Design of overhead line support foundations

Calcul des fondations des supports aériens

A.BOLT, Technical University of Gdańsk, Poland
W.CICHY, Technical University of Gdańsk, Poland
E.DEMBICKI, Technical University of Gdańsk, Poland
W.ODROBIŃSKI, Technical University of Gdańsk, Poland

Synopsis: This paper presents the results of a research study at improving the state of art in the designing practice of laterally loaded foundations for single shaft transmission structures.
The presented method was established for rigid foundations in a wide range of cross-section shapes (square, circular, rectangular) with straight shafts or with an enlarged base for general soil conditions.
A comparison of the predicted values of ultimate moments and those measured in experimental field tests is shown.

1. INTRODUCTION

Electrical transmission line structures are subjected to a unique loading conditions which for the majority of structures result in maximum loads which occur infrequently and act only for short periods of time. It needs a special attention in load evaluation and the use of load factors to account for the statistical nature of variation of climatic loads as well as to provide extra reliability in important lines or greater safety for conditions where failure can injure the workman, and sometimes to control a sequence of failures. The foundations for transmission line structures must satisfy the same fundamental criteria as the foundation for any other types of structure, but the manner in which these criteria apply greatly differs depending on the type of structural system.

This paper describes the design criteria for foundations of single shaft structures. These structures have a single foundation subjected to a large overturning moment with relatively small horizontal shear, compression and torsional loads. In this case typical foundations are:

- drilled shaft foundations, (straight or belled concrete shafts, tubular steel shafts, precast-prestressed, hollow concrete shafts),
- directly embedded structures (the capacity of these foundations is very much dependent upon the quality of the backfill).

The analysis of the practical cases for concrete shaft foundations indicates that most of them, in the range of relative depth $0.5 \leqslant D/B \leqslant 5$, can be regarded as rigid foundations. Embedded rigid foundations loaded by overturning moment rotate around their axis of rotation and mobilize passive and active zones of failure around the foundation, as shown in Figs.1 and 2.

The proposed theoretical model was described in (Bolt, 1976, Dembicki, 1977,1978). Generally the ultimate reaction of soil against the applied, external forces was established with the use of a rigid-plastic model of soil behaviour in a plane system with transition from the plane to a three-dimensional one, by analysing the parameters of the displacement wedge.

In order to determine the bearing capacity of the entire foundation-soil system the static conditions of equilibrium between the internal and external forces were employed.

Generally the applied moment is the resultant of all load components

$$M_{appl} = \Sigma H_{appli}(h_i + z_o) \qquad (1)$$

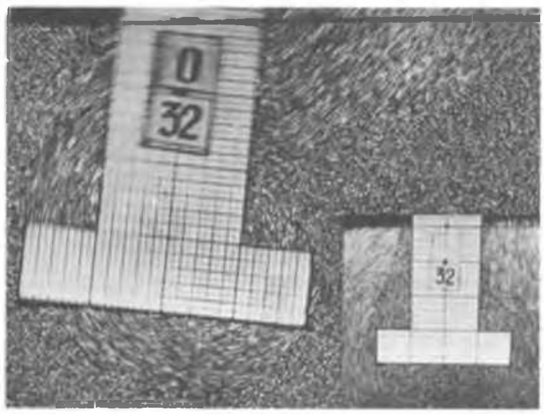

Figure 1. The observed mode of failure for an embedded rigid foundation loaded by an overturning moment.

Figure 2. Typical rupture zone at the soil surface for an embedded rigid foundation loaded by an overturning moment.

where:

H_{appli} - horizontal load component applied on the height h_i above the ground surface,

z_o - depth of the rotation centre below the ground surface.

The mobilized forces H_i and V_i acting on arms r_i and u_i are as follows:

H_1 - passive lateral soil resistance for shallow mode of failure (shearing surface to the soil surface),
H_2 - passive lateral soil resistance for deep mode of failure (local shearing around the centre of rotation),
H_4 - active lateral soil reaction (local shearing around the centre of rotation),
H_3 - horizontal component of the soil bearing capacity under the base,
H_5 - active pressure component assumed for approximation of the side friction component in this zone,
V_i - vertical component of soil resistance in each zone (side shear forces and vertical component of the bearing capacity under the base).

$$M_{ult} = \sum_1^n H_i(r_i + u_i \mathrm{tg}\, \delta_i) \qquad (2)$$

where:

δ_i - average angle of stress inclination on the wall surface in the particular zone due to the failure

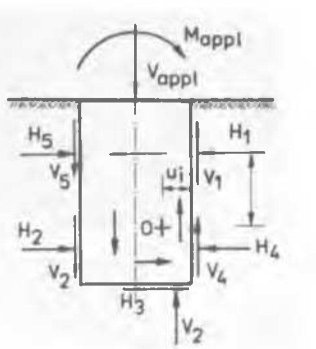

Figure 3. General scheme of applied and mobilized resultant forces.

