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NUMERICAL ANALYZES OF PILED REINFORCED EMBANKMENT WITH GEOSYNTHETIC PLATFORM

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Abstract.*This article presents the analyzes of a typical case of a piled reinforced embankment with geosynthetic platform. The piled embankment technique is used as foundation of embankment on soft soils in order to avoid the settlement arising from the load of soft soils. These types of soils result in problems related to settlements and load capacity. Therefore, the technique allows the load to be conveyed through the pile to a higher shear strain soil layer, preventing the mentioned problems. This work presents results of embankment vertical displacements in the piles projection and between the piles. As project criteria, differential settlement on top of the embankment must be null. It also presentes the distribution of vertical stress created by the embankment load on the joint piles-geosynthetic. The numerical model is based on the finite element method, reproduced through the commercial software ABAQUS. It uses plate elements to represent the geosynthetic and a constitutive model of Mohr-Coulomb to represent the soil.*

Keywords: *Geosynthetic, Piled Embankment, Numerical Analyzes, soft soil*

1 INTRODUCTION

The piled embankment technique is helpful to stabilize embankments over soft soils. It represents an interesting alternative among several techniques mentioned by Almeida and Marques (2013), as Encased Stone Columns, Deep Soil Mixing columns, Geosynthetic Reinforcement, Prefabricated Vertical Drain with Loading, and others. Although these approaches can increase stability, they have limitations with respect to controlling embankment deformations. This can be particularly problematic for embankments that form part of facilities where deformations must be limited (Rowe and Liu, 2015).

The principle of the piled embankment consists in transfer the embankment load to concrete piles and posteriorly to a soil layer with higher shear strength (Riccio et al., 2012). The piles are distributed on square or triangular meshes. The arching of the soil (Terzaghi, 1943) is directly related with the distance between the pile caps and it is calculated in function of embankment height and loading properties (Ehrlich, 1993, Kempfert et al., 2004). According to van Eeekelen et al. (2015), besides to help with arching phenomenon, the geosynthetic contributes with the support of the loading that acts under the formed arch. In this case, one of the follow alternatives must priorized: structure without the geosynthetic reinforced, however with piles located very closer each other and pile caps with bigger dimensions; or structure including geosynthetic reinforced on basal region, what can allow an increase in pile spacing an minimize the size of pile caps. The second structure has been featured as the most economic and effective option. The basal geosynthetic reinforced may be used to bridge across the top of the piles and distribute the embankment loading to the piles (Rowe and Liu, 2015).

Some advantages of the geosynthetic reinforced piled embankment refer to costs, deadline accomplishment and decrease of the settlement (Fonseca, 2017). The technique allows the construction of the embankment immediately after the installation of the piles, pile caps and geosynthetic, differently of other techniques of construction over soft soils, which is necessary to wait months before the embankment construction. On geosynthetic reinforced piled embankment, the settlements are very small, because the most part of the loading is supported by the soil layer localized under the soft soil. Quickness and reduction of settlement makes the work more practical, secure and economic. The installation of vertical drains ally to overload is other technique that is commonly utilized on soft soils regions. It causes an acceleration of the consolidation process and It promotes time gain, however requests the exploration of large soil deposits. Thus, the geosynthetic reinforced piled embankment is more advantageous also in environmental aspects (Giffoni et al., 2016).

The application of geosynthetics in civil engeneering begun in the course of 1990s in Germany (Alexiew et al., 2005). This material has been more and more studied and used in Brazil, since it is a suitable solution to soft soils - commum soils of sedimentary basins and coastal regions.

There are sereval proposals to dimension the geosynthetic reinforcement (Low et al., 1994), (BS8006, 1995), (Kempfert et al., 2004), (van Eekelen et al., 2015). In all that, the calculus is summarized in determine the magnitude of the traction force that acts on the element. The compression resistence of the geosynthetic is despised.

Although the reinforced piled embankments have been widely studied and applied, the knowledge about load distribution and vertical strain on the middle of the massif soil is not totally consolidated. Besides that, there are few researches which investigate the strain suffered by the reinforcement materials near the pile caps (Fonseca, 2017).

2 GEOSYNTHETIC REINFORCED PILED EMBANKMENT

This topic reports some aspects of the geosynthetic reinforced piled embankments, as concept, elements properties, some considerations about the transfer mechanisms and tension distribution.

2.1 Concepts and considerations about geosynthetic reinforced piled embankments

The association of piles systems and geosynthetic reinforcement represents a solution for the construction of embankments over soft clay foundation soils, with low long-term cost, reduction of total and differential settlements and short construction time (Bhasi and Kajagopal, 2015).

