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NUMERICAL ANALYSES OF PILED EMBANKMENT: EFFORTS ON CONCRETE PILES UNDER ASYMMETRIC LOADINGS

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Abstract: *Constructions of embankment over soft soils demands solutions to elimination or reduction of settlements and prevent foundation failure due to the external loadings imposed by embankments. A solution here analyzed is the use of piles, pile-caps and geosynthetic. The piles are normally designed accounting only axial effort. However, for situations where there is an asymmetric loading significant bending moment and shear efforts can arise, leading to the failure of structural element, as can be seen in real situation. Several authors (Tschebotarioff, 1973; De Beer e Wallays, 1972) propose analytical solutions to evaluate those efforts. These, however, have some limitations that can be related to geometrical issue or even the use of empirical factors as observed in Tschebotarioff formulation. In this paper, parametric analyses with variations of adjacent load, height of embankment and stiffness modulus of geosynthetic, using PLAXIS software to analyze a hypothetical pile embankment, was done. It was observed that the embankment generates shear and moments on piles and the asymmetric loading can act in way to modify the magnitude of these efforts. The parametric analyses showed that this magnitude depends on the embankment loading, lateral loading and is not significantly modified by the stiffness modulus of geosynthetic.*

Keywords: *Piled embankment, efforts on piles, soft soil, numerical analyses, asymmetric loading*

1 INTRODUCTION

According to Almeida and Marques (2013) the piled embankment is nowadays a reliable alternative to construct embankments over soft soils. This technique prevents problems related to settlements and failure of soft soil foundation. Analytical methodologies to design of piled embankments can be found in Ehrlich (1993), Kempfert et al. (2004) and van Eekelen (2015), for example. The technique consists in drive rigid piles, couple caps on piles top and use geosynthetic to cover the space between piles supporting the soil over the piles/caps spam. Normally the piles are driven in triangular or square mesh pattern and the geosynthetic is often a geogrid. The figure 1 shows schematically the elements that compose an embankment supported by pile/cap/geogrid system.

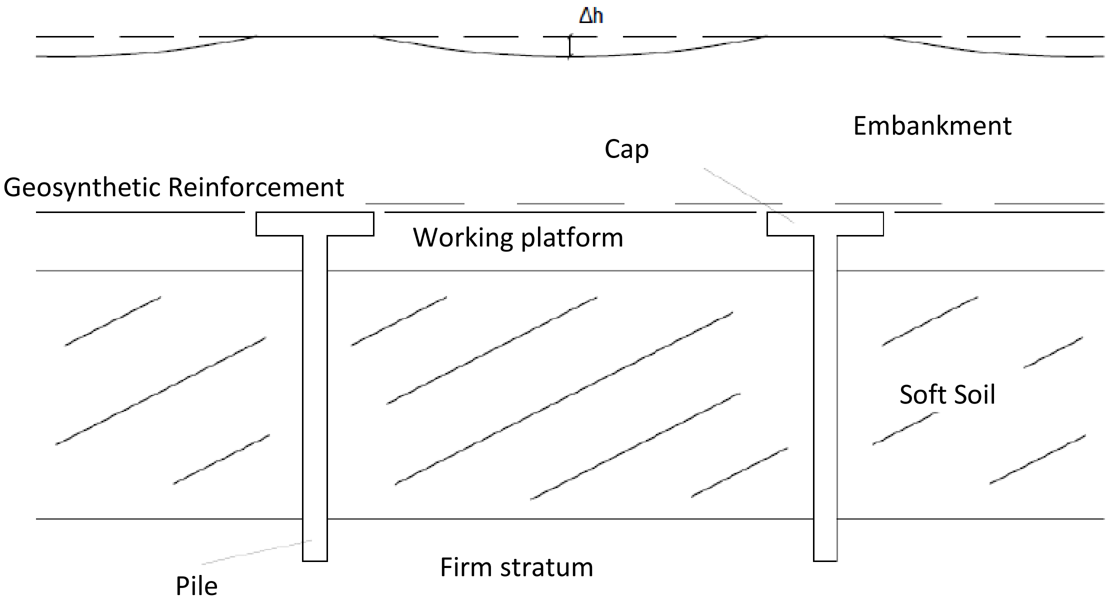


Figure 1 – Concept and elements of piled embankment.