In this method it is assumed that the foundation is rigid. For all practical purposes the straight foundation has to be categorized as a rigid one when:

$$\frac{D}{\sqrt[4]{\frac{EJ}{k}}} \leqslant 2 \qquad (3)$$

where:

EJ - bending stiffness of the foundation,
k - horizontal modulus of subgrade reaction of the soil-foundation system.

For foundations more complicated in shape (for example, with bells at the bottom) since there is no simple formula and condition of rigidity can be conservatively calculated ignoring the presence of bell.

2. THE PROPOSED METHOD

According to Polish standard PN-80/B-03322 the ultimate equilibrium of the calculated value of applied external forces or moments and ultimate capacity of the soil surrounding the foundation requires that the following condition should be achieved:

$$\gamma_n\, Q_r \leqslant m\, Q_f \qquad (4)$$

where:

Q_r - the design value of the applied resultant loads or moments,
γ_n - factor of the consequences of failure. This factor is equivalent to assigning a class to the line.For the first class it is equal one, for higher classes it increases (a concept similar to IEC, 1984). In each case for a value over one a special economic and statistical analysis is needed,
m - a strength (or resistance or capacity) reduction factor which is used to adjust the reliability of component within a subsystem. For a single shaft foundation subjected to a large overturning moment this coefficient is equal 0.8,
Q_f - the design value of ultimate capacity of soil-foundation system with regard to the variability of the soil properties and strength.

3. LOAD SELECTION

Generally loads for foundations of the transmission structures can be classified as steady-state loads (those that are imposed on a structure for a long or continuous period), transient loads (imposed on a structure for a short-time duration), construction loads (imposed during the erection of structure and during wire installation), maintenance loads (a result of line maintenance activities). The Polish standard PN-83/B-03322 provides specific overload capacity factors for the particular load component and foundation must be sized to withstand this load. The design value of the applied resultant load Q_r can be achieved by selecting a load factor corresponding to the particular load component as described next:

$$Q_r = \sum_{i=1}^{n} \gamma_{fi}\, Q_{ni} \qquad (5)$$

where:

Q_{ni} - the nominal value of the component of applied load,
γ_{fi} - load factor applied to the component i

According to [3] the load factors are as follow:

a) dead loads for structure conductor insulators shield wires and foundation,	1.1 (09)
b) ice loads,	1.4
c) wind loads,	1.2
d) tension loads,	1.3
e) construction loads	1.1

4. SOIL PROPERTIES SELECTION

The disign parameters of soil are defined as follows:

$$x^{(r)} = \gamma_m \cdot x^{(n)} \qquad (6)$$

where:

γ_m - the material uncertainty factor.

This factor depends on variability of the particular soil parameters and the method of approximation of the engineering soil properties. For cases with full site and laboratory test data it cannot be closer to one than γ_m = 0.95 (Method A, PN-80/B-03322), with data about the type of soil and CPT test data then γ_m = 0.90 (Method B, PN-80/B-03322) and with only reconnaissance data, then γ_m = 0.80 (Method C, PN-80/B-03322). For stratified soils the average design parameters can be established from:

$$x_m^{(r)} = \frac{\sum x_i^{(r)} \cdot h_i}{\sum h_i} \qquad (7)$$

where:

$x_i^{(r)}$ - geotechnical parameter for layer i

The major difference between the geometry of directly embedded foundations and drilled shaft foundations is the presence of the backfill annulus surrounding the perimeter of the directly embedded structure. The influence of this material on ultimate capacity of the lateral reactions and vertical side shear forces must be considered when the strength properties of backfill differ from those of the surrounding soil. The loading rate and frequency establish whether drained or undrained analyses are warranted and therefore they determine the type of geotechnical parameters necessary. For coarse-grained soils both dead and transient loads normally result in drained behaviour. However, for fine-grained soils, dead loads result in undrained response.

