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Effects of non-linear response of shallow foundations on transmission tower serviceability

Effets de la réponse non linéaire des fondations superficielles sur l'aptitude au service des pylônes de transmission

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ABSTRACT: Electrical transmission tower design has been a key global topic driving the development of reliability analysis of foundations since the 1980s. Many reliability-based ultimate limit state design methods for shallow foundations can be traced to seminal work conducted in the 1980s by EPRI. However, current design simplifies the foundation behaviour to be equivalent to a perfectly pinned or fixed point which does not allow proper consideration of serviceability conditions. This paper presents a simplified assessment of the soil-structure interaction of a transmission tower to better understand the effect of the non-linear behaviour of the foundation on the structural response and elucidate whether it is relevant.

RÉSUMÉ: Depuis les années 1980, la conception des pylônes de transmission électrique est un sujet mondial clé qui a conduit au développement de l'analyse de fiabilité des fondations. De nombreuses méthodes de conception de l'état limite ultime basées sur la fiabilité pour les fondations peu profondes peuvent être attribuées aux travaux fondamentaux menés dans les années 1980 par l'EPRI. Cependant, la conception actuelle simplifie le comportement de la fondation pour qu'il soit équivalent à un point parfaitement chevillé ou fixe, ce qui ne permet pas de prendre correctement en compte les conditions de service. Cet article présente une évaluation simplifiée de l'interaction sol-structure d'une tour de transmission afin de mieux comprendre l'effet du comportement non linéaire de la fondation sur la réponse structurelle et d'élucider si elle est pertinente.

Keywords: Foundation; transmission line; non-linear; serviceability.

1 INTRODUCTION

The determination of serviceability limit states in foundation design depends on the intended purpose of the structure being supported. This limit state may not reflect an intrinsic limit of the foundation itself, but rather a situation for which the foundation movement hinders the ability of the superstructure to deliver the desired service. The same foundation, subjected to similar levels of loading, may have different serviceability limit states depending on it being used to support a building, bridge, or other structure. Even within similar transmission structures, the same foundation can have different serviceability limits.

Currently, transmission tower design guidelines in Canada and the U.S. do not recommend specific serviceability limits for the design of foundations or transmission towers. However, standards from other countries address this issue. For example, the Chinese standard DL/T741-2001 (NDRC 2001) specifies limits for tolerable levels of inclination for pole towers. It also specifies limits at which corrective measures should be taken. Majcherczyk and Niedbalski (2017) indicate that a similar standard exists in Poland. BN-90/9056-01 specifies that the deviation from the vertical of the transmission pylon peak, standing without conductors, is H/300, where H is the height of the structure.

The analysis of the response of transmission towers to wind loads, whether in scaled models, finite element models or considering field conditions, is usually limited to the structure itself. However, few authors have investigated the behaviour of the foundations. (Savory et.al. 1998, 2008) measured the force applied in the foundations by placing strain gages in all the tower feet directly above the foundation. The structure was a 44 m tall transmission tower located in Dorset (England) between 1995 and 1999. Along with the strains, the wind speed at 10 m height was also recorded. The results showed that foundation load is proportional to the square of the wind speed, with measured loads below ultimate limit state loads specified by code, with a code overprediction of 14%.

Structural analysis of transmission towers subjected to wind loads does not consider the foundation stiffness and displacement. This omission can have significant consequences given the non-linear behaviour of the structure. This paper uses a simplified foundation model and assesses the effect on the structural response compared to the typical fixed supports. The objective is to better understand how to define a serviceability limit state for the foundations.

2 METHODOLOGY

2.1 Wind load

The wind load is modelled based on the combination of a mean wind speed and a turbulent wind component. For synoptic winds, the mean wind speed is considered constant at any given height. Variation with height is controlled by a power law wind profile with an exponential coefficient of 1/7, with the wind speed at 10 m height above the ground surface used as a reference value. The turbulent flow around the mean wind speed is modelled as a Gaussian random process based on the Kaimal spectrum and the Davenport coherence model (Simiu and Scanlan 1996), and simulated using the spectral representation method (SRM) (Shinozuka and Jan 1972). The wind speed is transformed into loads following CSA 60826-10 (2019) guidelines.

In this study, a 46.9 m tall transmission tower with adjacent spans of cables is considered. The investigation is carried out based on a location in Ottawa (Canada), and the corresponding 10-minute mean wind speed at 10 m height with a return period of 10, 25, 100 and 500 years equals 22.81, 24.07, 25.94 and 28.09 m/s respectively, which is adopted to generate the wind speed field for later analysis.

