Piles subjected to horizontal loads due to asymmetrical surcharges on the surface: a review of design methods

Pieux soumis à des éfforts horizonteaux dus à surcharges asymétriques: une révision des méthods de calcul

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ABSTRACT: Piles in soft soils are subjected to horizontal loads due to unsymmetrical surcharges. This is one of the main problems for construction on soft soils. Those loads cause displacements, rotations and, in some cases, failure of the piles. There are several published methods of calculation with different types of solutions and results, differing considerably. First published solutions were based on theoretical and empirical considerations, based only on equilibrium and basic soil mechanics. Other published solutions are based on numerical models, bi- and tridimensional, elastic and elastoplastic. The lack of recorded case histories and prototype measures make a closed solution difficult, but the paper intends to show which are the main guidelines for a safe and economic design. Additionally, effects of embankment width and distance of pile to embankment crest are discussed based on numerical analysis results.

RÉSUMÉ: Les pieux de fondation qui traversent des couches d' argile trés molle peuvent être soumis à des sollicitations de flexion et à des déplacements importants quand sur la surface du terrain este déposé une surcharge (un remblai, p. ex.) asymétrique. Parfois on peut arriver à la rupture du pieu. Il y a plusieurs méthodes de calcul qui conduisent à des différent résultats. Les premiéres méthodes publiées sont baseés sur des considérations statiques et empiriques. Il y a aussi des méthods basées sur des modéles numériques, elastiques ou elasto-plastiques, á deux ou trois dimensions. Une solution fermée est impossible d'être obtenue. Dans cet article, les auteurs essaient de motrer les principaux facteurs qui doivent étre considerés pour arriver à une solution sûre et économique. L'es influence de la largeur du remblai et de la distance du pieu à ce remblai sont analisées au moyen de calculs numériques.

1 INTRODUCTION

The effect of unilateral surcharges on piles partially embedded in soft clays, generating significant horizontal loadings, has been realized on several occasions (Aoki 1970, Heyman 1965, Marche & Lacroix 1972), like bridge abutments and piles close to surface loadings.

The most common design methods used in Brazil are the one proposed by Tschebotarioff (1973) and De Beer-Wallays (1972). Results, from a practically point of view have been satisfactory, with no known failures or problems when using these methods.

The great number of cases where the situation of unsymmetrical loading on piles happens has lead to a comprehensive bibliographic review of design methods.

Basically, design methods can be divided into two groups:

- "load methods"- methods that consider only loads on the foundation as a structural element - Tschebotarioff (1973), De Beer & Wallays (1972) and Schmiedel (1984);
- "stiffness methods"- methods that consider the loads and the relative stiffness of the foundation - Poulos (1973), Ratton (1985), Goh et al (1997) and Chen & Poulos (1997).

Few experimental data are available, for example one case of 2 test piles, where bending moments, horizontal reactions and displacements were measured (Heyman 1965). Interesting results were obtained using a centrifuge model (Stewart et al 1994).

However, some points, considered important for design, are not covered entirely by these methods. Particularly the effects of embankment width and the distance of the pile(s) to embankment crest are factors that, at least intuitively, can lead to reductions and changes in bending moments of the piles.

2 LOAD METHODS

As described above, the load methods consider the horizontal loads due to embankments independent of soil or pile stiffness, and only dependent on geometrical considerations.

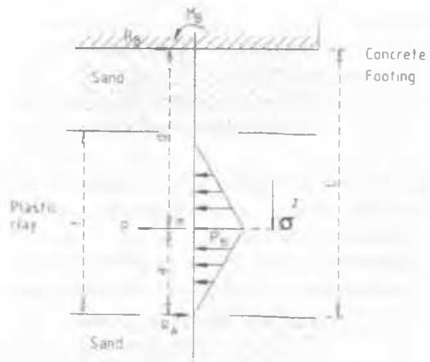

Figure 1 - Loads according to Tschebotarioff

2.1 *Tschebotarioff (1973)*

The design method proposed by Tschebotarioff is based on a first rough estimate by the author and some refinement resulting from field measurements (Tschebotarioff 1973). The author proposed that the horizontal load on a foundation could be described as a triangle, which reaches a maximum value P_H at the center of the soft clay layer (Figure 1).

$$P_H = K_{CE}\sigma_z b \tag{1}$$

where K_{CE} is the coefficient of consolidated equilibrium, equal to 0.4; σ_Z is the increment of vertical stress at the center of the clay layer and b is the width of the pile.

