Performance and modelling of a friction-piled embankment on a very soft clayey soil

Comportement et modélisation d'un remblai sur pieux flottants sur argile molle

Manuel J. Mendoza, *Institute of Engineering, Universidad Nacional Autónoma de México,* mjm@pumas.ii.unam.mx
Miguel Rufiar, *Institute of Engineering, Universidad Nacional Autónoma de México,* MRufiarJ@iingen.unam.mx
Salvador A. Mendoza, *Institute of Engineering, Universidad Nac. Autónoma de México*, SMendozaM@iingen.unam.mx

ABSTRACT: Construction of linear works on poor ground requires innovative, but reliable solutions. As a possible answer for the runways and platforms of the New International Airport for Mexico City, which is under construction on very soft lacustrine clayey deposits, piled embankments were proposed, among other techniques. All of them were built in a test field. The purpose of this solution is to transmit the embankment loads to deep strata, reducing so the contact vertical stresses on the ground surface. A piled structural embankment has four main components: a competent granular material, piles, pile caps and a basal geosynthetic reinforcement. The embankment loads should be transferred onto the pile caps, through an arching mechanism; and part of the weight falling below the arch is transferred by the reinforcement to the pile caps. The case history of a 2.1 m-height and 30 x 30 m test embankment is described in this paper. It was built on the very soft clayey soils of the ancient Texcoco Lake, near Mexico City. Performance of the system along two years is exposed, including the construction stage, emphazing the arching effect. Numerical modelling of the system is also discussed, comparing their results with those given by analytical solutions and measured values.

RÉSUMÉ : La construction d'ouvrages linéaires sur des sols médiocres oblige à faire appel à des solutions nouvelles mais fiables. Parmi les techniques qui ont été envisagées pour les pistes et les plateformes du Nouvel Aéroport de la ville de Mexico actuellement en construction sur des sols lacustres mous, la technique des remblais sur pieux a été proposée. Comme les autres, cette technique a été testée sur place. L'objectif de cette solution est de transférer les charges du remblai aux couches profondes et de réduire la pression de contact au niveau du sol. Un remblai sur pieux comporte quatre éléments principaux: une couche de répartition de matériau granulaire de qualité, les pieux, les chapiteaux sur chaque pieu et un renforcement géosynthétique. Les charges doivent être transmises aux chapiteaux par effet de voûte et le poids du sol sous la voûte être transmis aux pieux par le renforcement géosynthétique. Cette contribution décrit le cas d'un remblai d'essai de 2.1m de hauteur et 30x30m en plan, construit sur les argiles molles de l'ancien lac de Texcoco près de Mexico. On présente le comportement de ce remblai et les effets de voûte observés durant deux années y compris la construction. Les résultats d'un modèle numérique sont comparés à ceux donnés para des solutions analytiques et aux mesures réalisées.

KEYWORDS: Embankments, geosynthetic reinforcement, piles, soft soils

1 INTRODUCTION

Construction of roads, railways and runways on soft clayey soils requires particular geotechnical intervention, due to the low mechanical properties of the supporting soil. A potential solution is then the use of basal reinforced piled embankments (Van Eekelen 2015). These systems are constituted by a granular soil as embazznkment material, a geosynthetic reinforcement (GR) placed at the base of the embankment, pile caps and point bearing or friction piles. This solution does not have an improvement approach, but that of an actual piled foundation. Indeed, an advantage of this technique is that can be built quickly because it does not require to wait for the consolidation of the subsoil.

The load transfer mechanisms between the different components of the system are described in this paper, and the general criteria for its analysis and design are reviewed, according with different thinking schools. Additionally, a case history of a friction-piled test embankment is described. It consists of a 30 x 30 m, with 2.1 m in height, built on the very soft soils of the ancient bed of the Texcoco Lake, near Mexico City. The site is characterized by thick sediments of highly compressible and very low shear resistance clayey soils, with mean water contents as high as 350%. A conspicuous regional subsidence phenomenon, as well as a strong seismic environment are additional difficult conditions for this site.