The geosynthetic reinforced piled embankment is constituted by an embankment over the geosynthetic. Below, the pile caps and piles are immersed on the soft soil and in contact with a deeper soil layer. Piles to integrate piled embankments can be in concrete, with square, octagonal or circular section, distributed on square or triangular meshes. Their depth are determined in function of the local geotechnical profile which must assure that the piles are supported by a soil layer with higher shear strength. Pile caps or pile heads are located on the top of the piles. Their transversal sections are larger than the piles'. In some cases, there are no pile caps designed – or there are piles and pile caps with the same transversal section area. Then, the geosynthetic may be directly up the piles or on the top of the pile caps. Geosynthetics are materials with high stiffness, high traction strength and low thickness. The embankment is constructed over the geosynthetic layer, with specified design height.

In this type of foundation, the loading is transferred to the base of the embankment, to the geosynthetic platform, to the pile caps, to the piles and finally to the deep soil layer, in this sequence. The geosynthetic function is to redistribute the load, in order to save the soil portion situated immediately below the platform and to direct it to the pile caps.

The geosynthetic platform potentiates the arching phenomenon, which defines that the load transfer prioritizes the higher stiffness materials. Thus, in piled embankments, where there are piles immersed on low bearing capacity soils, the most part of the loading is supported by the piles, because of the difference between the stiffness of the concrete and the soft soil. A small part of the vertical load is directed to the geosynthetic and the soft soil (Fonseca, 2017).

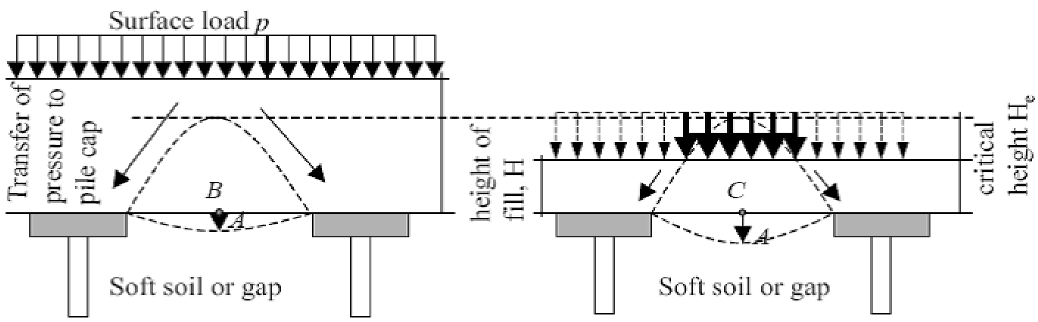


Figure 1. (a) Total arching (b) Parcial arching (van Eekelen et al, 2003).

The figure 1 shows the distribution of the vertical load due the embankment arching phenomenon. The total or parcial arching influences the size of the pile caps, where structures with total arching allow smaller pile caps than structures with partial arching. Besides that, it is possible to observe that the strain between the pile caps - supported by the geosynthetic and the soft soil - changes according to the embankment height and soil grainmetry. The most favorable situation occurs when the embankment is higher than the critical height and the embankment base is constituted by a granular material (van Eekelen et al., 2003).

2.2 Geosynthetic Platform

According to the Brazilian Technical Standard NBR ISO 10318 (2015), geosynthetic is defined as a product which at least one of its components is produced by a synthetic or natural polymer. It can presents blanket, strip or tridimensional structure, utilized in contact with soil and other materials, applied in geotechnical and civil engineering.

The most employed polymer in gesynthetic's fabrication are polyester (PET), polypropien (PP), polyethylen (PE) and polyvinyl alcohol (PVA), made from polymerization chemical processes (Ehrlich and Becker, 2009).

The geotextiles (non-woven) can have high traction resistance and high stiffness, and the woven geotextiles are stiffer than the non-woven geotextile. The strength of geosynthetic reinforcement can be uniaxial or biaxial. A uniaxial material is a material of which the strength in longitudinal direction is much stronger than in cross direction. A biaxial material is a material of which the strength in both the directions is similar (Den Boogert, 2011).

The applied of material is related with the type construction, active loading and terrain features. Among the commercial types, there are geosynthetics with higher and minor stiffness, uni or bi axials, woven or non-woven. It is carried of the engineer the selection of the type and the utilized in one or more layers on the base of embankment.

