Practical approach to optimum design of steel tubular slip-joint power transmission poles

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Abstract

Steel tubular poles are employed in large numbers when used for power transmission and consequently their most economical design is desirable. The pole weight affects the overall cost of a system of steel tubular pole structures considering the cumulative effects of material, manufacturing, transportation and erection costs. Therefore, an optimal design may be achieved by designing the lightest possible slip-joint pole which fulfills the geometric and limit states criteria under specified loading conditions. In this research, a computer-aided approach, for the optimum design of steel tubular slip-joint poles is developed. The pole weight is optimized to satisfy the specified geometric limits and limit states design criteria. The optimization procedure uses a simple algorithm to vary the pole diameter and its taper within the specified geometric limits. This results in a total of 121 poles with different geometric properties. For each pole, first the location of the maximum design stresses within each segment of the slip-joint pole is determined. Maximum stresses are then calculated and used to design the poles to satisfy ultimate strength and local buckling criteria. The serviceability limit states criteria for wind-induced vibrations; the effect of wind drag and the second order effects due to the presence of axial loads are included in the analyses. Using the developed optimization algorithm, simple equations and charts are developed for the optimal design of steel tubular slip-joint poles of various steel grade, and length and loading conditions.
1 Introduction

Steel tubular poles are employed in large numbers when used for power transmission and consequently their most economical design is desirable. Steel tubular poles have smaller plan dimensions and are composed of only a few number of pieces, compared to their counterpart, lattice type towers, which are also used in similar fields. Therefore, especially when used as power transmission or distribution line supporting structures, they are more economical considering the costs of erection and right of way.

Steel tubular poles are generally tapered and manufactured in number of pieces, which can slip into each other to form the entire pole structure. These types of poles are called telescopic or slip-joint poles. Customarily, the circumference thickness is varied for each pole segment along the pole height to obtain a lighter structure. The pole cross-sections may have a rectangular, circular or polygonal shape with 6, 8, 12, 16 or 24 sides. Steel tubular poles having cross-sections of polygonal shape are the most commonly used types. Generally, the number of sides in polygonal poles is determined considering the circumference thickness of the pole cross-section and its diameter.

The pole weight affects the overall cost of a system of steel tubular pole structures considering the cumulative effects of material, manufacturing, transportation and erection costs. Therefore, a most efficient or optimal design could be achieved by obtaining the lightest possible pole structure which fulfills the dimensional and limit states criteria under specified loading conditions. However, the design of tapered steel tubular pole structures is a relatively long process. The location of the design or maximum stress along the height of each pole segment is a function of the distribution and relative orientation of forces acting on the pole as well as the variable geometrical properties of the pole cross-section due to the pole's taper. During the design, the stress is calculated at numerous incremental points within each pole segment and at different points within the pole cross-section to obtain the design stress. Furthermore, steel tubular poles are laterally flexible structures and relatively large deflections may occur due to the effect of lateral loads. Consequently, they are potentially susceptible to large axial loads which could lead to substantial second order effects and therefore, these should also be considered in their design.

Generally, the design process is performed using trial-and-error, which could be inaccurate, tedious and time consuming. Considering this, a computer program, ODAPS for the optimum design and analysis of pole structures has been developed and is described herein.

2 General features of the program

In the program input, dimensional ranges rather than predefined dimensions are used for the purpose of optimum design. The pole with most suitable
dimensions satisfying the ultimate and serviceability limit states requirements but having the lightest weight within the specified dimension ranges is automatically selected by the program. Minimum and maximum limits for the pole bottom diameter and circumference thickness and a minimum limiting value for the pole's top diameter are provided. An upper bound value for the pole's top displacement is also provided in the input data as a serviceability limit state measure. In some cases the lower and upper bound limits for the bottom diameter of the poles are selected considering the appearance of the structure and manufacturing restrictions. Additionally, machinery used for bending of sheet metal to obtain poles having polygonal cross-section can not accommodate more than a certain width and thickness of sheet metal. The dimensions of the galvanizing pool may also restrict the pole size. Therefore, the thickness and diameter of the pole structures are sometimes restricted due to manufacturing limitations. A minimum limit for the circumference thickness is usually specified by the design codes considering the wearing of metal due to climatic effects and attacking chemical agents.