5. EVALUATION OF THE ULTIMATE BEARING CAPACITY

The ultimate bearing capacity Q_f of the foundation is expressed as the ultimate resultant moment about the centre of rotation at the depth z_o by the following formula:

$$Q_f = M_f = \nu_1 \cdot \nu_2 \cdot \nu_3 \, \overline{M} \, \gamma_D^{(r)} \, D^{(4)} \qquad (8)$$

where:

- D - the total embeddment length of the foundation,
- $\gamma_D^{(r)}$ - the design average effective unit weight of soil,
- $\overline{M}$ - dimensionless bearing capacity factor established as ultimate moment $\overline{M} = M_{ult} / \gamma_D D^4$ for rigid square foundations with the vertical load factor $V_o / \gamma_D^{(r)} BLD = 1$ as function of the design value of soil internal friction $\phi^{(r)}$ and relative width of the foundation base $\beta = B/D$ in range $0.2 \leqslant \beta \leqslant 2$. Values of $\overline{M}$ are shown in Table I and Fig. 4.

Table 1. Bearing capacity factor M

β \ ø	5°	10°	20°	25°	30°	35°	40°
0.20	0.10	0.13	0.14	0.19	0.27	0.41	0.64
0.30	0.16	0.19	0.22	0.29	0.41	0.61	0.97
0.40	0.22	0.26	0.29	0.38	0.54	0.82	1.29
0.50	0.27	0.32	0.36	0.48	0.68	1.02	1.91
0.70	0.43	0.50	0.57	0.72	0.97	1.39	2.11
1.00	0.84	0.97	1.10	1.31	1.65	2.20	3.14
1.20	1.26	1.44	1.64	1.91	2.31	2.96	4.07
1.50	2.19	2.49	2.81	3.18	3.71	4.54	5.92
2.00	4.63	5.21	5.98	6.60	7.40	8.60	10.52

Foundation geometry, influence of vertical components of the applied loads and cohesion in cohesive soils are expressed by factors ν_1, ν_2 and ν_3. Cohesion factor ν_1 is the single parameter which incorporates these factors with design cohesion $c_u^{(r)}$ by the following relationship

$$\nu_1 = \left[1 + \frac{1.6 \, c_u^{(r)} \, (1 - \ln B/D)}{\gamma_D^{(r)} D} \right] \qquad (8.1)$$

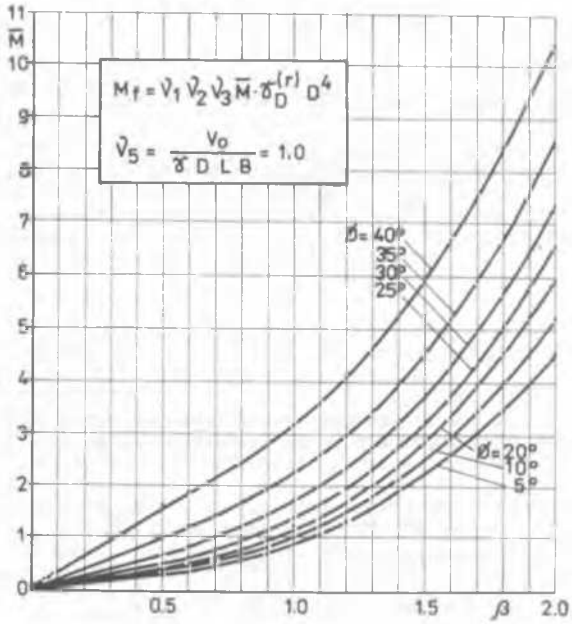

Figure 4. Dimensionless bearing capacity factor $\overline{M}$.

The foundation geometry factor ν_2 incorporates actual changes in width of the foundation and in the shape of its cross-section by the ratio B_1/B and coefficient m_3 established for foundation shown in Fig. 5 (square foundation, square foundation loaded diagonally, drilled shaft and rectangular, straight and with an enlarged base) using the following relationship:

$$\nu_2 = \left[1 + \sin\phi^{(r)} \left(\frac{B_1}{B} - 1\right) \right] m_3 \qquad (8.2)$$

Vertical load factor incorporates the changes in vertical load component by the following relationship:

$$\nu_3 = \left[1 + 0.4\frac{B}{D} \left(\frac{V_o^{(r)}}{\gamma_D^{(r)} DLB} - 1 \right) \cos\phi^{(r)} \right] \qquad (8.3)$$

where:

$$V_o^{(r)} = (V_{appl} + G_f + G_s) \, \gamma_f \qquad (8.4)$$

- V_{appl} - applied vertical load,
- G_f - weight of soil on the foundation base
- γ_f - dead load factor

The horizontal ultimate lateral resistance of the system can be computed as

$$H_f = \frac{M_f}{h + z_o} \tag{9}$$

where:

M_f - ultimate resulting moment from equation (8),
h - desired height of application of H_f above the soil surface,
z_o - depth below the ground surface to the centre of rotation.