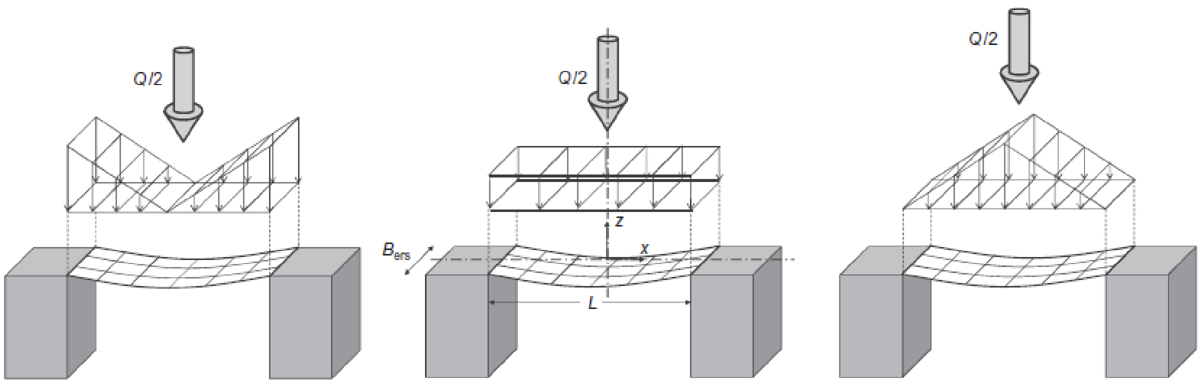
2.3 Vertical load Distribution

The embankment generates vertical load on the geosynthetic platform and on the pile caps. The vertical load distribution is treated with distinct perspectives, according to the method of calculation for reinforced pile embankment and each type of distribution results in different traction force on the geosynthetic and influences the pile caps loading. Kempfert et al. (2004) and van Eekelen et al. (2015) proposed two different models of the load distribution.

The calculation model presented by van Eekelen et al. (2015) is based on a triangular distribution of vertical load, with major values on the top of piles and minimum values on the between piles region. Thus, the logical conclusion is that the major strain values occur on the top of the pile caps and the minimal strain values are located on the between piles region, where the geosynthetic is in contact with the soft soil. van Eekelen et al. (2015) emphasizes that as much as the geosynthetic is in contact with the soft soil, which the soft soil contributes with the geosynthetic through a reaction force, the load distribution tends to be rectangular.

Kempfert et al. (2004) adopted a triangular load distribution, with maximal vertical strain on the between piles region and minimal vertical strain on the top of the pile caps. The Kempfert et al. (2004) model expects that the major vertical strain is verified on the middle of the between caps, on the base of the embankment.

The figure 2 presents the vertical load distribution according to (2004) and van Eekelen et al. (2015) models.



(a) (b) (c)

Figure 2. (a) and (b) Models proposed by van Eekelen et al. (2015). (c) Model proposed by Kempfert et al. (2004). Figure extract of Blanc et al. (2014).

2.4 Design Considerations

A central question to design a reinforced piled embankment, even to design many projects in engineering is the differential settlement occurrence. The maximal differential settlement in piled embankments is verified comparing two points located on the top of the embankment. One of them is positioned on the piled vertical projection and other is situated on the between piles projection. To determine the embankment, height is necessary to consider the differential settlements on the top of the embankment. It is recommended to assume dimensions that result in null differential settlement on the top of the embankment. The embankment height which ensures the condition of null differential settlement is named critical height (McGuire et al, 2012).

Many studies determine the critical height of the embankment using the variables *'s'* (centre-to-centre distance between piles) and *'a'* (lateral dimension of pile caps) through the multiplication of the factor *(s – a)* to a certain coefficient. McGuire et al. (2012) reports, on a table, the critical height dimension according to some authors.

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Table 1. Critical height according to some authors (McGuire et al, 2012).

Author	Critical height
British Standard, BS 8006 (1995)	0.7 (s - a)
Carlson (1987)	1.0 (s - a)
Nordic Handbook (2002)	1.2 (s - a)
Chen et al. (2008)	1.6 (s - a)
Demerdash (1996)	1.7 (s - a)
Hewlett and Randolph (1988)	2.0 (s - a)

s represents the space of centre to centre between the piles.

a represents the lateral dimension of the pile cap.

McGuire et al. (2012) presents valuable concepts for the piled embankments design, as unit cell, centre-to-centre distance of piles and pile caps sizes. The McGuire's method is based on the following equations:

$$H_{crit} = 1.15s' + 1.44d \qquad (1)$$

$$s' = \frac{\sqrt{(s_1^2 + s_2^2)}}{2} - \frac{d}{2} \qquad (2)$$

$$s' = \frac{\sqrt{(2s_1^2 + s_2^2)}}{3} - \frac{d}{2} \qquad (3)$$

Equation (2) is used for square meshes of piles, equation (3) is used for triangular meshes.

Where:

Hcrit = critical height

s_1 and s_2 = centre-to-centre distances between the piles

d = pile cap diameter (for a circular pile cap) or equivalent side of a square pile cap - with $a(side) = 0.886d$.