To prevent the differential settlement (Δh in Figure 1) on embankment top is necessary that the height of embankment be greater or at least equal to critical height hcr. The critical height hcr is determined as function of caps size and spacing between piles axes. The critical height can be calculated according to the methodology presented by McGuire et al. (2012). Embankment height greater than hcr allow the full soil arching formation, preventing the differential settlement (Δh in Figure 1).

Normally, the piles are dimensioned with respect of vertical stress on cap caused by embankment. Due difference of soil-pile/cap stiffness, the vertical stress on pile/cap is greater than vertical stress on soil.

Despite of axial stress is normally the major force acting on plies is important to verify the horizontal efforts that appear when there are any loading aside (asymmetrical loadings) the piled embankment. Horizontal efforts (bending moment and shear forces) can lead the structure to fail. There are some analytical formulations to calculation of horizontal force acting on piles of piled embankment. Examples of these formulations are presented by Tschebotarioff (1962, 1970, 1973), Wenz (1963), De Beer e Wallays (1969, 1972), De Beer (1972). Additional studies can be found in Aoki (1970), Marche e Lacroix (1972) among others.

In the presented study is presented numerical parametric analysis considering variation of the magnitude of asymmetric loading aside the piled embankment, height of embankment and stiffness modulus of geosynthetic (geogrid). The numerical calculations were done using finite element method by means of PLAXIS 2D v.8.2 program.

2 GEOMETRY OF STUDIED CASE OF PILED EMBANKMENT

The geometry of hypothetic studied case was choice seeking reproduce dimensions of the structure often used in real works (Fagundes et al. 2015). In the Figures 2 and 3 are shown the geometry of piles and caps and the complete structure, respectively.

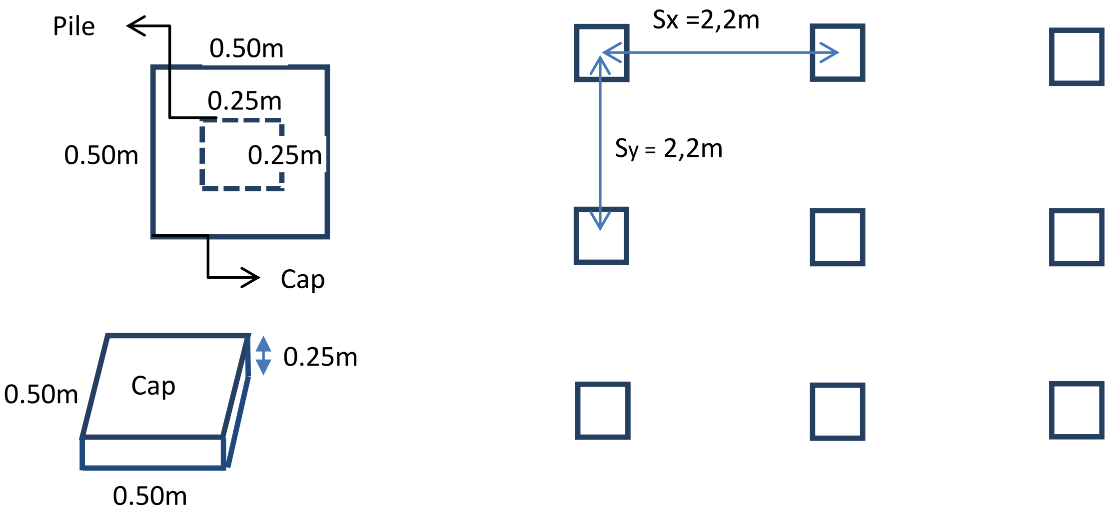


Figure 2 – Square mesh pattern and geometry of piles and caps.