2.2 *De Beer & Wallays (1972)*

These authors proposed a method that considers the slope of the embankment and the relative position of the piles. Results were based on some experimental data. The authors emphasize that their method is safe only if the overall safety factor of the embankment is greater than 1,6. For lower factors of safety, some cases with failure of the piles are presented. Marche & Chapuis (1974) present similar results, regarding horizontal displacements: these authors conclude that for factors of safety below 1,4, the behavior of the displacements change dramatically.

The method proposes P_H as a rectangular load acting on the pile along the soft clay layer. In cases of vertical embankments, for example a bridge abutment, P_H is considered equal to P, the vertical surcharge.

In cases where the lateral surcharge is limited by a slope ε, a reduction factor f is introduced in the calculation of the horizontal pressure. The value of P_H is then given by the expression:

$$P_H = f \cdot P \tag{2}$$

$$f = \frac{\alpha - \varphi/2}{\pi/2 - \varphi/2} \tag{3}$$

where α is the angle of a fictive slope of a fictive height H_f as defined in Figure 2 and the formula below and φ is the apparent angle of friction of the soft clay layer and γ_K is the specific gravity of embankment soil (kN/m^3)

$$H_f = H(\lambda_k/18) \tag{4}$$

The authors emphasize that this semi-empirical method is rough and the only aim is to obtain estimates of maximum bending moments in the piles. Bending moment distributions along the pile are certainly not reliable.

2.3 *Schmiedel (1984)*

The author limits the need for verifications to 3 situations:

- factor of safety is below 1.5, where water content is below 75% and the weight reduction due to fire is less than 15%;
- factor of safety is below 1.8, where water content is higher than 75% and loss of organic material is higher than 15%;
- consistency index is below Ic = 0.25.

The proposed method considers the horizontal loads on piles limited by two different ways: "flow pressure" of the soft clay and earth pressure.

The flow pressure is the maximum pressure that the soft soil can apply on to the pile. This value is limited by a value of 7 to 10 c_u (undrained shear strength of the soft clay).

The maximum earth pressure is defined as the difference between an active earth pressure on one side of the pile and a passive earth pressure on the other side of the pile.

$$e_a = \gamma z + \Delta P - 2c_u \tag{5}$$

$$e_p = \gamma z \tag{6}$$

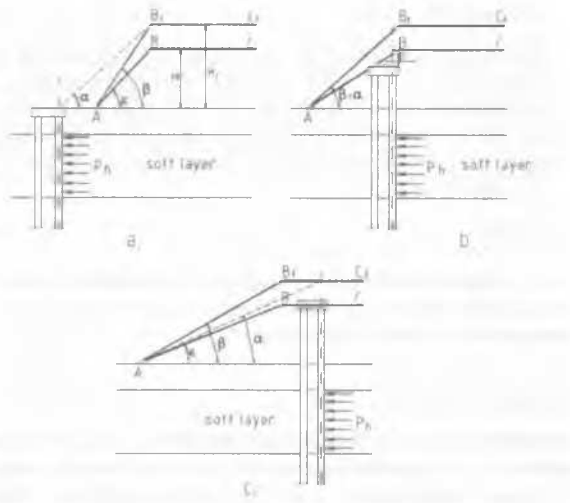

Figure 2 - Geometrical deffinitions, according to De Beer & Wallays

where e_a is the active earth pressure; ΔP is the unsymmetrical surcharge and e_p is the reduced (for safety reasons) passive earth pressure.

The pressure on the pile is defined as:

$$P_H = (e_a - e_p)\,a \tag{7}$$

where a is the mean pile spacing, or 3 times pile width, or thickness of soft soil layer, or total width of pile group, divided by the number of piles.