The program is devised to analyze and design pole structures either connected by a base plate and anchor bolts to the foundation or directly embedded into the soil or into a drilled concrete foundation. Different optimum designs could be obtained for two poles subjected to identical forces but having different foundation types. Poles with direct embedment foundations might have smaller base diameter than those with traditional base plate-anchor bolt type foundations to reduce the negative effect of extra weight, resulting from the additional pole length used for embedment, on the optimum design.

Currently, the program is designed to accommodate point loads and concentrated moments in three orthogonal directions as well as a linearly varying wind pressure distribution along the height of the structure. Although it is also possible to implement in the program a more complicated exponential or any other kind of wind pressure distribution if necessary, the assumed linear wind pressure profile is reasonably sufficient for design purposes.

The program is also devised to accommodate a variety of dimensional units. The force and length dimensions are externally provided by the user. Any of the metric, SI or British units could be used in the input data. Since some of the equations used in the program are unit dependent, the program first converts the input data into internally predefined dimensional units and then conducts the calculations. Then, the program outputs the results in user defined dimensional units.

The program is capable of performing a second order analysis due to the effect of axial forces. A flag is provided in the program for the user to control the incorporation of second order effects in the design process.

3 Forces acting on pole structures

Radial stresses could be produced at the pole base due to the expansion or
contraction of metal. Power transmission line poles are subjected directly, to wind loads acting on the pole shaft as well as indirectly, to wind loads acting on the conductors and earth wires. They are also subjected to vertical loads due to ice and dead load imposed by the conductor and earth wires in addition to lateral loads due to either unequal wire spans on each side of the pole, or conductor and earth wires connected at an angle to the pole, Campbell [1]. Such lateral loads due to the conductor or earth wire tension usually act on the cross-arm of a power transmission line pole structure. They are first translated into a concentrated horizontal force and a torsional moment acting on the pole shaft and then input in the program. The wind force acting on the pole structure however, is in direct contact and therefore a linearly varying wind pressure is automatically applied by the program.

4 Design philosophy

The design of pole structures have traditionally been based on ultimate or maximum anticipated loads. The load factors used in the design of pole structures is generally larger than the ones used for building or bridge structures (ranges between 1.5 and 2.5 depending on the type of loading combination and function of the pole structure). This primarily results from the uncertainties in the estimation of loadings, such as wind and ice loads on the pole, earth wire, conductors, projectors and antennas. These loads are functions of the shape of the structure and its accessories as well as the distribution of the loadings along the structure height. Unfactored resistance is usually used to check the ultimate capacity of the structure subjected to factored loadings [2], although factored resistance is used in some design codes, Gonen [3]. The yield stress, $F_y$, considering the overall strength or the allowable stress, $F_a$, considering the local buckling stability of the structure is used as a limiting strength value in the design. Accordingly, stress calculations for steel tubular pole structures are based on elastic analyses. However, the effect of second order forces or geometric non-linearity is always included in the analysis and design. Thus, the elastic stability of pole structures is checked during the determination of member forces and inelastic buckling need not be checked since the pole structures are not permitted to yield. Plastic analyses methods are used only to more accurately predict the response of connections to loading since elastic theory based analysis methods could lead to inaccurate estimation of the response due to the many simplifying assumptions made in spite of the complex behavior of connections [4]. Elastic-plastic analyses methods are also used when considering the effects of selected loading events that have low probabilities of occurrence such as collision of a vehicle with the structure. However, the effect of such rare events is not generally included in the design and therefore is not considered in this paper.