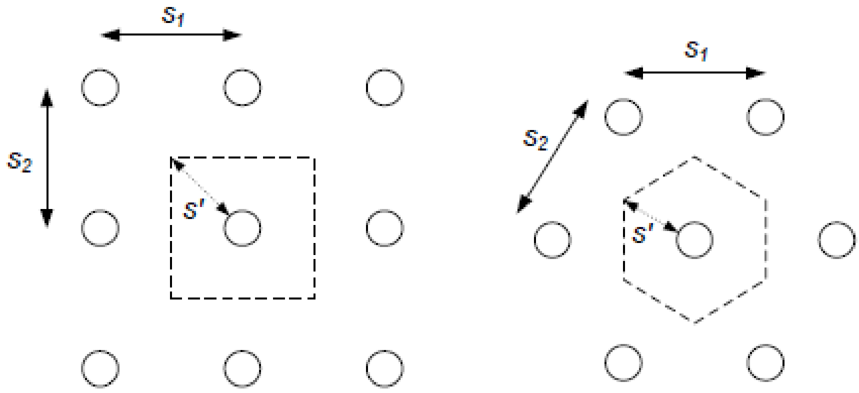

Figure 3. s_1, s_2 and s' values for triangular and square meshes (McGuire et al., 2012).

There are standards to guide the reinforced piled embankment project, as the Brithish Standard BS 8006 (2010), the German Standard EBGEO (2011) and the Dutch Standard CUR 226 (2015).

2.5 The Reinforced Piled Embankment Construction

The implantation of the structure is proceed in stages. Initially, it is common to install a soil layer with better geotechnical properties, with 0.5m of height, named working platform. This layer facilitates the machines activity and the pile driving, which is difficult on soft soils due their low bearing capacity. Afterwards, the piled are drived and the pile caps are located on the ground level. Then the geosynthetic platform is placed and the embankment is constructed.

The working platform contributes with a reaction force on the geosynthetic, easing the requested traction of the reinforcement. However, dealing with soft or very soft soil as foundation soil, the working platform can suffer settlement, resulting in detachment between the geosynthetic and the working platform soil. In this case, the working platform no longer acts decreasing the geosynthetic traction force (Giffoni et al., 2016)._The studied model is based on the described structure, as can be observed at figure 4. Intending to simplify the calculation, the work platform is not computed.

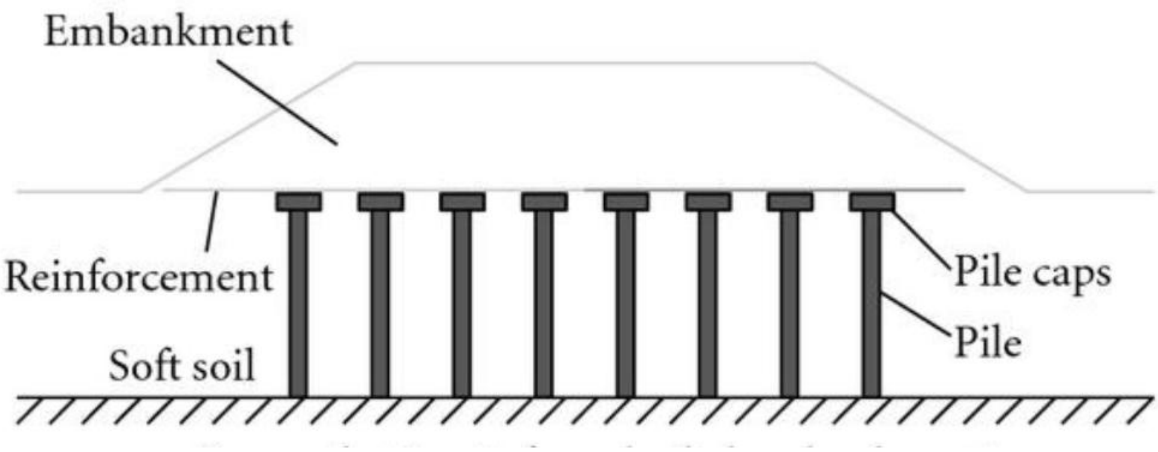

Figure 4. Geosynthetic Reinforced Piled Embankment (Jennings and Naughton, 2012).

2.6 Numerical Model

The 3D numerical model was developed through the software Abaqus version 6.11. It is constituted by the elements: soft soil, piles, embankment, and geosynthetic. The structure's geometry can be observed in Figure 5.

