Numerical Modeling of Test Embankment Supported by Stone Column Foundation

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Abstract. Embankments on soft soils are a sensitive engineering issue. Over the time, many alternatives have been developed to mitigate the problematic behavior of those structures and the use of stone columns is a frequent strategy that increases its performance. This present study has as aim, analyze numerically, using the finite element software PLAXIS 2D, v.8.2, the performance of an embankment. This structure is hypothetical and has some similarity with a real test embankment. In this way, it is possible to observe the development of settlements and general stability to prove whether the proposed parameters are valid. Using a simplified plane-strain model based on a geometrical transformation that reduces the original column width (Tan et al. 2008), the model was developed considering the proposed geometry and typical soil parameters from shore of Rio de Janeiro city. In summary, this study provides the final safety factor, behavior of the soil-column-embankment in terms of settlement and excess pore-pressures.

Keywords: *Stone columns foundation, 3D for 2D transformation, safety factor against failure*

1 INTRODUCTION

The use of granular columns is a well disseminated practice when dealing with embankments over soft soils. According to Almeida and Marques(2013), granular columns can provide less displacements, both horizontal and vertical, allowing also the dissipation of pore pressure due to its high permeability. Since the columns are composed by a stiffer material, if compared to the surrounding soil, a great part of the embankment load is transferred to underlying layer, reducing the generation of pore pressure. Consequently, as demonstrated by Han and Ye (2002), column stiffness and permeability are accountable for the dissipation of pore water pressure. This characteristic leads to an increase in clay strength and settlement velocity. Thus, this method increases the embankment's general stability and reduces expected settlements.

The technique used to drive the columns is referred as vibro replacement. It consists of a substitution of cohesive soil for granular material, commonly gravel, combined with a constant upward-downward movement compacting the coarse material. Once the determined depth is attained, a jetting is performed to get a clean granular material. The usual method for design incorporates the concept of a unit cell, as highlighted by Almeida and Marques (2013). For a square mesh, which is the arrange chosen in the case study, the equivalent diameter is represent as de=1,13·l. Figure 1, illustrates the unit cell concept in a schematic view.

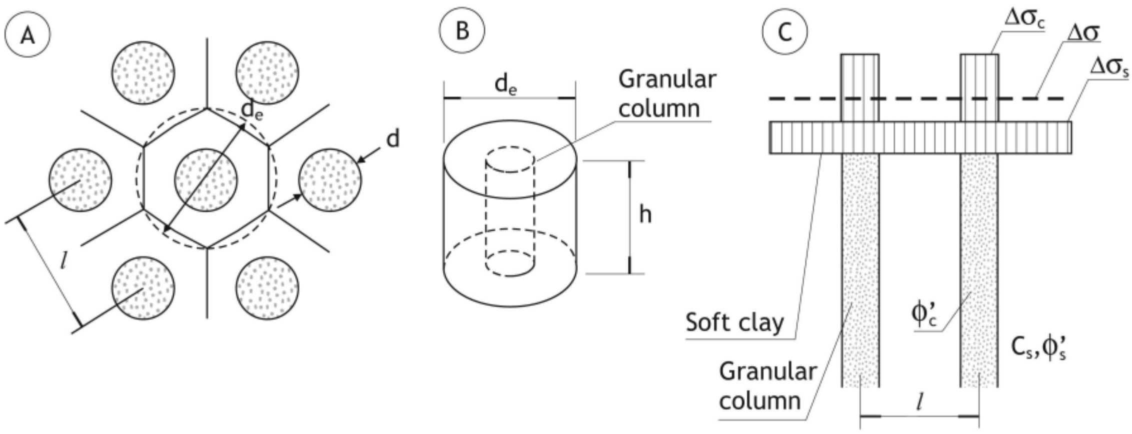

Figure 1 - Schematic view of an unit cell (Almeida and Marques, 2013)

Equation (1) shows the area replacement ratio parameter, where A is the area of soft soil around de column, A_c is the area of the column, and c is equal to π/4 for a rectangular mesh pattern.