For design, the lower value between P_H and 7 c_u should be chosen. For cases where the piles are located at a certain distance from the embankment, a reduction of the acting pressures is proposed (Table 1):

Table 1. Proposed reduction of the acting pressures.

Distance L (m)	10-25	10-25	25-40	25-40
Thickness t (m)	5-15	15-30	5-15	15-30
Reduction to (%)	10-20	5-15	5-15	~5

2.4 *Discussion about "Load Methods"*

The three methods assume a symmetrical pressure distribution along the soft clay layer, leading to symmetrical bending moment distributions. Experimental results show different results, with high bending moments close to the pile cap level or at the interfaces between soft and stiff layers (Stewart et al 1994, Heyman 1965).

3 STIFFNESS METHODS

Stiffness methods consider the problem of horizontal loads due to surcharges on the surface a soil-structure interaction problem, where the relative stiffness of the pile in relation to the soil plays a key role. The stiffness methods can be divided into two groups:

- methods of the modulus of subgrade reaction;
- methods that consider the soil as a continuous medium, being linear elastic or not.

The first methods are based on the classical equation:

$$E_p I\, d^4 u_p / d z^4 = K_H (u_p - u_s) \tag{8}$$

where E_P is the elastic modulus of the pile; I is the moment of inertia of the cross section of the pile; u_p is the horizontal displacement of the pile at the depth z; u_s is the horizontal displacement of the soil if at the site of the pile, if no pile is present and $K_H = F(z, u_p - u_s)$ in F / L^2 units

Two main problems arise when using this kind of method in practice: determination of K_H and u_s values. If u_s is obtained theoretically (for example by the theory of Boussinesq), large differences can occur (De Beer 1977).

3.1 *Poulos (1973, 1980)*

This method supposes an elastic soil, characterized by a modulus of elasticity E_S, which may vary along the pile, and a Poisson's ratio ν_S. The horizontal pressure between pile and soil has a limiting value P_Y, which also may vary along the pile.

A solution to the problem is obtained by imposing displacement compatibility between the pile and the adjacent soil. The method requires the knowledge of the free-field displacements of the soil, which depends on the nature of soil, the loading and boundary conditions.

A number of cases evaluating the influence of relative pile stiffness, boundary conditions, soil movement distribution and magnitude and distribution of soil properties along depth are presented.

In terms of bending moments, results of the analysis showed a great influence of pile stiffness, little influence of the boundary conditions with exception of the case with a fixed tip, almost linear relation for different soil displacements magnitudes and little influence of soil property distribution.

For practical use as a design tool, the determination of the soil displacements is not an easy task, and, as said above, has great influence on results. Marche & Chapuis (1974) present an interesting paper about horizontal displacements, showing difficulties in predicting these values. Oteo (1974) presents an interesting discussion of the paper presented by Poulos (1973).

3.2 *Oteo (1977)*

In his paper, Oteo (1977) takes into account the deformability of the pile, like the other "stiffness methods". An interesting concept introduced is the division of piles into stiff piles and flexible piles. This division is defined by following equation:

$$H/\left(E_p I/G\right)^{1/4} \leq 5 \tag{9}$$

where H is the thickness of soft soil layer and G is the shear modulus of the soft soil layer.

For values of the equation above higher than 5, piles can be categorized as flexible.

With respect to design, for stiff piles, the "load methods" give reasonable results, but for flexible piles, methods that use the soil-structure interaction should be used.

3.3 *Goh et al (1997)*

The method proposed by Goh et al is a refinement of the method of the modulus of subgrade reaction: the foundation is modeled as a beam, supported by hyperbolic springs that simulate the complex soil-structure interaction problem. The authors propose a modeling sequence, giving some guidelines to obtain soil-spring data from conventional soil tests, like CPT and pressuremeter tests. Comparisons of the results of published measurement of displacements and bending moments showed relatively consistent when compared to calculated values.

Finally, the authors present a nondimensionalized chart, from where one can obtain bending moments, if the flexural properties of the pile and the elastic modulus and undrained shear resistance of the soil are known. The geometry of the embankment and the position of the pile are fixed. For other geometries, a specific numerical program has to be used.

$$M^{*} = M_{max}/\left(c_u d h_s^2\right) \tag{10}$$

$$K_R = E_P I/E_{50} h_s^4 \tag{11}$$

where d is the pile width and h_s is the thickness of the soft soil layer.