2.2 Structural model

A finite element model of the structure developed in ANSYS Multiphysics 18.0 (ANSYS 2014) was used for the analysis. The model was employed by (Mara and Hong 2013) considering rigid foundations. Each tower member is modelled using three 2-node BEAM188 beam elements, with 6 degrees of freedom at each node. The material nonlinearity in the tower member is included by using a bilinear hysteretic model. Each span of cables consists of 2 conduct lines and 1 ground line with a span length of 488 m. Each cable within a span is modelled using thirty 2-node link elements with only axial loading capacity. The ends of the tower-line system are connected to springs that represent the stiffness of the remote towers and cables. The details of the geometric variables and material properties of the tower and cables can be found in Yang and Hong (2016). The structural model includes the effects of the geometric non-linearity using a large-deformation analysis.

2.3 Reaction loads

Figure 1 shows the vertical, longitudinal, and horizontal components of the reaction loads in the foundations of the lattice transmission tower for the 500-year event. The reactions include the effects of the dead load (45 kN), which accounts for the offset between the uplift and compression directions. The main effect of the wind load relates to the axial components required to balance the overturning moment, with the mean value of the horizontal components being less than 10%.

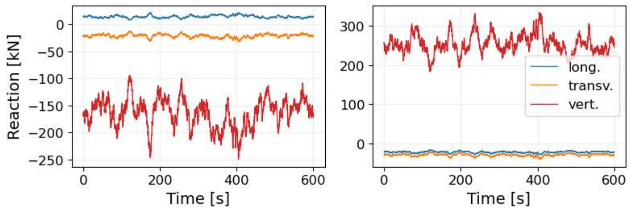

Figure 1. Transmission tower reactions to the 500-year event.

Table 1 shows the temporal mean value and standard deviation of the reaction loads due to the wind alone for different return periods (TR). The horizontal component is generally below 10% of the vertical components, with the significance reducing as the return period increases. It can also be seen that the variation around the mean value, indicated by the standard deviation, is relatively constrained as well.

Table 1. Reaction load mean (standard deviations) in kN.

TR [yr]	Vertical	Long.	Trans.
10	138.2 (17.0)	12.1 (1.5)	16.7 (2.1)
25	153.9 (19.0)	13.5 (1.7)	18.6 (2.3)
100	178.7 (22.4)	15.8 (2.0)	21.6 (2.7)
500	209.5 (26.6)	18.5 (2.4)	25.4 (3.2)

2.4 Foundation design

Following the ASCE (2020) guidelines, foundations were designed for the 100-year event. Uplift capacity was computed using the Meyerhof and Adams (1968) method with a safety factor of 1.5 in a uniform stratigraphy with a loose Hostun sand. The required design parameters are provided by Schanz et al. (1999): friction angle at failure $\varphi_p = 34^{\circ}$, dilatancy angle at failure $\psi_p = 0^{\circ}$, a reference stiffness modulus $E_{50}^{ref} = 20\,MPa$, a stress dependency power $m = 0.65$, an unloading reloading Poisson ratio $\nu_{ur} = 0.2$, a reference Young modulus for unloading and reloading $E_{ur}^{ref}/E_{50}^{ref} = 3.0$. A unit weight $\gamma_s = 19\, kN/m^3$ is adopted. The IEC 60826 (2003) and CIGRE (2002) design guidelines are concerned only with the ultimate limit state, and thus, do not consider the foundation stiffness or force-displacement behaviour. Two circular chimney shallow foundation designs are adopted, a "deep" foundation with an embedment depth (D) of 2.00 m and a width (B) of 0.75 m, and a "shallow" foundation with D of 1.25 m and B 1.75 m. In both cases, foundation and column thicknesses of 30 cm were adopted. According to current design guidelines, these foundations are equivalent to each other (with a safety factor slightly above 1.5).

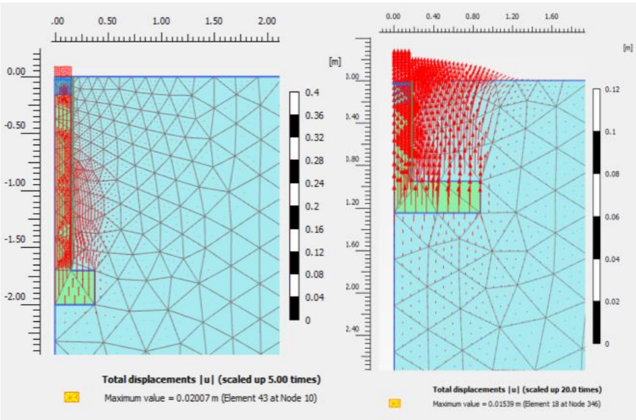

Figure 2. Displacement field at pull-out failure for the "deep" (left) and "shallow" (right) foundations.

2.5 Load tests

Axisymmetric PLAXIS (Bentley 2020) finite element models of the foundations were developed to analyse the response when subjected to different levels of excitation. The Hostun sand is modelled using the Hardening Soil model, an associated elasto-plastic model that combines the Mohr-Coulomb failure surface with an elliptical cap and has a stress-dependent stiffness that distinguish between virgin and un/reloading. Concrete is modelled as an elastic material. Interface elements were included between the column and the soil, so no shear transfer took place. The models were loaded statically first to the dead load level, and then subjected to load cycles corresponding to the 100/500-year events, to determine the foundation stiffness. This stiffness is then used in the structural model to determine the transmission tower responses. Figure 2 shows the displacement (velocity vector) field at pull-out failure for the two foundations with D/B of 2.7 and 0.7 respectively. A clear difference in the behaviour can be seen, where the deep foundation mechanism is far more localized around the foundation (base/column), whereas the shallow foundation mechanism extends to the free surface with an apparently outward-curving edge (related to the angle of dilation of the sand).