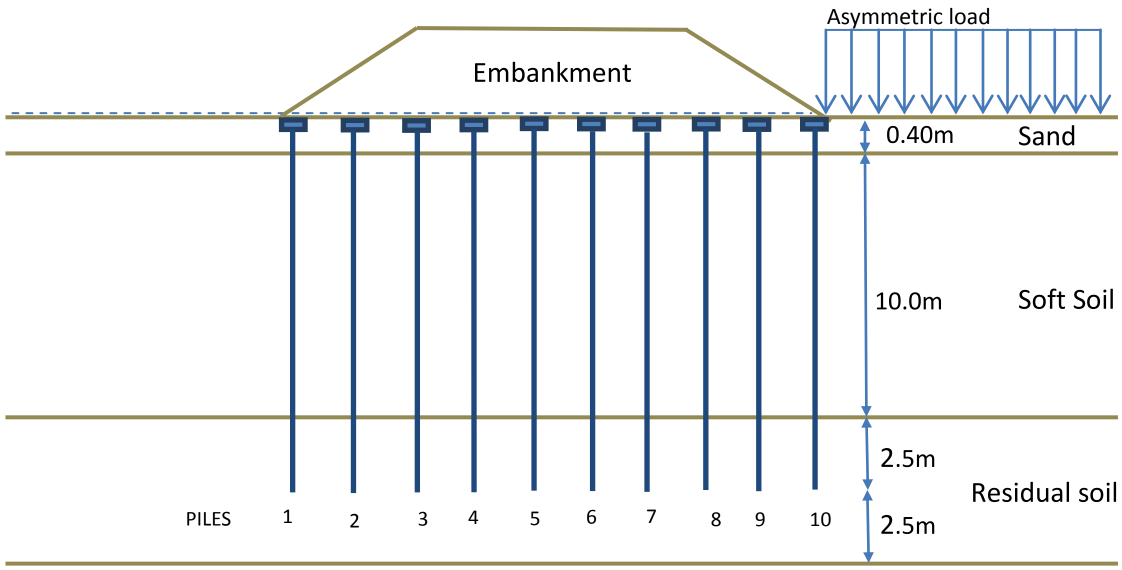


Figure 3 – Complete structure of piles embankment (2D representation).

The geometry of piled embankment accounts for the presence of a working platform just above the soft soil. The working platform is a thin embankment with 0.50m thickness constructed to allow heavy equipment to operate on construction site. This layer is constructed using often granular soils as sand. Heavy equipment cannot be supported directly on soft soil due to lack of bearing capacity of this kind of soil. The geogrid is placed just above the piles to support the soil above this one. The caps were considered enchased on piles, so the caps cannot rotate over piles. The water table (WT) was considered in position coincident with top of soft soil.

As the program PLAXIS 2D works in two dimensional condition (2D), a transformation of real condition 3D to 2D was performed. This transformation was explained in the item 3.

3 MATERIAL PROPERTIES

The Table 1 presents the properties of soils and the constitutive model utilized for each one as well. In this Table is possible to note that for soft soil was used the SSM model (Soft Soil Model) that accounts for consolidation of soil with respect to time. For the other soils it was used the Morh-Coulomb constitutive model.

The results presented in terms of bending moments and shear forces are related to condition near final consolidation, i.e., when the excess of pore-pressure is 2.0 kPa. The other soil materials (Embankment, sand and residual soil) are not subject to consolidation process and were modeled as drained material.

For all soils, except the soft soil, it was considered, regarding permeability, that the soils are isotropic, i.e., the vertical (kv) and horizontal (kh) permeability are equal. For the soft soil it was considered a typical relationship kh = 1.5·kv. Values of cc (compressibility coefficient), cs (swelling coefficient) and e0 (initial void ratio) are typical of clays found in Rio de Janeiro city (Almeida et al. 2008).