The position of the centre of rotation can be established from the following approximate formula:

$$z_o = D \xi_o \nu_4 \tag{10}$$

where:

$$\xi_o = \left(\frac{m_o + \nu_5}{10}\right) \exp\left[0,15\frac{B}{D}\,(1-2\sin\phi^{(r)})\,\frac{0.1 c_u^{(r)}}{\gamma_D^{(r)} D}\right] \tag{10.1}$$

$$\nu_4 = \left[m_1 + \left(\frac{0.07\, c_u^{(r)}}{\gamma_D^{(r)}\, D} - m_2\right)\frac{B_1}{B}\right] \tag{10.2}$$

$$\nu_5 = \frac{V_c^{(r)}}{\gamma_D^{(r)} \cdot D \cdot B \cdot L} \tag{10.3}$$

$$m_o \begin{cases} 5 \text{ for cohesive soils with } c_u^{(r)} > 0 \\ 6 \text{ for granular soils with } c_u^{(r)} = 0 \end{cases}$$

m_1, m_2 } dimensionless coefficient from Table II dependent on the foundation shape

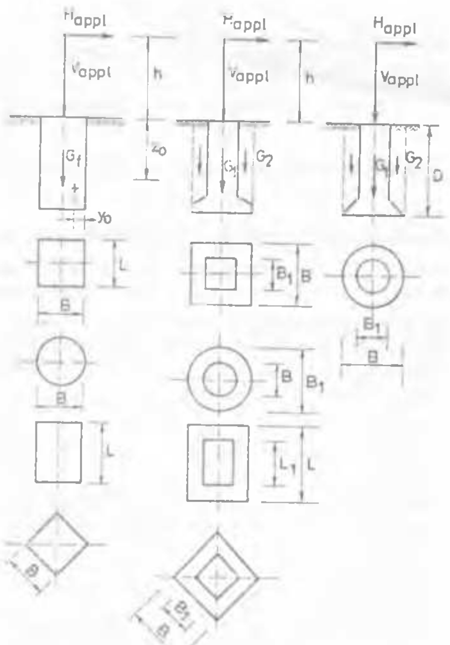

Figure 5. Foundation schemes

Table 2. Shape coefficients m_1, m_2, m_3

m_1	1.22	1.22	1.20	1.16
m_2	0.2	0.20	0.20	0.18
m_3	1	$0.5(1+\frac{L}{B})$	$1.4 \cos\phi^{(r)}$	$0.85\cos\phi^{(r)}$

6. COMPARISOM OF MODEL RESULTS WITH FIELD TEST DATA

Lapeyre, Cagneaux, Veille (1986) have reported field test data on 13 foundations, done for EDF in France. These 13 foundations and 3 Autors' own results are used for a comparison of the proposed-method results with the field test data. The incidence between experimental data and the predicted results (Fig.6) is quite good. In this respect it may be concluded that the proposed method which was calibrated to more than 300 scale model tests (Bolt, 1976) in cohesionless and cohesive soils yields good predictions of the ultimate moment capacity for a very wide range of geometric parameters of the foundations.

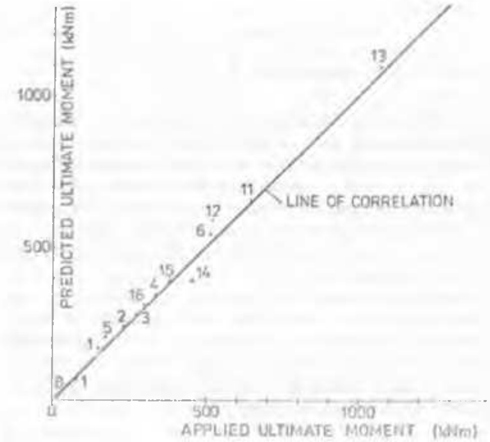

Figure 6. A comparison between experiment data and the predicted results.

7. CONCLUDING REMARKS

A simplified method has been developed for an analysis of the ultimate bearing capacity of rigid foundation embedded in general soil that are subjected to large lateral loads and overturning moments. Based on detailed analyses of the limit state of soil around the foundation a global formula was developed with regard to soil parametrs, such as friction angle ϕ, cohesion c_u and unit weight γ, geometry parameters of foundation L, B, L_1, B_1, D, cross-section shape and actual vertical load component. The method can be used for any rigid foundation with relative depth $0.2 \leq \frac{D}{B} \leq 5$ loaded with a large overturning moment ($\lambda = \frac{h}{D} \geq 4$).

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