2. LOAD TRANSFER MECHANISM

Arching (Terzaghi 1943) in the structural embankment is the main mechanism of this foundation system, through which its own weight and loads on the embankment are transferred directly to the piles, as is depicted as load portion A in Fig 1. This arching mechanism occurs because the area of the pile caps are stiffer than the surrounding portions, attracting a large proportion of the embankment loads, when some settlement of the subsoil and deformation of the GR occur. Due to this arching effect, a dome is developed, resting its base on the piles, and the embankment material that falls below the dome and belonging to the area without pile caps rests on the GR (load portion B) and the underlying subsoil (load portion C). Load B is finally transferred to the pile caps through tensile forces by the membrane effect.

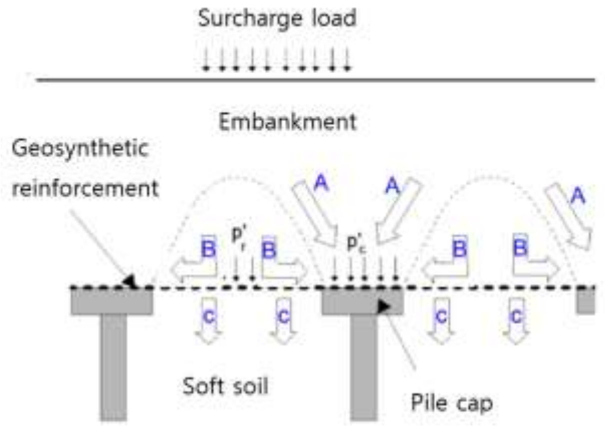

Figura 1. Load transfer mechanism in a piled embankment (Van Eekelen 2008)

2.1 *Conducive elements to the arching mechanism*

The main elements of a piled embankment that are propitious to the arching effect occurence are as follows.

2.1.1 *Rigid supporting elements*

Point bearing or friction piles can be used as supporting elements of the embankment, to which a pile cap should be added. These caps have as an objective to cover a larger area in plant, reducing so the area without support. If no cap is included, it could happen a pile head penetration in the embankment material (Mendoza 2006), considering pile dimensions and relative density of the embankment material

2.1.2 *Fill material*

Fill material for the embankment plays a key role in the system, because it defines the occurrence of the arching effect, transferring the most proportion of vertical loads towards the piles, according with the increase of its resistance properties. That is why a competent granular fill with high resistance to shear stresses must be placed directly on the traditional working platform material. It is recommendable to place a small thickness of fill material on the pile caps, being useful as a cushioning element between the pile caps and the GR. It also help to reduce friction (Gangatharan 2014) between the pile cap surface and such GR.

In the case history that is reported in this paper, a volcanic scoria, locally known as tezontle, has proved in triaxial tests (Mendoza and González, 2016) to be a very convenient granular fill material, mainly to low levels of confining stresses. It is not only a fill material with high values of internal friction angles, but a light weight fill too.

2.1.3 *Geosynthetic reinforcement*

The geogrid is placed in the embankment fill and on the pile caps, and could be uniaxial or biaxial, sometimes using multiple layers, as it is recognized further on. Residual load that is not transferred to the piles through the arching effect, is received directly by the reinforcement. It suffers deformations, generating a catenary shape with tensile stresses between pile caps, transferring so part of the embankment weight to the piles.

As much load is transferred to the GR, less load is applied to the soft subsoil, reducing so the settlement of the system. That is why some GR types with high tensile resistance are adopted as design approach (Van Eekelen 2015). Most design models have proved that the span between piles and the embankment height are important factors for defining the type and required resistance of the GR.

3 DESIGN CRITERIA

3 .1 *Different design approaches*

There are different ways to design piled embankments, being able to distinguish two great schools: American and European. Among the European ones are the German EBGEO (2010) method, the British Standard BS8006 (1995, re-approved in 2010), the Nordic Standard (2004), the French ASIRI (2013) and the Dutch CUR 226 (2010). On the other hand, in America, authors like Collin, Watson and Han (2005) have proposed another criterion

3.1.1 *European and American approaches*

In the American approach, the pressure between pile caps is practically zero, since the GR takes almost all the load caused by the embankment and transfers it to the caps. In addition, it requires rigid concrete piles directly supported on a resistant stratum. Such elements have an enlargement in their head, where at least two layers of GR between friction material rest, which constitutes a sufficiently rigid load transfer platform.