Excessive deflections of pole structures should be prevented to avoid immoderate wind induced vibrations, which may also lead to fatigue distress in
the structure, Collins [5], and to control the magnitude of second order forces. Generally, the top deflection of pole structures is restricted to 1/100 of their height in the design to preclude the detrimental effects of such vibrations. If the structure is very flexible, the fluttering effect due to the wind forces might induce additional forces on cantilever pole structures. Therefore, the design forces on such pole structures are increased by multiplying them by a coefficient called cantilever factor to account for this effect. Deflections can also play an important role in the appearance of a pole structure. Line angles and unbalanced phase arrangements in power transmission poles may create load situations which may cause a pole to look bowed.

5 Estimation of pole displacements

In the program, displacements are determined at the top of the pole, at each slip-joint, at locations of maximum stress within each pole segment and at locations of vertical point loads for the purpose of calculating the second order effects. The unit dummy load method is used to determine the displacements, Ghali [6]. Starting from first principles and going through a series of substitutions for the applied loads and the moment of inertia of the cross-section, the lateral displacement, $\Delta_{pl}$, at a distance $z_d$ from the pole top, due to a lateral point load, $P_l$, located at a distance $z_p$ from the top of the pole is expressed as:

$$\Delta_{pl} = \int_{z_p}^{z_d} \left[ \frac{P_l(z-z_p)(z-z_d)}{E[c_i/(D_t+s_p z)]} \right] dz$$

(1)

where $c_i$ is a constant which is a function of the cross-section shape, $H_p$ is the pole height, and $E$ is the modulus of elasticity of steel.

Similarly, the lateral displacement, $\Delta_{ml}$, at a distance $z_d$ from the pole top, due to a concentrated moment, $M_{hl}$, acting about a horizontal axis perpendicular to the pole's longitudinal axis and located at a distance $z_p$ from the pole top is expressed as:

$$\Delta_{ml} = \int_{z_p}^{z_d} \left[ \frac{M_{hl}(z-z_p)}{E[c_i/(D_t+s_p z)]} \right] dz$$

(2)

In the above two consecutive equations $z_p$ is assumed to be smaller than $z_{th}$, otherwise the lower limit of the integral should be $z_p$. The displacement, $\Delta_{wbl}$, at a distance $z_d$ from the pole top, due to a linearly varying wind pressure distribution along the pole height is expressed as;
Two subroutines, DISPLW and DISPLP are developed for the calculation of the displacements. The trapezoidal rule of numerical integration procedure, Maron [7] is used for the calculation of individual displacements, then the ensuing results are summed to obtain the final displacements.

The pole displacements obtained following the above-defined procedure was verified by comparing the results with those obtained using a finite element model of the structure in the program SAP90, Wilson [8].

6 Optimization procedure

The program first divides the difference between the upper and lower limits of the bottom diameter of the pole structure into ten equal segments. Then a loop is constructed in the program, which varies the bottom diameter of the pole structure between its predefined lower (DBL) and upper (DBU) limits using these segments as incremental steps (SB).

At each new incremental step, the ensuing bottom diameter, DB, of the pole structure is used as an upper limit for the top diameter, DT, of the structure. Now, the top diameter of the pole structure is divided into ten equal segments between its lower (DTL) and new upper (DB) limits. Next, using these new segments as incremental steps (ST), an inner loop is constructed in the program which varies the top diameter of the pole structure between its original lower and internally defined upper limits. Following this procedure, a total of 121 poles with different diameters and tapers are obtained.

These poles are then designed excluding the effect of geometric nonlinearity to determine the circumference thickness and slip-joint length of each pole segment constituting the pole structure. It is noteworthy that during the design, the location of the maximum stress in each pole segment is stored. Next, a subroutine named WEIGHT is called in the program to obtain the weight of each pole structure. Another subroutine named WSORT is called to sort the 121 pole structures in increasing weight order.

The top displacement, \( \Delta_T \), of each pole structure is calculated starting from the one having the lightest weight and the result is compared with the limiting top displacement, \( \Delta_{TL} \), Figure 1.