$$a_c = \frac{A_c}{A} = c.(\frac{d_e}{l})^2 \tag{1}$$

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In addition, there are two factors important to measure the effectiveness of granular columns. The first one, stress concentration factor,Eq. (2), is defined as the ratio between the increments of vertical stress acting in the column $\Delta\sigma_c$ and in the surrounding soft soil $\Delta\sigma_s$ (as shown in Figure 1). This ratio indicates that when the system comprising soil and column is loaded the columns concentrate the increments of vertical loading due to its higher stiffness.

$$n = \frac{\Delta\sigma_{vc}}{\Delta\sigma_{vs}} \qquad (2)$$

For purposes of design the settlement reduction factor is generally used as method for settlement analysis. It can be defined as the ration between the settlement of the soft soil Δh and the settlement of the treated soil Δh_s, Eq. (3). The value of β is referred as Improvement Factor.

$$\beta = \frac{\Delta h}{\Delta h_s} \qquad (3)$$

Naturally, the analysis should follow a settlement evaluation in order to determine its magnitude and velocity. Many methodologies have been proposed throughout the years with different considerations and assumptions. Najjar (2013) gives a general overview of those studies over the past 40 years. Examples of those formulation are presented by Priebe (1995), Han and Ye (2002), Balam and Booker (1981) among others. Recently, however, the use of computational tools such as finite element packages have been used to get more realistic factors together with new possibilities not available in the previous formulations. It is worth noting that the use numerical methods must consider effects of column installation, 3D-2D transformation, soil model and smear effect.

In this present study, a numerical analysis of an embankment on soft soil was conducted to evaluate its performance concerning settlement, pore pressure dissipation and safety factor. The geometrical transformation proposed by Tan et al. (2008) was used to convert 3D dimension into an equivalent 2D Plane Strain model. The calculations were conducted using PLAXIS 2D v.8.6 package.

2 MODEL GEOMETRY

2.1 Embankment dimensions and soil profile

The geometry of the case study is based on a project of a test embankment projected as a minor part of a bigger embankment and created to validate an unusual column spacing. Figure 2 illustrates a cross sectional profile of the soil-embankment system. The layer of soft soil is 8.0 meters thick and divided in 5 distinct profiles and sits above a layer of sand. Upon the clay layer a work platform was constructed to give proper operational conditions to the vibro substitution machinery. In order to provide better pore pressure dissipation, it is usual to construct a layer of sand between the soil profile and the embankment. Additionally, two mono-axial geogrid was placed (orthogonally to each other) in between the sand blanket and the work platform to increase stability.