Table 1 – Material properties of soils and constitutive models utilized

Material / Property	Embankment	Sand	Residual Soil	Soft Soil
γ_n (kN/m³)	19	20	18	14
γ_{sat} (kN/m³)	21	22	20	15
E (kPa)	30.000	20.000	25.000	-
ν [-]	0.30	0.30	0.30	-
c_c [-]	-	-	-	1.329
c_s [-]	-	-	-	0.125

e_0	-	-	-	2.756
c' (kPa)	3	0	10	3
ϕ' (°)	30	35	28	25
Model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	SSM
Condition	Drained	Drained	Drained	Undrained
k_v (m/day)	$1{\cdot}10^{-2}$	1	$1{\cdot}10^{-2}$	$1{\cdot}10^{-4}$
k_h (m/day)	$1{\cdot}10^{-2}$	1	$1{\cdot}10^{-2}$	$1.5{\cdot}10^{-4}$

Note: γ_n = unit weight of soil; γ_{sat} = saturated unit weight of soil; E = Young´s modulus; ν = Poisson coefficient; c_c = compressibility coefficient; swelling coefficient; e_0 = initial void ratio; c’ = effective cohesion; ϕ’ = effective friction angle; k_v = vertical permeability; k_h = horizontal permeability.

Table 2 presents the properties of structural elements (piles and caps) and property of geogrid (geosynthetic). To represent the real tri-dimensional condition (3D) using the computer program that works in two-dimensional (2D) it was necessary to calculate the equivalent values for EA and EI (E is the Young modulus of concrete, A is the cross section of structural element and I is inertia moment of the structural element). These equivalent values are given by EAeq = EA/S and EIeq = EI/S, where S is the spacing between axes of the piles. According to Figure 2 the value of S is 2.20m. So, in the program the pile is represented by an equivalent wall with reduced values of EA and EI. The caps received the same treatment, observing that the pile in the program is represented by a continuous slab.

To perform the parametric analysis the value of J (stiffness modulus of geosynthetic) was varied. The used values were 1000 kN/m, 2000 kN/m and 3000 kN/m (Table 2).

Table 2 – Material properties of structural elements and constitutive models utilized.

Material / Property	Pile	Cap	Geosynthetic (geogrid)
E (GPa)	21	21	-
EA (kN)	$1.31{\cdot}10^{6}$	$5.25{\cdot}10^{6}$	-
EI (kNm²)	$6.84{\cdot}10^{3}$	$1.36{\cdot}10^{4}$	-
EA_{eq} (kN)	$5.95{\cdot}10^{5}$	$2.38{\cdot}10^{6}$	-

EI_{eq} (kNm²)	$3.11 \cdot 10^3$	$6.18 \cdot 10^3$	-
ν[-]	0.20	0.20	-
J (kN/m)	-	-	1000, 2000, 3000
γ (kN/m³)	25	25	-
Model	Linear-elastic	Linear-elastic	Linear-elastic

Note: γ = unit weight of concrete; E = Young modulus of concrete; I = inertia moment of structural elements; ν = Poisson coefficient of structural elements; J = stiffness modulus of geosynthetic (geogrid).

4 NUMERICAL MODELS AND PERFORMED ANALYSIS

The numerical model and its finite element mesh are showed in Figure 4. The piles were numbered from 10 to 1, where the pile 10 is the closer one to the asymmetrical load and the pile 1 the thither from asymmetric loading. Parametric analyses were performed varying the following variables: height of embankment, stiffness modulus of geosynthetic (geogrid) and asymmetric load. In total, nine analyses have been performed and each analysis provided values of bending moment and shear forces on piles. The finite element mesh has 10763 nodes and each element 15 nodes.