The European approach adopts also the pile caps in order to reduce the span without support. However, they accept that a relatively larger proportion of the embankment load be assimilated by the subsoil between rigid elements, and defining a less rigid load transfer platform. Additionally, the design of piled embankments involves also the use of friction piles (Satibi *et al.*, 2007).

4 ANALYTICAL METHODS FOR THE CALCULATION OF PRESSURES IN THE SYSTEM

4 .1 *Marston´s formula*

This method adopts the British Standard BS8006 (2010), which allows to obtain the pressures A, B and C (Figure 1) as is described below:

The load due to the action A is that one transferred from the embankment to the piles, which is transmitted as a vertical pressure p_c' (kPa) by the soil arching mechanism. Pressure can be assessed as indicated by equation 1:

$$\frac{p_c'}{\sigma'v} = \left(\frac{C_c a}{H}\right)^2 \quad (1)$$

where: p_c' =pressure on the cap; σ'_v=effective vertical stress; a=width of the cap; H=height of the embankment on the GR; and C_c=arching coefficient.

For piles working primarily by friction, the arching coefficient is evaluated by the Marston method [see Spangler and Handy (1973)], as indicated by equation 2:

$$C_c = 1.5\left(\frac{H}{a}\right) - 0.07 \quad (2)$$

The total load (in kN/pile) transmitted by the embankment load, considering a separation between piles equal to s, results:

$$(\gamma\ H + q)\ s^2 \quad (3)$$

where: q=surcharge on the embankment; and γ=unit weight of the embankment material.

The load (kN/pile) that receives the pile caps is calculated with equation 4, assuming square caps:

$$W_c = p_c'\ a^2 \quad (4)$$

For the calculation of the load (kN) that is transferred to the GR, equation 5 is presented:

$$W_r = p'_r\ (s^2 - a^2) \quad (5)$$

Given the balance of vertical forces, with equation 6 we can establish that:

$$p'_r\ (s^2 - a^2) = (\gamma H + q)s^2 - p_c' a^2 \quad (6)$$

and finally, the pressure that receives the GR is calculated with the equation (7):

$$p'_r = \frac{(\gamma\ H + q)s^2 - p_c'\ a^2}{s^2 - a^2} \quad (7)$$

The remaining pressure is transferred to the supporting subsoil. It is noteworthy that these equations do not depend on the internal friction angle of the embankment material. Only depend on the geometry of the piled embankment.

4 .2 *Zaeske´s formula*

With the EBGEO model (2010) it is possible to obtain the A, B and C pressures, which are indicated in Figure 1. For this purpose, Zaeske suggests the following equations (8) y (9).

$$A = A_p \cdot \sigma_{v,p} = A_p \left[\left((\gamma \cdot h + w_s) - \sigma_{v,r} \right) \cdot \frac{A_i}{A_p} + \sigma_{v,r} \right]$$

$$B + C = A_r \cdot \sigma_{v,r} =$$

$$A_r \cdot \left[\lambda_1^{\chi} \cdot \left(\gamma + \frac{w_s}{h} \right) \cdot \left(\begin{array}{l} h \cdot \left(\lambda_1 + h_g^2 \cdot \lambda_2 \right)^{-\chi} + \\ \qquad h_g \cdot \left(\left(\lambda_1 + \frac{h_g^2 \cdot \lambda_2}{4} \right)^{-\chi} - \left(\lambda_1 + h_g^2 \cdot \lambda_2 \right)^{-\chi} \right) \end{array} \right) \right]$$

with:

$$K_{crit} = \tan^2 \left(45^{\circ} + \frac{\varphi'}{2} \right) \qquad \chi = \frac{d \cdot (K_{crit} - 1)}{\lambda_2 \cdot s}$$

$$\lambda_1 = \frac{1}{8} \cdot (s - d)^2 \qquad \lambda_2 = \frac{s^2 + 2 \cdot d \cdot s - d^2}{2 \cdot s^2}$$

where: A_i= area of influence of a pile (S_x * S_y), in m²; A_p= pile cap area, in m²; A_r= A_i-A_p=area of the GR per pile, in m²; s= diagonal distance center to center between piles, in m; d= diameter of the pile or pile cap, in m; $\sigma_{v;p}$= stress on the pile cap, in kPa; w_s= additional pressure on the embankment surface, in kPa; h_g= arc height, in m: h_g=s/2 for h=s/2 or h_g=h for h<s/2; h= height of the embankment fill, in m; J= stiffness of the GR, in N/m²; ε = Unitary deformation obtained from a graph for GR with different stiffness, EBGEO (2010).