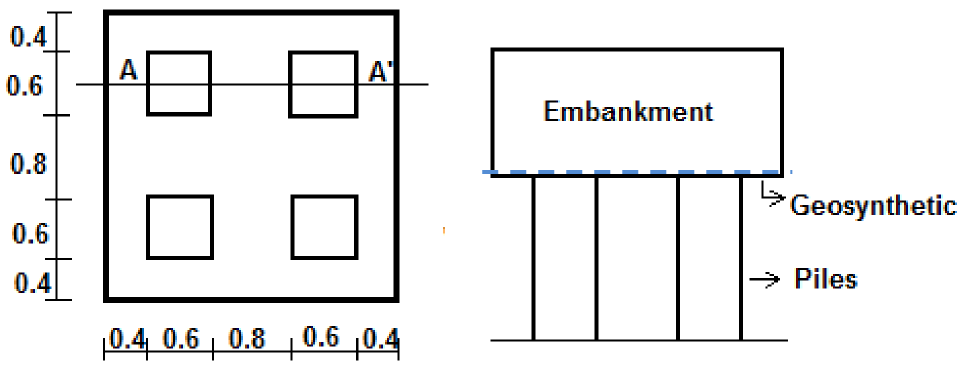

Figure 5. Structure's geometry, plan and section views.

The portion studied is formed by soft soil, with transversal section measuring 2,80m x 2,80m and 8m of depth. Four piles with square section (0.60m) are immersed on the soil. The layer of soft soil is localized over an unaltered rock. Thus, the both clayey soil and the four piles are in contact with the rock. The geosynthetic material has despicable thickness, installed between the embankment and the soft soil (under the embankment, over the soft soil). The embankment is the most external layer, with 1.72m of height.

The piles and soft soil are represented by merge. Then, these elements constitute one instance, with partitions formed by different materials. The embankment also is subdivided in partitions. It is represented by 8 layers, on top of one another. With the porpoise to obtain a better behavior simulation, the geosynthetic is modeled as a shell planar and the other, as a solid extrusion.

The software utilizes the finite element method to compute the structure. Therefore, it is necessary the creation of a mesh to divide the elements to smaller pieces. In this model, each node is separated by 0.2m in piles, soft soil, embankment and 0.1m in geosynthetic.

All the materials are determined as linear-elastic, except the embankment, which is modeled as a plastic material, according to Mohr Coulomb Plasticity Theory, with friction angle (ϕ') of 30º, dilatation angle (ψ) of 4º, cohesion (c') of 3 kPa. In the table 2, it can be observed more characteristics of the elements, as modulus of elasticity (E), Poison Coefficient (υ) and specific weight (γ).

Table 2. Parameters attributed to materials

Parameters/ model	Embankment	Soft Soil (Clay)	Piles (concrete)	Geosynthetic (basal reinforcement)
E (kPa)	$1 \cdot 10^4$	$5 \cdot 10^2$	$2.1 \cdot 10^7$	$2 \cdot 10^7$
υ	0.33	0.47	0.20	0.20
γ (kN/m³)	18.5	14.0	25.0	0.20
ϕ' (°)	30	-	-	-
c' (kPa)	3	-	-	-
ψ (°)	4	-	-	-
Model	Mohr-Coulomb	Linear-elastic	Linear-elastic	Linear-elastic

The two geosynthetic faces – that is in contact with the embankment and that is contact with the piles and soft soil merged – are typed as surface-to-surface, with shearing and normal behavior and friction coefficient μ=0.8. This representation allows relative displacement between the elements, what can approximate the model to the real situation.

The boundary conditions are: encastre in the basal region (inferior surface, formed by soil and four piles – z direction) and zero displacement in directions x and y. These considerations are related to the fact that the piles are laid in firm stratum. The zero displacement in x and y directions is motivated by the fact that a portion of soil analyzed are involved by other portions submitted to similar conditions, all side by side.

Only gravity acts on the model, thus the structure is supporting no more than its own weight. It is considered that the clay soil is already stable with respect to its own weight, so, only the embankment and geosynthetic specific weights are used.

The model is run with the initial plus 8 steps. Each step consider the addition of one more layer of the embankment, beginning with step 1 (before the initial step), with the inferior layer and finishing with the last step, with 8 layers of the embankment. This method intends to simulate the real conditions of embankment construction.

To contribute with calculus convergence and because of the geometrically non-linear contact problem, it is used an increments of 0.01 (1%), that is applied on the first step and propagated to the others. The 'non-linear geometrically' option should be connected all long the analysis, due the friction contacts and the shell element modeling.

A similar model is run, which reproduces the same hypothesis and methods, however without the geosynthetic, with the pretense of to compare the results and conclude about the reinforcement contribution.

3 RESULTS AND DISCUSSION

In this item will be presented the main results obtained in the simulation of the piled embankment. The results will be presented in terms of displacements and stresses in the structure.