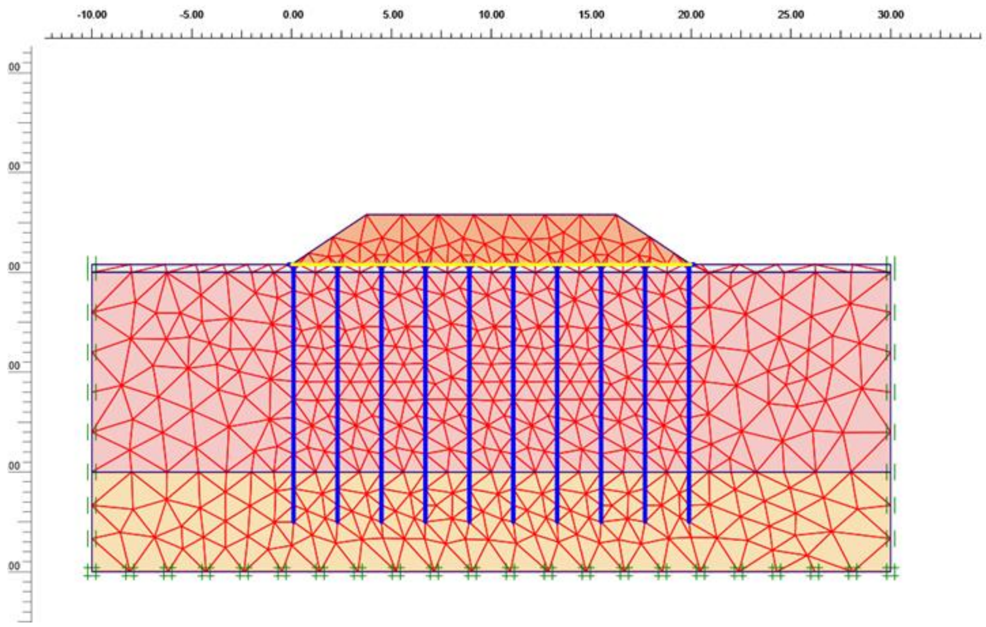


Figure 4 – Model and finite element mesh of piled embankment in plane strain condition.

Table 3 – Characteristics of analysis performed.

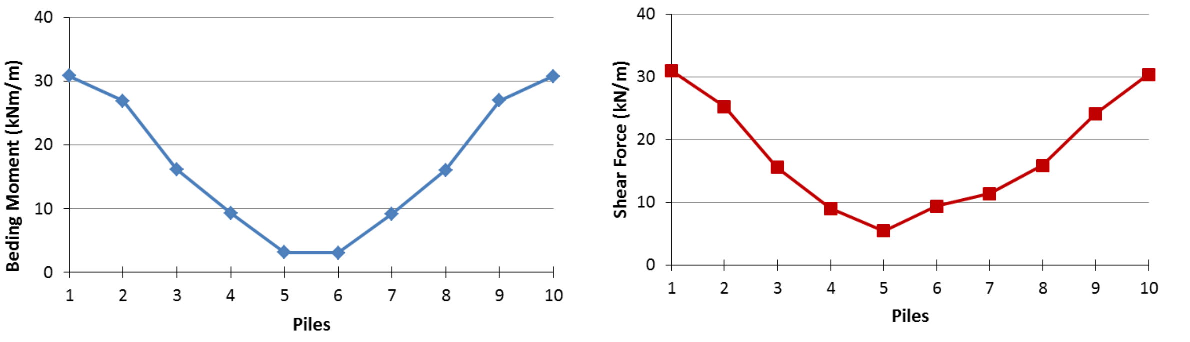
*Analysis/ variable	Embankment Height (m)	Asymmetric load (kPa)	Geosynthetic stiffness modulus (kN/m)
L0	2.50	0	2000
L25	2.50	25	2000
L50	2.50	50	2000
L70	2.50	70	2000
H1.5	1.50	0	2000
H2.5	2.50	0	2000
J1000	2.50	50	1000
J2000	2.50	50	2000
J3000	2.50	50	3000

Note: Each analysis provide results of bending moments and shear forces for all 10 piles of the model. The analysis L0 is the same of H2.5 and the L50 is the same as J2000.

5 RESULTS AND DISCUSSION

The results of the analysis presented in Table 3 are presented in the Figures 5 to 10 ahead. The variation of absolute value of bending moments and absolute value of shear forces on piles from the embankment's toe and middle of embankment is contemplated in graphs.