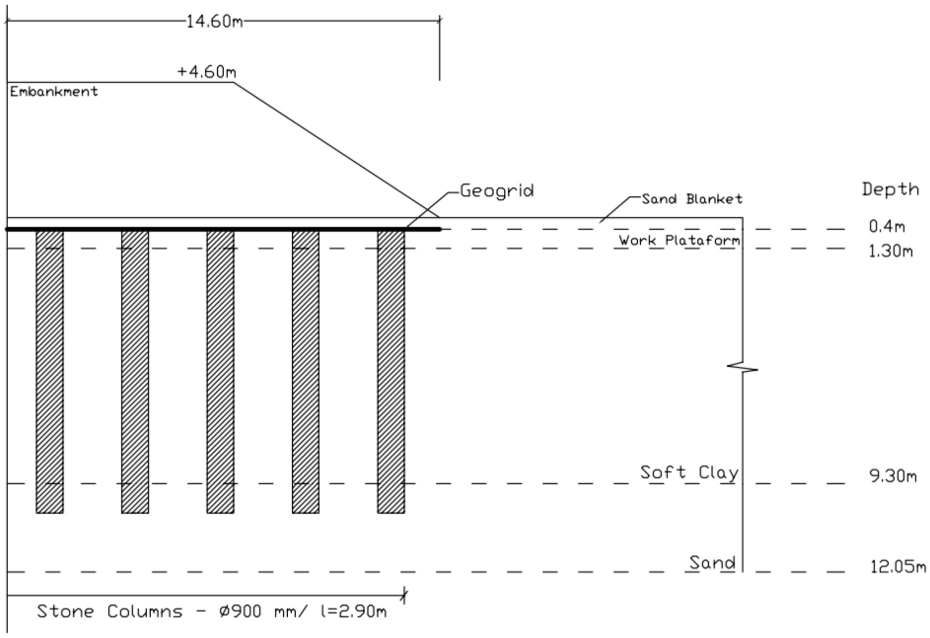

Figure 2 – Cross sectional profile of the soil-embankment system

The test embankment is 4.6m high and sustained by 100 stone columns with a 900-mm diameter, 9.90 m high and spaced by 2.90 m center-to-center. However, as mentioned before, the subsequent numerical analysis is a plane-strain modelling; hence a transformation is needed to convert the unit-cell dimensions to the proposed numerical approach.

2.2 Geometrical Transformation

Tan et al. (2008) proposed two methods of transforming the axisymmetric unit-cell into a plane-strain model. The first method considers the plane-strain wall width equals to the axisymmetric column diameter, and the soil permeability is correlated following proper analytical equation; while in the second method, the plane-strain wall width is matched per an analytical equation, and the soil permeability remains the same as the axisymmetric case. Both methods were tested following validations with unit-cell simulations, classic analytical solutions and field data. Method 2 showed a satisfactory validation results, whereas Method 1 failed to reproduce soil plastic yielding giving thus inaccurate settlement values. In this way, Method 2 is the indicated conversion method.

Figure 3 shows the geometry conversion proposed for Method 2. This approach converts vertical drains in plane-strain walls considering the compatibility between drainage capacity in axisymmetric and plane-strain models. Equation 4 gives the value for plane-strain column width.

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$$b_c = B\frac{r_c^2}{R^2} \tag{4}$$

The parameter R indicates the radius of influence of a single stone column in axisymmetric conditions, while B is its equivalent in plain-strain conditions. Their relationship for a square pattern of columns is expressed by R=1.13·B. Following the previous equations, the plain-strain column analyzed has b_c=0.11 m and B=1.45 m.

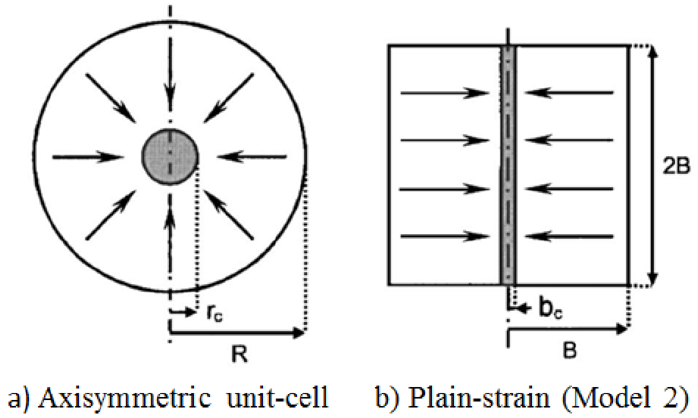

Figure 3 – Cross-section of conversion parameters (Adapted from Tan et al. (2008))

3 MATERIAL PROPERTIES

Table 1 and Table 2 present soil properties and respective constitutive models used in this study. For granular materials, the constitutive model adopted was Mohr-Coulomb and for soft soils, the Soft Soil Model (SSM) was used. In this way, soil consolidation can be properly modeled as a time dependent variable. Additionally, in this model stiffness is a stress dependent variable and the failure criteria follows Mohr-Coulomb formulation.