Note that equations 8 and 9 include not only the embankment geometry, but the friction angle (ϕ) of the embankment fill too. The properties of the support soil and those of the GR do not influence these first calculations.

5 CASE HISTORY IN THE EX-TEXCOCO LAKE AREA

As part of a test field carried out during previous works for the construction of the New International Airport for Mexico City, a piled embankment was built on the former Texcoco lake. As in the case of the other possible solutions, geotechnical instruments were arranged to, through their monitoring, to know the behavior of their different geotechnical variables.

Some information for this case is as follows: s=4 m, a=1 m, h=2.1 m. 90 friction piles were easily driven without preboring; square shape with 30 cm by side, and L=27 m

5 .1 *Geotechnical characterization of the site*

A very simplified geotechnical characterization of the site is included in Table 1. After a sandy silt very heterogeneous Dry Crust, the thick Upper Clayey Formation corresponds to very soft lacustrine clay deposits; then a thin Hard Layer (relatively talking). Underneath, there is a Lower Clayey Formation, down to 43 m in the site. Then we found granular alluvial sediments and eventually deep and thin additional clayey strata.

Table 1. Simplified stratigraphy of the site

Depth (m)		Material	Point resistence in CPT (MPa)	Water content (%)
from	to			
0	1	DC	0.05 to 1.50	90
1	30	UCF	0.03 to 0.35	100 to 350
30	31	HL	N > 40	30 to 45
31	43	LCF	0.6 to 2	95 a 190

It is to highlight the presence of very thin sandy silt lenses both in the upper and lower formations. Such lenses could serve as drainage layers, expediting the process of consolidating and promoting the vertical deformation increase in the medium term.

5 .2 *Numerical predictions for the calculation of pressures in the system*

In order to reproduce the phenomenon and to understand the variables involved in the arching mechanism, analyses were carried out using a numerical model, making use of a commercial computer program that uses the Finite Element Method (MEF). The geometry used is that shown in Figure 2, which is similar to that of the studied case history.

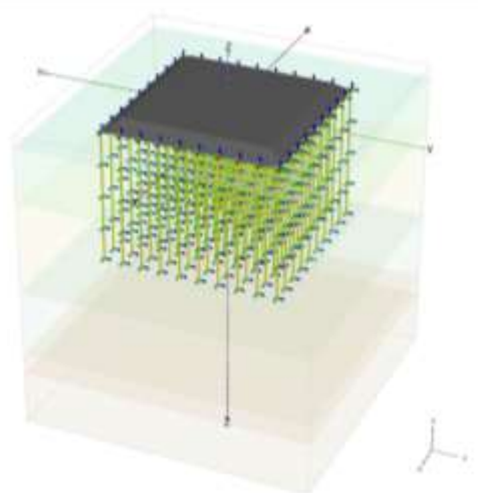

Figure 2. Geometry of the adopted numerical model.

The variable of interest was the internal friction angle. Thus, different parametric analyses were performed by varying this resistance parameter and the pressures were obtained on some pile caps and on the soil in different zones of the studied case, see Figures 3 and 4.

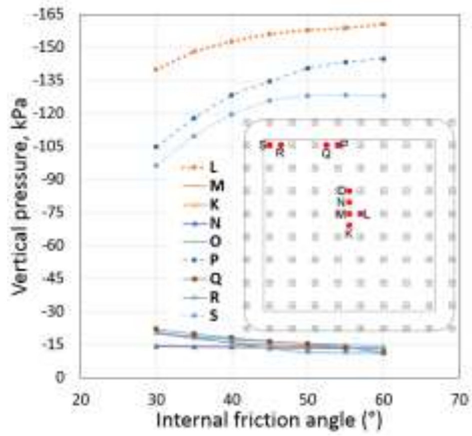

Figure 3. Numerical pressures on the soil and on pile caps

5 .3 *Measurement of total pressures below the embankment*

From the measured variables, this paper only deals with the pressures generated in the different elements of the system, in order to compare the experimental results with the theoretical and numerical predictions exposed in the previous sections. It should be noted that adjacent to the piled embankment was constructed an embankment on rigid inclusions, whose behavior has proven to be similar to that of the piled embankment.