3.1 Vertical stresses on soil mass and piles

The figure 6 shows the distribution of vertical stress (S33) acting on piles and embankment. From Figure 6 it is possible to observe that over the pile top there is a vertical stress (S33) concentration. This vertical stress concentration occurs because the piles are stiffer than the surrounding soil. For example, the vertical stress at the pile top and center is equal to approximately 70 kPa while the vertical stress at embankment base would be 31,8 kPa (1,72m x 18.5 kN/m³) in the absence of difference in stiffness of materials (soil and piles). So, for pile design purposes the force in the top of the pile is 70 kPa multiplied for the top pile area and not the unit weight of soil multiplied by the embankment height.

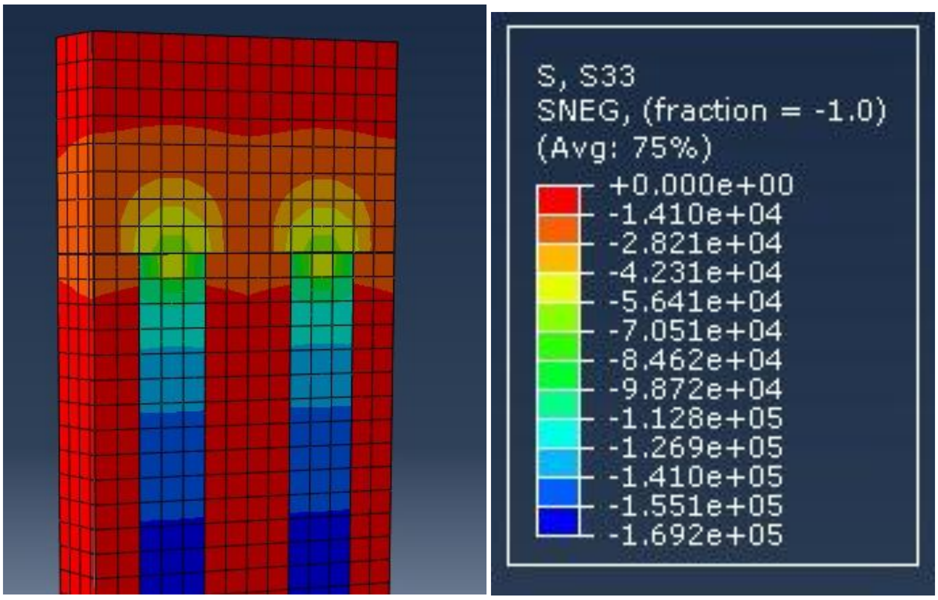

Figure 6. Vertical stress (S33) acting on piles and embankment – cross view.

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The shape of vertical stress (S33) distribution is closer to the load distribution considered by van Eekelen et al. (2015), Figure 2a, and far away to the load distribution proposed by Kempfert et al. (2004), Figure 2c. In this way the approach considered by van Eekelen et al. (2015) seems to be more appropriated to represent the load acting on piles and geosynthetic.

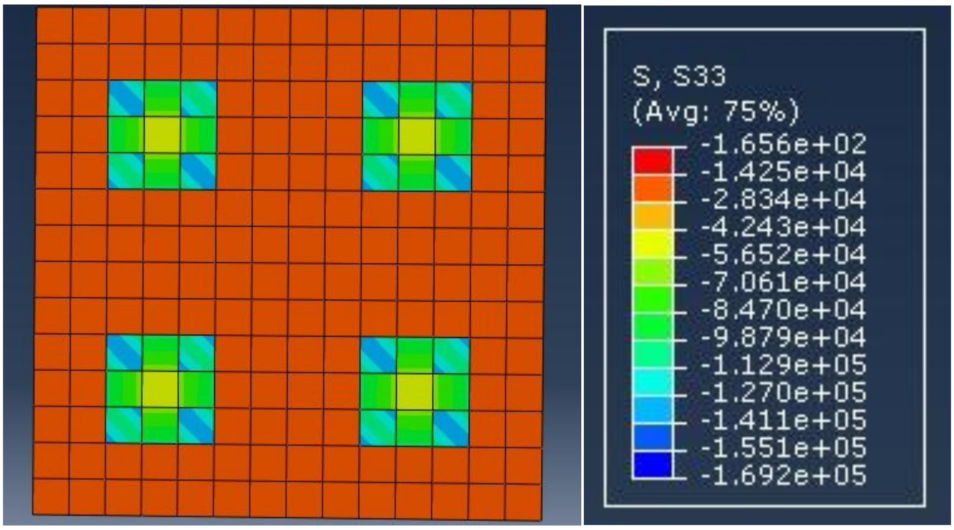


Figure 7. Vertical stress (S33) acting on piles and embankment – plan view.