3 RESULTS

3.1 Soil-structure interaction

Figure 3 show the response of the foundations when subjected to loads alternating between pull-out and compression cycles corresponding to repeated 100 (blue) and 500-year (red) design storms. The design loads, corresponding to the 100-year event, are shown as dashed grey lines. A clear difference in the behaviour of both foundations is seen. Wide hysteresis loops with increasing permanent deformations characterize the response of the "deep" foundation, while very narrow hysteresis loops are seen for the "shallow" foundation. This is due to the failure mechanisms observed in Figure 2 and reflected in the pull-out and compression test also shown in Figure 3. A common feature of both foundations is that the initial loading is always less stiff than the unload-reload behaviour. This can be in part explained by the high $E_{ur}^{ref}/E_{50}^{ref} = 3.0$ used for the Hostun sand.

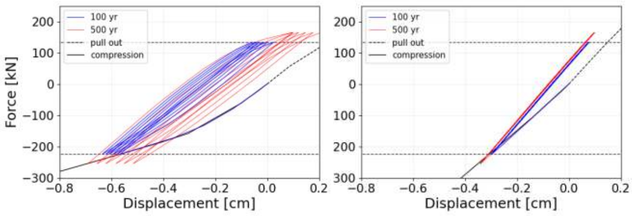

Figure 3. D/B of 2.7 (left) and 0.7 (right) foundations subjected to 100 and 500-year events.

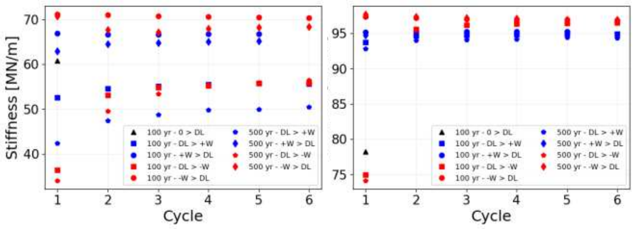

Figure 4. D/B of 2.7 (left) and 0.7 (right) foundations secant stiffness for 100-year and 500-year events.

The hysteresis loops in Figure 3 are transformed into a secant stiffness to represent loading or unloading behaviour, in the compressive and pull-out directions. The resulting secant stiffnesses are presented in Figure 4. After the first load cycle, the stiffness stabilizes. The "shallow" foundation has a more uniform stiffness across load levels (100 or 500-year) for loading or unloading. The latter is reflected in the narrower hysteresis loops shown in Figure 3. The deep foundation exhibits a markedly different behaviour under loading or unloading, with a stiffer unloading, explained by the wider hysteresis loops. Interestingly, whether the loading is in the compressive or pull-out direction, there is no significant change in stiffness. Based on the results, a 60 MN/m stiffness is used for the "deep" case & 95 MN/m for the "shallow" case in the structural model.

Table 2. Transmission tower peak displacements [m].

TR [yr]	Fixed	Deep	Shallow
10	0.1379	0.1760	0.1616
25	0.1543	0.1969	0.1808
100	0.1804	0.2303	0.2113
500	0.2131	0.2723	0.2496

Table 3. Natural frequencies [Hz].

Mode	Fixed	Deep	Shallow
Transverse	1.155	1.033	1.074
Longitudinal	1.219	1.074	1.121
Vertical	1.555	1.555	1.555

The effect of the foundation stiffness can be seen in the transverse and longitudinal natural frequencies of the structure. As Table 3 shows, the vertical stiffness of the foundation affects the overall horizontal stiffness of the structure, with a more flexible foundation inducing a more flexible (lower natural frequency) structure. Therefore, models that consider fixed foundations tend to over-estimate the stiffness and natural frequency of the structure.

4 CONCLUSIONS

It is well-known from theoretical approaches that foundation geometry affects stiffness and non-linear behaviour under repeated load cycles. However, the absence of a proper quantification precluded the estimation of its significance in terms of the SLS. Our analysis has shown that it plays a crucial role in the response of transmission towers to wind loads. This has so far been ignored by design standards. The comparison of different foundation geometries (that from a design point of view are equivalent) shows that the dissimilar responses can affect the SLS and the long-term response after many load-cycles associated with significant weather events. Thus, the structural serviceability cannot be effectively studied without the foundation component, and the foundation serviceability limit is a function of its stiffness and non-linear response to load cycles. This preliminary numerical analysis demonstrates the problem. Further calibration using model testing in a centrifuge and available full-scale test results is required to refine the design approach to be calibrated.

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