In order to know the distribution of the total pressures under the embankment, eight in total pressure cells were placed on some pile caps, between pile and pile and in the center between piles that form a diagonal, see Figure 4. In these last two cases, the cells were placed just in contact with the natural ground. Monitoring of the cells began since their installation as can be seen in Figure 5. The fill loads began shortly after day 60.

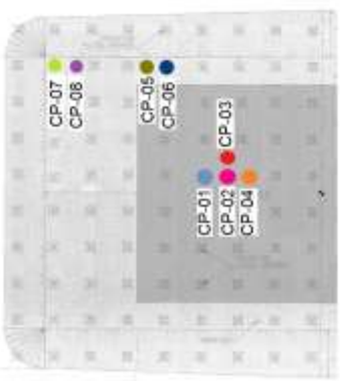

Figure 4. Position of the pressure cells.

Solid line in Figure 5 corresponds to the pressure evolution due to the own weight of the fill and pavement. During the first 30 days, some transient large increments were measured, which are associated to the movements of the construction machinery. The measurements show that the maximum pressures occur on the pile caps, varying from 176 to 222 kPa, while pressures as low as 20 to 40 kPa occur on the ground. These results are direct evidence of the occurrence of the arching effect. An abrupt change in the measured pressures occurred on the pile caps during a seismic event, 128 days after the initial recording.

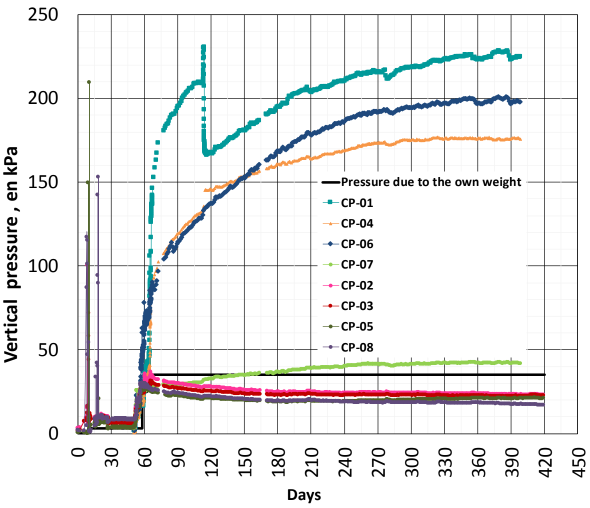

Figura 5. Measurements of the total pressure below the embankment.

6 COMPARISON OF THE MEASURED AND CALCULATED VALUES

A comparison of the measured, numerical (FEM) and analytical values are included in Table 2. It is clear that the Marston solution underestimates the arching effect and gives as a result lower pressures on the pile cap, in comparison with the measured values. The numerical values as well as the Zaeske analytical solution give the closest results to the measured values, although certain underestimate on the pile cap pressure is observed in the numerical result. On the contrary, for this case, Zaeske gives a little larger values than those measured.

Table 2. Measurement and calculated values

	Measurement	FEM	Martson	Zaeske
Groud	20 - 40 kPa	20 kPa	32 kPa	15 kPa
Cap	127 - 22 kPa	120 - 150 kPa	75.3 kPa	200 kPa

7 CONCLUSIONS

The load transfer due to the own weight of the embankment fill and the surcharge load imposed by vehicles or aircrafts requires a compacted granular material, in which an arching mechanism could be developed. Through this phenomenon, most of the embankments loads are transferred to the piles, as a result of the greater vertical rigidity of the piles compared to the space not covered by the pile caps. The weight do not supported by the piles is partially taken up by (1) a GR that through tensioning stresses and the generation of a catenary, finally transmitting that load to the pile caps; directly to the ground. We have shown theoretical solutions to quantify the magnitude of each of the loads involved in the system. From them, it was possible to verify through a case history built in the very soft subsoil of the former Texcoco lake, that the proposal by Zaeske, better predicts the loads on piles and on the ground, than the solution of Marston. Similar values to the measured pressures under the embankment, both on pile caps and on the ground, were obtained in a FEM numerical analysis.

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