3.4 *Chen & Poulos (1997)*

This method is specific for situations with an unstable upper soil layer, separated by a sliding surface from the lower soil layer. The reasons for the sliding are not discussed: the input data for the method and the proposed design charts are K_R, the thickness of the sliding layer and the displacements of the sliding layer. The authors give some references of how to estimate these values. It is important to notice that this method is not specific for the problem of horizontal loads due embankments.

3.5 *Continuum Models*

Ratton (1985) presents results of finite element tridimensional linear elastic analysis for a case where 2 lines of piles are located close to the toe of an embankment. The results, varying thickness of soft soil layer, spacing of piles, distance to embankment toe, embankment height, pile diameter and properties, and elastic soil properties are presented in a number of charts which can be used for design.

The paper concludes that there can be made a clear division between rigid piles and flexible piles. For the first group, maximum displacements occur at the surface and bending moment distribution has only a single curvature. For the second group, flexible piles, maximum soil displacements occur in depth and bending moment distribution has more than one curvature.

Another conclusion is that the geometrical parameters are much more important than the physical parameters.

Finally, the author concludes that for rigid piles, the load methods are adequate, but for flexible piles, these methods tend to overestimate bending moments.

The ideal way to estimate the behavior of a pile inserted in different soil layers and subjected to embankment loadings is to built a tridimensional model, simulating the constructive process, including pile installation, embankment construction, soil consolidation, non linear behavior of the soil, etc. The variabilities of the different parameters should be considered, preferentially in a probabilistic way. But at least for common design practice, this kind of approach is not viable:

- tridimensional finite element or finite difference softwares are slowly becoming more applicable in daily practice, but this kind of analysis is time demanding and costly. Only for special situations this type of analysis may be viable;
- bidimensional finite element or finite difference softwares are more common and much easier to use, but have a series of limitations, mainly due to the fact that a single pile will be modeled as a continuos diaphragm wall.

Another limitation of numerical continuum models is the problem in choosing adequate reologic model and correct parameters. On the other hand, numerical continuum models have the advantage that it is very simple to simulate different conditions, performing parametric studies. In this way, it is possible to determine the relative influence of the different factors that affect pile behavior. But for everyday practice, numerical continuum models still remain not viable.

4 COMPARISON BETWEEN METHODS

Direct comparison of methods is not a simple issue, because of the different boundary conditions assumed in each method. In the load methods, a P_H value is calculated and applied on a beam model with some boundary conditions on each end of the soft soil layer or structural restraints. But in the stiffness methods, specifically in the Goh et al method, the case for which a design chart is available, maximum bending moments are obtained for a pile with only one formal restraint at his top, fixing rotation. In a simple static model of a beam, this will lead to a hipostatic structure, depending of the boundary conditions.

Comparisons, in terms of bending moments, of the three presented load methods for the specific case presented by Heyman (1965) give the results presented in Table 2, assuming as representative a simple beam model with free support at each end (top and toe of soft soil layer):

Table 2 shows a great dispersion of values, with results of Tschebotarioff's method leading to values lower than the measures values. Using the chart presented by Goh et al, and assuming the same undrained shear resistance, the resulting bending moment for pile II is approximately 60 kNm.

Table 2. Comparison between load methods.

Pile	Tschebotarioff *	De Beer & Wallays**	Schmiedel ◇	Measured
I	2.6 kNm	75 kNm	50 kNm	24 kNm
II	37 kNm	147 kNm	500 kNm	68 kNm

* using Boussinesq equations to obtain an estimate of the vertical stress increment,
** assuming $\phi = 0$
◇ assuming an undrained shear resistance of 20 kPa

