in any structural member exceeds its respective ULS and it can be written as $M_i(\mathbf{y}) = R_i(\mathbf{y}) - Q_i(\mathbf{y})$, where R and Q are respectively the strength and load on each ith structural member, while $\mathbf{y}$ is the vector of random variables. The strength $R_i(\mathbf{y})$ of each bar depends on the most critical ULS (tension/compression or connection), and $Q_i(\mathbf{y})$ is evaluated considering the wind load and employing a given mechanical model. Thus, if $M(\mathbf{y}) < 0$ for any member i it is considered a failure.

3.1 Example 1

The first example is a 39.85 meters high, 230 kV double circuit, real TL tower built in Rio Grande do Sul state (Brazil). The structure presents a rectangular base, with 5m and 7m in width on the longitudinal and transverse face, respectively. Figure 1 (left) shows the tower design, views, section and dimensions. It is employed both A572-G50 and A36 steel for the angles and A394 for bolts, with 5/8 (1.5875 cm) and 1/2 (1.2700 cm) inches in diameter. Since this is an old design, the [25] protocol was not originally followed. Thus, the wind load was applied in the tower through an uniform pressure of 1,373.4 N/m^2. Additionally, the tower was divided into panels, considering drag coefficients (based on the solidity ratio) and net projected area for each one of them. Then, four different scenarios are considered in example 1 to assess the mechanical model uncertainty in the failure probability determination:

Figure 1. Tower design of examples 1 and 2.



Scenario 1: Original design modeled with 3D-frame elements and a linear elastic analysis. **Scenario 2**: The central members in the four faces of the Sections DD and EE are now added in the structural model. All the other redundant members remain the same (and not included in the mechanical model). The employed finite elements and analysis follows Scenario 1. **Scenario 3**: All the redundant members are included in the mechanical model. In addition, the profile cross-sections orientations and a free in-plane rotation for single bolted members consideration are included. The analysis remains linear elastic. Finally, **Scenario 4**: The same as Scenario 3, but now with the inclusion of the nonlinear bolt slippage effect.

To evaluate the failure probability it is employed the Monte Carlo simulation with importance sampling (MC-IS) adopting 7000 samples. Two variations of each scenario are assessed (i) a fixed wind direction and (ii) the direction is considered as an additional random variable (V_{dir}) with uniform distribution between 0 and 2π. The obtained failure probabilities are illustrated in Table 2.

Furthermore, the [25] targets for P_f for the overall TL system can be used to put the obtained results into perspective. These targets are ($1/T$ to $1/2T$), for $T = 150$ years, leading to P_f targets of $0.33\% - 0.67\%$. However, note that several components (i.e., N towers), rather than a single one, are exposed to the limit load during any single occurrence of the same storm. Therefore, following [25] recommendations, the targets are adjusted for a single structure, which is the case in the present simulation, leading to P_f targets of $0.31\% - 0.61\%$.

Thus, in general, except for Scenario 1, the obtained P_f values are not in accordance with more modern reliability criteria as those recommended by the reference document. Some specific comments can be highlighted:

- **Scenario 1**: The yearly P_f is below the [25] prescribed target for both wind direction variations. Additionally, the failure probability is even slightly higher when the wind direction is taken as a random variable. This is explained because the fixed orthogonal wind direction is mainly governed by the members in groups *26-29* and *30-34*, both leg members in the straight and inclined tower body respectively. However, for yawed winds, the diagonal members (*61d'* and *60'*, for instance) are now subjected to slightly higher loads, increasing P_f.
- **Scenario 2**: The P_f is significantly high for Scenario 2 (much above the target) for both wind directions consideration. In both cases, the limit state in the members *61ab'* entirely governs the failure. Through this scenario, the impact of the model uncertainty associated with redundant member definition becomes clear. The diaphragm in the lower position and its modeling leads to a significantly increase of the axial forces in members of group *61ab'*.
- **Scenario 3**: The inclusion of all remaining redundant members leads to a slightly reduction in the axial force in bars *61ab'* in comparison with Case 2. Consequently the P_f of this case presents a small decrease.
- **Scenario 4**: The yearly P_f is significantly lower to Scenario 4, but, even so, above the recommended target. This is explained because the inclusion of the bolt slippage effect strongly influences the structural balance at the vicinity of the sections DD and EE, reducing the internal force of the *61ab'*, for instance.

In addition, observe that the P_f in Scenarios 2, 3 and 4 is almost the half that when the wind direction is taken as a random variable. This is mainly because the more the wind angle is closer to 0^o, the lower the member *61ab'* is the protagonist.

Table 2. Failure probabilities of numerical example 1

	Scenario 1	**Scenario 2**	**Scenario 3**	**Scenario 4**
Fixed $Vdir$	0.06%	51.19%	36.27%	2.06%
Random $Vdir$	0.18%	25.58%	17.06%	0.83%

3.2 Example 2

The second example is a 46.275 meters high, 230 kV single circuit, real TL tower built in Minas Gerais state (Brazil). Figure 1 (right) shows the tower design, views, section and dimensions. It is employed A572-G50 steel for the angles and A394 for bolts, with 1/2 inches (1.2700 cm) in diameter. In contrast to example 1, this structure was originally designed following the [25] recommendations for the wind load, considering an reference pressure of 409 N/m^2. Additionally, since prototype tests were not performed, a ϕ_R of 0.90 was considered, following the Brazilian industrial practice. Furthermore, note in Figure 1 (right) that this structure presents topology configurations that result in low impact of the bolt slippage effect.