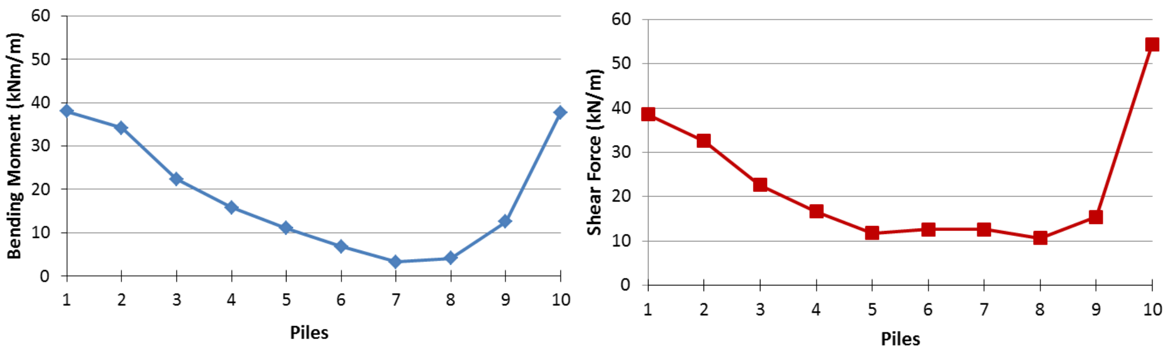
The results of analysis L0 are shown in Figures 5a and 5b. From Figures 5a and 5b is possible to observe that the bending moments and shear forces are higher on the piles that are in the extreme points of the embankment, with approximately symmetric behavior when the asymmetric loading value is equal to zero.



(a) **(b)**

Figure 5 – Analysis L0 accounting: a) Bending moments; b) Shear forces.

The results related to the analysis L50 (load = 50 kPa, embankment height = 2.5m and J=2000 kN/m) are illustrated in Figures 6a and 6b.



(a) **(b)**

Figure 6 – Analysis L50 accounting: a) Bending moments; b) Shear Forces.

It is possible to observe that the increase of the load from 0 to 50 kPa causes an alteration in the symmetry of the stress curves. From the pile 5, the values of bending moment and shear stress do not rise again, as with null loading. The stress values at piles 6, 7, 8, and 9 are less than the values at the piles that are further from the load. This is primarily due to the fact that the load (50 kPa) counterbalances the load imposed by the embankment with a height of 2.50 m. The embankment produces a vertical (and therefore horizontal) stress on the left side of the pile, while the asymmetric load produces a vertical (and therefore horizontal) stress on the right side. Figure 7a and 7b compares the analysis L0 (asymmetric load zero) and L50 (asymmetric load equal to 50 kPa) in terms of horizontal displacements vectors, respectively. Figure 7a indicates that the displacement is mainly directed from left to right and in Figure 7b the displacement present vectors directed from right to left, in a condition closer to the equilibrium.

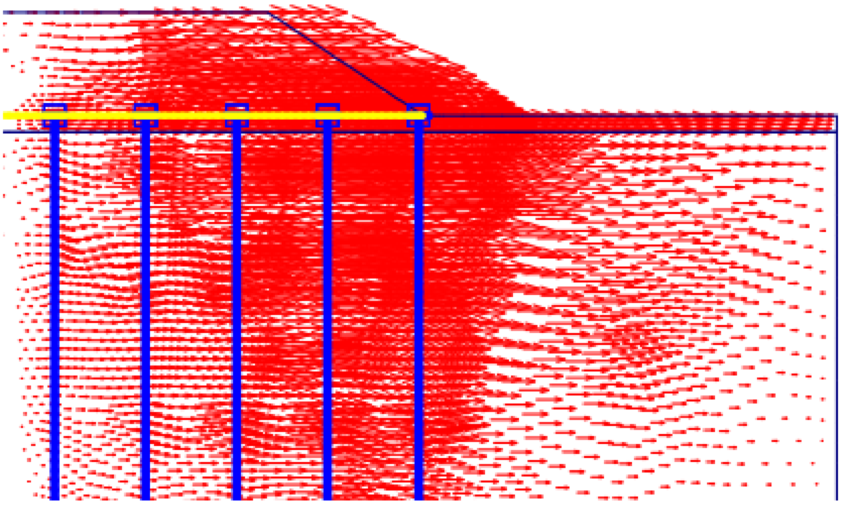


Figure 7a – Horizontal displacement vectors of analysis L0 (asymmetric load 0, embankment height 2.50 and J = 2000 kN/m).