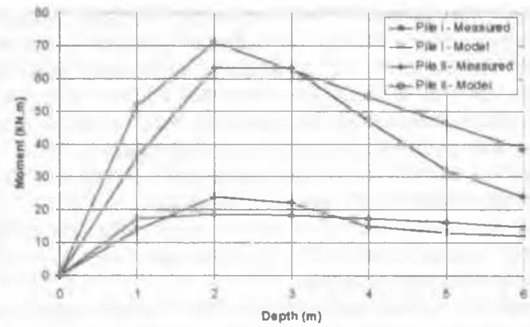

Figure 3 – Measured and calculated values (numerical model)

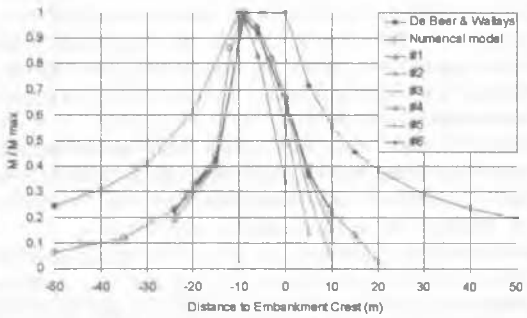

Figure 4 - Influence of embankment width and distance to embankment

5 PERFORMED NUMERICAL ANALYSIS

In practical foundation engineering, several times the real situations differ from idealized design methods. To allow an estimate of influence of two factors considered to have significant influence on bending moments of piles, a simple numerical analysis was performed, varying, for a known case, some geometrical conditions.

The chosen case was the one published by Heyman (1965), which shows results in terms of bending moments for two different locations of piles. For the analysis a bidimensional finite differences program (FLAC 1996) was used. To adjust the numerical model, the results were compared to the published measured values. After adjustment of soil parameters, some geometrical variations were performed to evaluate the influence of the embankment width and relative position of pile.

The adjustment of soil parameters showed an interesting point: intuitively the Poisson's ratio should be high, but the analysis showed best fit with a value of 0.25. An explanation for this value could be the fact that the horizontal loads are associated, at least partially, to an effective stress problem.

Figure 3 shows the bending moments of the numerical model, in comparison to the measured values. Figure 4 shows the results of the analysis varying embankment width and relative position of the pile. The embankment width varies linearly from 7 (#1) to 42m (#6). The bending moments are nondimensionalized dividing the maximum bending moment of each case by the overall maximum bending moment. To allow comparisons, the same figure shows the nondimensionalized bending moments calculated by the De Beer & Wallays method, nondimensionalized by the maximum bending moment of this method. The purpose of this figure is to show the relative influence of embankment width and relative position, and not absolute values.

It can be seen that the greater the distance of the pile to the embankment is, the smaller embankment width influences bending moments. Further, De Beer & Wallays model seems to overestimate bending moments on piles located at a certain distance from the embankment crest. Bending moments of piles located 2H inside the embankment are almost negligible. On the other side, for piles located 2H away from the embankment, the bending moments will be reduced to 20%. The method proposed by Schmiedel (1984) assumes greater reductions.

Results of the analysis should be considered more qualitatively than quantitative: for a comprehensive evaluation, more sophisticated parametric analysis should be performed.

6 CONCLUSIONS

The review of design methods showed that the lack of more real measurements continues to be the main fallacy of the issue concerning horizontal loads due to asymmetrical surcharges.

A number of methods try to consider soil-structure interaction, but for these methods, the knowledge of soil properties continues to be one of the key questions. Assuming linear elastic behavior, a variation of 50% of the relative stiffness, which for the soil type considered is not an exaggerated variation will result, with reasonable accuracy in a variation of 50 % of bending moments.

The concept of stiff and flexible piles, proposed by Oteo (1977), is a valuable design tool, allowing simplifications in case of stiff piles, with a reasonable adjustment between "load methods" and reality. Among the published methods, for stiff piles, the one proposed by De Beer & Wallays (1972) is considered to be a safe and consistent method. For flexible piles, methods like the one proposed by Goh et al yielded for 1 particular case good adjustment to measures. But for practical use, more design charts should be provided for different geometrical configurations.

Finally a qualitative analysis evaluating influence of embankment width and relative position of pile showed that:

- reduction of bending moments is greater for piles inside the embankment than for piles at a certain distance from the embankment;
- the greater the distance from the pile to the embankment, the smaller is the influence of embankment width.

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