Finally, note that the diaphragm is now located in the upper position (section EE). This leads to a simpler definition of the redundant members in its vicinity. Thus, in this example, the Scenario 2 becomes unnecessary. This results in the consideration of three different scenarios to assess the mechanical model uncertainty in the failure probability determination. Additionally, to maintain the labeling consistent with the former example, the following are adopted:

Scenario 1: Original design, considering a linear elastic analysis and 3D-frame elements. **Scenario 3**: All the redundant members are included in the mechanical model, considering a linear elastic analysis with 3D-frame elements. Additionally, the profile cross-sections orientation and free in-plane rotation of single bolted bars consideration are included. **Scenario 4**: The same as Scenario 3, but now with the inclusion of the nonlinear bolt slippage effect.

The evaluated failure probabilities for example 2 are shown in Table 3, where the same targets from Example 1 are to be considered. As before, two variations of each scenario are addressed (i) the wind direction is fixed (ii) the wind angle is considered a random variable (V_{dir}) with uniform distribution between 0 and 2π. Note that for the three scenarios the obtained P_f values are in accordance (below) with the criteria recommended by the reference document.

Furthermore, for the fixed wind direction the failure probability is governed by leg members members, specially between groups *M6* to *ME3*. The results for Scenario 1 and 2 indicate that the model uncertainties related to the redundant member definition, free in-plane rotation of single bolted bars and the profile cross-section orientation had almost no effect in the obtained failure probabilities. Additionally, in contrast with example 1, the results for Scenario 4 indicate that the inclusion of the bolt slippage effect have a negligible impact on the P_f result.

In addition, similarly to Example 1, when the wind direction is taken as random variable the P_f is reduced by approximately half on the three cases. This reduction occurs since the axial forces in the leg members are reduced when considering lower wind angles (i.e. closer to 0^o about the line directions). In this context, the diagonal members, such as groups *DE3'*, *E'*, *DE4a'* and *DE4c'* play a more important role in failure probability.

Table 3. Failure probabilities of numerical example 2

	Scenario 1	**Scenario 2**	**Scenario 3**
Fixed $Vdir$	0.137%	0.137%	0.118%
Random $Vdir$	0.057%	0.057%	0.053%

Note that the topology configuration of example 2 leads a significantly lower impact of the bolt slippage considerations. In this context, very distinct P_f results are obtained in example 1 (Table 2) depending on the redundant members (central bars in the four faces of Sections DD and EE) and the inclusion of the bolt slippage effect. However, in example 2 these modeling definitions play a minor role in obtained failure probabilities (Table 3). It is highlighted that the major difference between examples 1 and 2 is their topology, specially the diaphragm in the lower position (in example 1). This also reaffirms the observation of previous authors.

4 Conclusions

The impact of the topology definition on the connections' slippage of TLTs is investigated in the present study. In the first example, the Scenario 1 (representing the original design assumptions) lead to P_f lower than the [25] target values. However, Scenarios 2 and 3 consider the members in the four faces of Sections DD and EE in the mechanical model. Nevertheless, only the latter includes the consideration of the profile cross-section central axis orientation and the free in-plane rotation of single bolted bars. Note that in both cases the failure probability is drastically increased, reaching values much higher than the [25] targets. Then, in Scenario 4 the bolt slippage effect is included in the analysis (inserted in the Scenario 3 model). Thus, it is observed a significant reduction in the failure probabilities, despite still slightly higher than the target values. Consequently, an important conclusion, comparing the results for Scenarios 1 and 4, is that the failure probability can indeed be higher than the originally expected due to mechanical model uncertainties.

On the other hand, for example 2 comparing the results for models with and without the bolt slippage effect (Scenarios 3 and 4), the difference is negligible. For this second example, the P_f values in all of the Scenarios remained bellow the [25] target. Additionally, the [25] target represent upper bounds estimates of the towers P_f. Due to industrial production and the assembling in tower families, only a small amount of the supports in a TL are used at their maximum design spans combined at their maximum heights. According [26], the fact that many structures are not loaded as expected contributes to an increase in overall reliability.

Then, although the investigated model uncertainties, specially the bolt slippage effect, can significantly impact the tower behavior, they can be mitigated through proper design. Note that example 2 attains most of the design recommendations in [6], which is not the case in example 1. It is important to highlight that second tower was designed 15 years ago, while the first one was around 40 years ago. Thus, since then several TLs operating in Brazil adopt such solution. Consequently, the results presented herein indicate that could be important to assess these old structures through modern modeling assumptions, giving special attention to the bolt slippage effect. This situation is even more important when considering TL uprating.

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