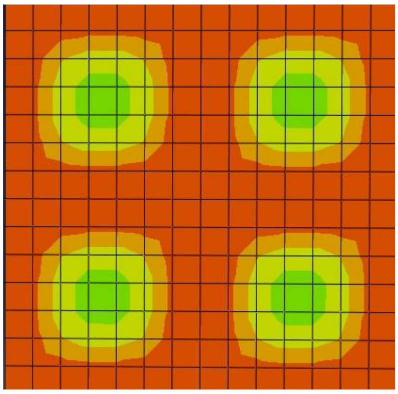
The figure 7 shows the vertical stresses (S33) on the level of top piles. Not that the higher stresses occurred in the border of top piles. So, to prevent damage in the geosynthetic the EBGEO (2011) recommends that the border of top piles would be smoothed.

The figures 8a, 8b and 8c exhibits the verticals stresses in plan view at the base of embankment, at 0.30m above base of the embankment and 0.90m above the base of the embankment. From figure 8 is possible to observe that there is a concentration of vertical stresses over the piles and the opposite effect between piles.

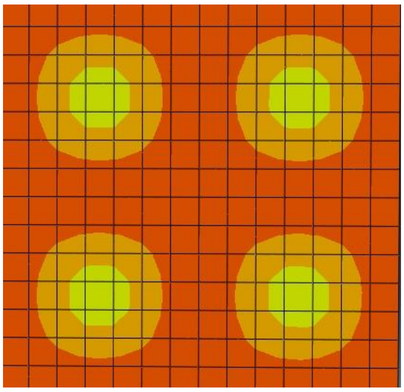
Considering a point just over the pile center and base of embankment (Figure 8a), the vertical stress (S33) is about 70 kPa, that is higher than the geostatic vertical stress (S33) given by the height of embankment multiplied by the unit weight of soil embankment, i.e., 31.8 kPa. On the other hand points located in the middle space between piles and at the base of the embankment shows of vertical stress (S33) varying from 28 to 42 kPa. So, the vertical stress can be lower than the geostatic vertical stress imposed by the embankment.

Figure 8b shows the vertical stresses for a plan located 0.30m over the base of the embankment. This Figure exhibits similar behavior showed by Figure 8a, but the difference between vertical stresses just above the piles and between piles are smaller than the difference presented I Figure 8a.

Figure 8c present the values of vertical stresses for a plan located 0.90m from the base of embankment. In this condition the vertical stress between piles (in diagonal) is still smaller than the geostatic vertical stress. The opposite effect occur in the region above the piles.



(a)



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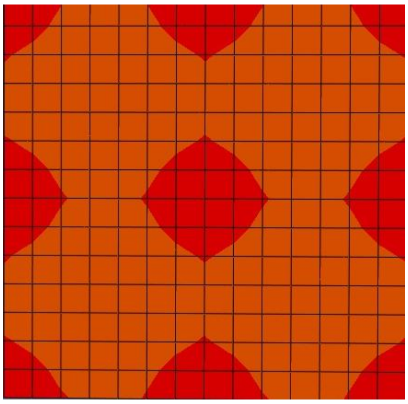

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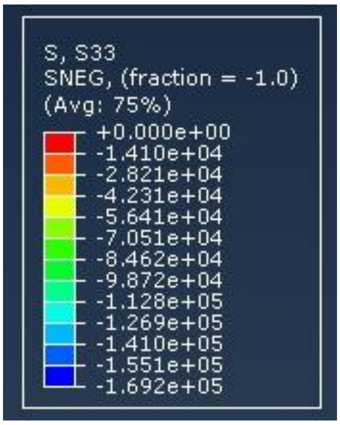


Vertical stress: color scale

Figure 8 (a) Vertical stress (S33), plan view and level of embankment base. (b) Vertical stress (S33), plan view and level of 0.30m above embankment base. (c) Vertical stress (S33), plan view and level of 0.90m above embankment base.

The vertical displacements (U33) are shown in Figure 9 in cross view layout. The results of vertical displacements show that the displacements on the piles are practically nulls and the higher vertical displacements occur between piles. Thus, differential settlement of embankment takes place near the base of the embankment, i.e., considering the same embankment height (for example 0.20m) the difference between the vertical displacement above the center of the pile is smaller than the vertical displacement in points located between these ones.