The pole that satisfies this serviceability requirement is selected. Next, second order analysis is performed to calculate the second order moments at points of maximum stress within each pole segment for this selected pole. Using the sum of the first and second order moments at critical locations, the pole is redesigned (i.e. the thickness of each pole segment is recalculated). Then, the top displacement of this new pole structure is calculated including the second order effects and compared with the limiting value. If this condition is satisfied, then the geometrical
properties and weight of this new pole are first stored in temporary variables and then its weight is compared with that of the old one (i.e., the one designed excluding second order effects). If both have identical weights then the program outputs the result (The second order effects could still be resisted by the same structure designed excluding these effects, since in the program, the circumference thickness is rounded up to the nearest integer value in the order of millimetres). Otherwise, second order analysis is performed for the next heavier section and its weight is calculated after it is designed. The same procedure is followed for this case also. The weight of this pole \(W_2\) is compared with that of the previous one \(W_1\) designed considering the second order effects and the lighter one is selected.

![Flow-chart of the optimization procedure.](image)

Figure 1: Flow-chart of the optimization procedure.
7 Derivation of design tools

Using the program ODAPS, more than 100 poles having various lengths and material yield strength were analyzed and designed for different top point loads. The poles were assumed to have 8, 12 and 16 sided polygonal cross-sections and composed of only a single segment which are believed to be the most commonly used types. Two dimensionless parameters were considered to study the relationship between the geometrical properties, strength and top displacement of a pole structure designed to have a minimum weight. The strength parameter, $\beta_n$, and the displacement parameter, $\beta_d$, are defined as follows;

where $M_b$ is the factored base moment, $\Delta_T$ is the top displacement, $D_B$ is the bottom diameter of the pole structure measured across the flat sides of the polygonal cross-section and all other variables have been defined previously.

Figure 2 shows the weight of optimally designed octagonal pole structures as a function of circumference thickness for various pole’s base moment. The weight of the structure increases as the circumference thickness increases. This clearly indicates that for pole structures of the same flexural capacity, ones with thinner circumference thickness but larger base diameter are lighter than those with thicker circumference thickness and smaller base diameter. However, using a small circumference thickness in the design may lead to a very large base diameter, which subsequently may result in a peculiar conical appearance of the structure, which is aesthetically undesirable.

![Figure 2: Variation of pole weight as a function of circumference thickness.](image)
Figure 3: Sample design charts for base diameter as a function of base moment.

Furthermore, the $D_B/t$ ratio of these structures should not exceed the local buckling strength limits of the structure. Using these limits along with eqns (4) and (5) the base diameter and circumference thickness can be expressed as follows:

$$D_B = \left( \frac{M_b \alpha}{0.70 F_y} \right)^\frac{1}{3}$$  \hspace{1cm} (6)

$$t = \left( \frac{M_b}{0.70 \alpha F_y} \right)^\frac{1}{3}$$ \hspace{1cm} (7)

The above equations could be used for the design or quick design verification of a polygonal tubular steel pole structure subjected to horizontal loads and negligible axial loads.

Alternatively, design charts similar to that shown in Figure 3 are developed to obtain the base diameter of optimally designed pole structures as a function of base moment for various yield stress values.

Design charts are also developed to obtain the top displacement of optimally designed pole structures as a function of the base moment for various yield stress values and top to bottom diameter ratios, Figure 4.

8 Conclusions

A computer program for the analysis and optimum design of steel tubular telescopic pole structures has been developed. Although several other computer programs exist for the analysis and design of steel tubular pole structures, they are limited to cases where dimensions are pre-defined by the user.
Figure 4: Sample design charts for top displacement as a function of base moment.

Different than these conventional programs, the developed program is able to automatically design the pole structure having the lightest weight within given boundary conditions and dimensional ranges. It is also possible to design or verify the design of a pole structure by prescribing in the input data, identical lower and upper limits for the dimensions of certain parts of the structure.

References