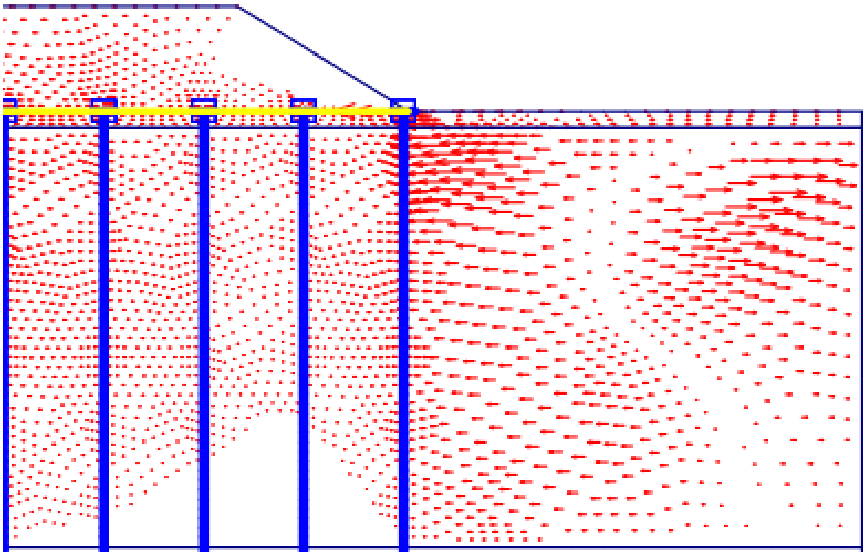
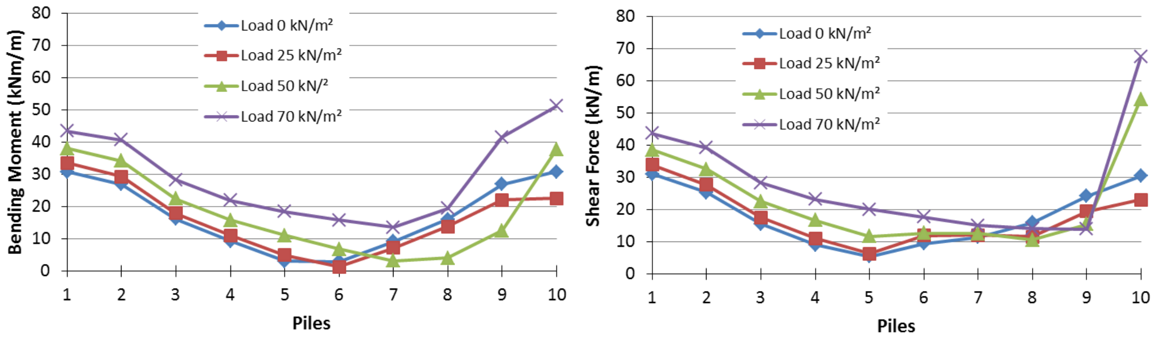


Figure 7b – Horizontal displacement vectors of analysis L0 (asymmetric load 50 kPa, embankment height 2.50 and J = 2000 kN/m).

Figures 8a and 8b shows the variation of bending moments and shear forces on piles, respectively, as a function of asymmetric load imposed.

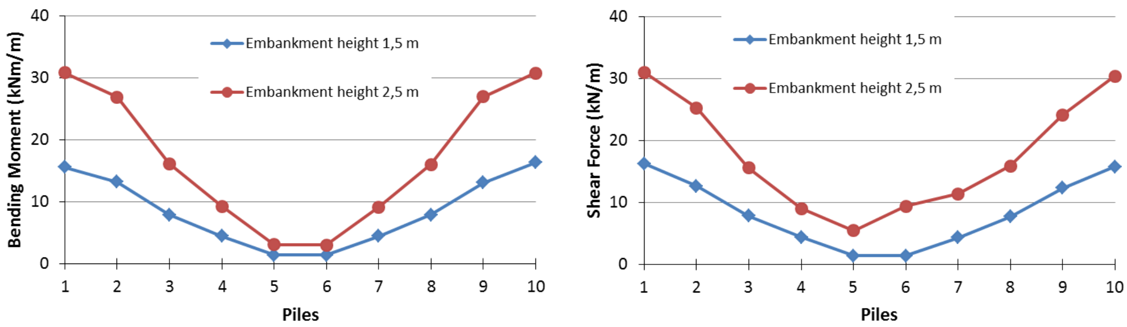


(a) **(b)**

Figure 8 – Analysis L0, L25, L50 and L70 (variation of load and fixed embankment height 2.50 and J = 2000kN/m): a) Bending moments; b) Shear forces.