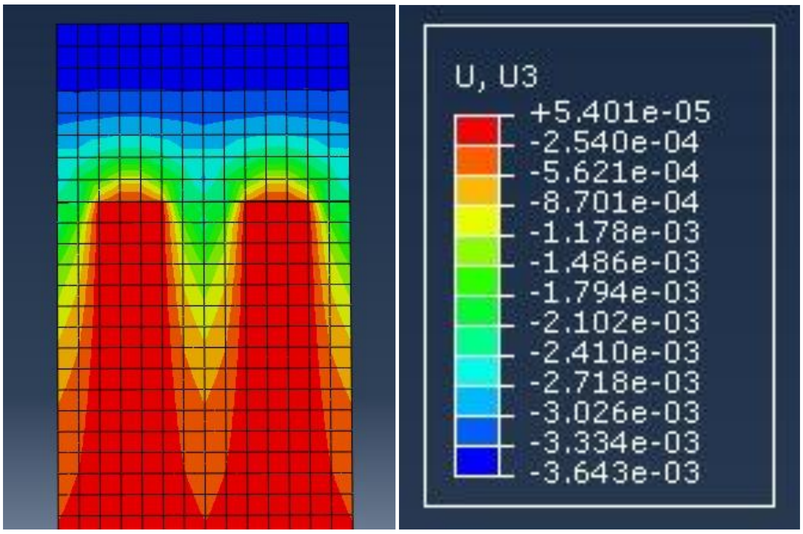
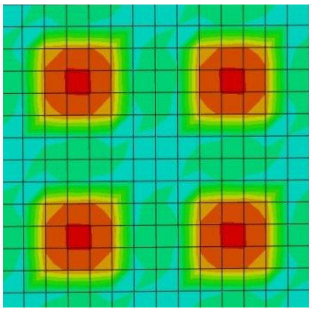
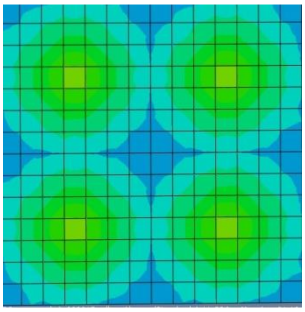


Figure 9. Vertical displacements (U33), cross view.

The Figures 10a, 10b, 10c and 10d also present results regarding vertical displacements (U33), but in plan view. Four elevations, in relation to the base of embankment, were considered: 0.0m (base), 0.30m, 0.60m and 0.90m.



(a)



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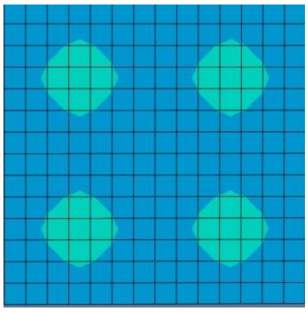

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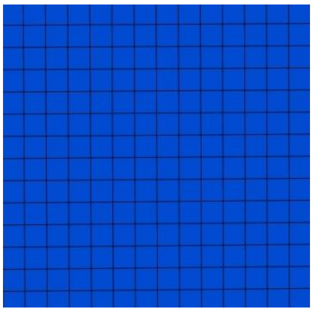
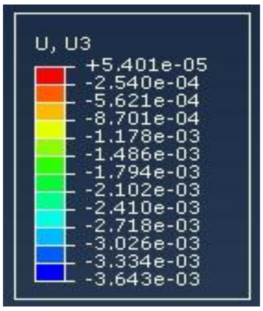


(d)



Vertical displacement: color scale

Figure 10. (a) Vertical displacements (U33), plan view, base of embankment. (b) Vertical displacements (U33), plan view, 0.30m above base of embankment. (c) Vertical displacements (U33), plan view, 0.60m above base of embankment. (d) Vertical displacements (U33), plan view, 0.90m above base of embankment.

The Figures 10a and 10b shows the vertical displacements for planes crossing the embankment at a 0.0m and 0.30m above the base. From these Figures is possible to observe that the differential settlement is greater for points just above the center of pile and points in the center of piles (middle of diagonal passing trough axes of piles). This differential settlement is greater than the differential settlement given by points located just above the pile and adjacent to the piles (between piles). Considering a plan 0.60m (Figure 10c) above the base the differential settlement is almost the same considering the middle of diagonal and the middle of adjacent piles. From Figure 10d and points located above 0.90m from base the differential settlement is negligible.

4 CONCLUSIONS

This paper presents results regarding a numerical modeling of a piled embankment reinforced with a basal geosynthetic. The main conclusions obtained are the following:

- The vertical stresses just above the piles and between piles are different from the geostatic vertical stress. The stresses just above the piles are greater than the geostatic vertical stress and between piles the vertical stress can be smaller than the geostatic vertical stress;

- The load distribution of vertical stress at base of embankment presents more similarity with the model proposed by van Eekelen et al. (2015) than the model proposed by Kempfert et al. (2004);

- The differential settlement between points located just above the piles and between piles is higher near the base of embankment and tends to be null for heights of embankments closed to the critical height as proposed by McGuire et al. (2012);

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