According Figure 8a and 8b, generally the absolute values of the bending moments and the shear forces increase as the loading value increases. This increase occurs similarly in piles arranged in the farthest half of the asymmetric loading. From the pile 6, for the loading values of 50 kPa and 70 kPa, the absolute values of bending moment and shear force are reduced and reach their maximum values at pile 10.

Figure 9a and 9b show the variation of absolute value of bending moments and absolute values of shear forces, respectively, with relation the height of embankment. For these analyses the asymmetric load was considered zero.

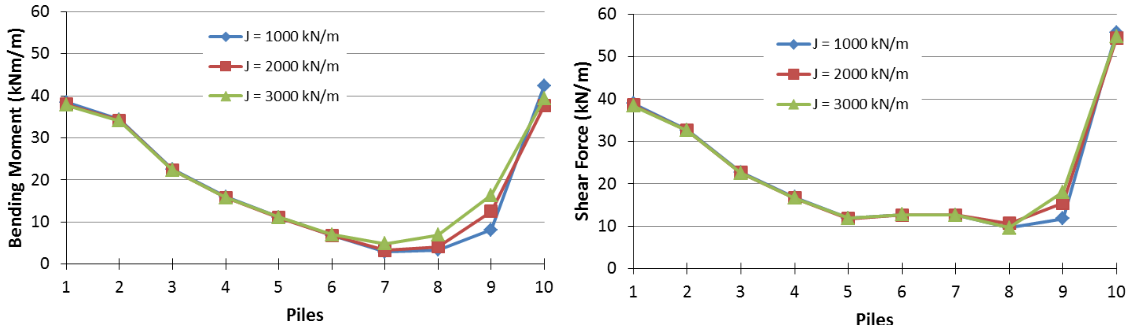


(a) **(b)**

Figure 9 - Analysis H1.5 and H2.5 (variation of height of embankment, load zero and J = 2000kN/m): a) Bending moments; b) Shear forces.

According to the Figure 9a and 9b and for condition when the asymmetric load is equal to zero, both absolute values of bending moments and shear forces decrease when the height of embankment also decrease. The differences between efforts related to the embankment height are higher to the piles which are at the extreme points of the embankment, nest to the embankment's toe.

Figure 10a and 10b show results of absolute bending moments and shear forces considering different values of stiffness modulus of geosynthetics (J). Piles efforts increase only at piles 7, 8, 9 and 10, and their variations are small, indicating little or null interference of the geosynthetic stiffness for the analysis of lateral efforts in piles caused by asymmetric loading in the studied case.



(a) (b)

Figure 10 - Analysis J1000, J2000 and J3000 (variation of stiffness modulus of geosynthetic, height of embankment 2.50m and load 50 kPa): a) Bending moments, b) Shear forces.

6 CONCLUSIONS

It was presented a numerical modeling of piled embankment and bending moments and shear forces in the piles were determined. Some asymmetrical loads were imposed on the right side of piled embankment and the stiffness modulus of geosynthetic was also varied. The main conclusions are:

The efforts (bending moments and shear forces) acting on piles are not equal in all piles. In general, the higher efforts in piles occur next to the embankment's toe and decrease toward to the center of embankment.

The numerical modeling was capable to determine these efforts in all piles and not only in the outer pile as provided by analytical solutions.

For the condition in which the asymmetric load is zero, the higher is the embankment height the higher is the efforts in the piles.

With relation to the asymmetric load, the observed general behavior is the decay of the value of the efforts in the piles closest to the side where the asymmetric load acts. It occurs because the asymmetric load promotes a kind of equilibrium of horizontal displacement near the outer pile.

The value of the modulus of stiffness of the geosynthetics has negligible interference in the efforts acting on piles.

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