Table 1 – Soil Properties (Granular Materials)

Material /Property	Embankment	Sand Blanket	Work Platform	Sand	Stone Column
γ_n (kN/m³)	20	17	20	20	20
γ_{sat} (kN/m³)	22	19	22	22	22
E (kPa)	25.000	45.000	30.000	45.000	80.000
ν [-]	0.33	0.28	0.30	0.25	0.30
ϕ' (°)	30	35	30	35	42

Model	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb	Mohr-Coulomb
Condition	Drained	Drained	Drained	Drained	Drained
k_v (m/day)	1	32.4	0.864	1.728	86.4
k_h (m/day)	1	32.4	0.864	1.728	86.4

Table 2 – Soil Properties (Soft Soil)

Material /Property	1st Layer	2nd Layer	3rd Layer	4th Layer	5th Layer
γ_n (kN/m³)	15	13	14	13	13
γ_{sat} (kN/m³)	15.48	13.90	14.41	13.6	13.54
c_c [-]	0.42	1.36	0.8	1.79	1.91
c_s [-]	0.14	0.22	0.17	0.22	0.28
e_0	1.6	3.1	2.2	3.2	3.2
c' (kPa)	5	5	5	5	5
ϕ' (°)	32	32	32	32	32
Model	SSM	SSM	SSM	SSM	SSM
Condition	Undrained	Undrained	Undrained	Undrained	Undrained
k_v (m/day)	1.695E-04	1.119E-04	1.108E-04	1.78E-04	2.967E-05
k_h (m/day)	4.24E-04	2.797E-04	2.769E-04	4.45E-04	7.416E-05

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.

4 NUMERICAL MODEL AND ANALYSIS

The numerical model is shown on Figure 4 together with its finite element mesh. The analysis was performed in construction stages, following the typical field operation including the time needed for each phase. A consolidation analysis was run in order to account for soft soil undrained behavior in both staged construction and consolidation phase.

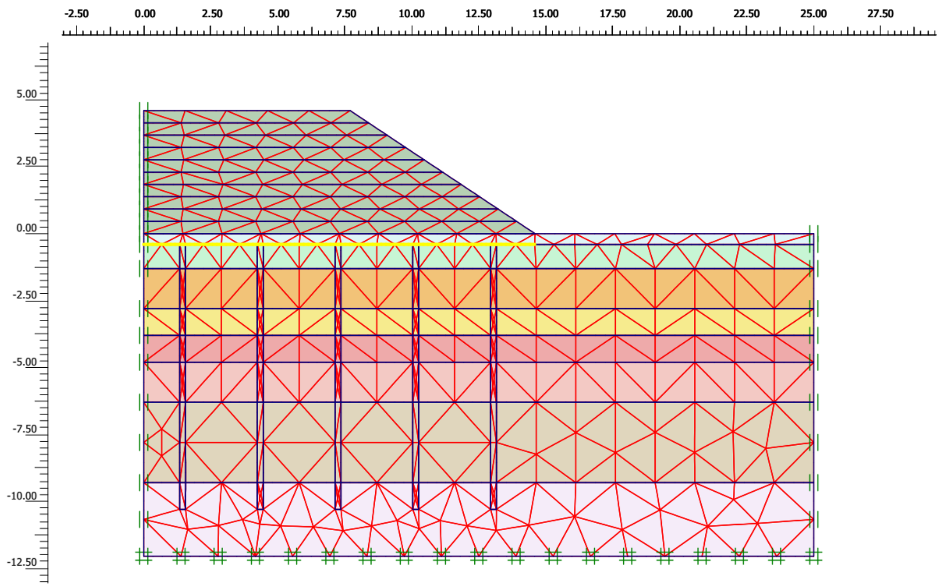

Figure 4 – Plane Strain Representation of Numerical Model and Finite Element Mesh.

The finite element mesh has a total number of 5917 nodes, with an average element size of 0.759 meters.

5 RESULTS

As a first result, Figure 5 illustrates the settlement behavior at the middle of the embankment. It was verified a total vertical displacement at the center of the embankment of 1.71 meters after approximately 1600 days of stabilization.

The results for dissipation of pore water pressure are presented in Figures 6 and 7. In both it is possible to observe that the accumulation of excess pore water pressure happened during the embankment construction period (30 days) and its dissipation listed until stabilization. It was considered a minimum value for excess pore pressure of 5kN/m^2 for complete equilibrium of the embankment. Hence, total stabilization corresponds to the time required to reach the minimum value of pore water pressure.

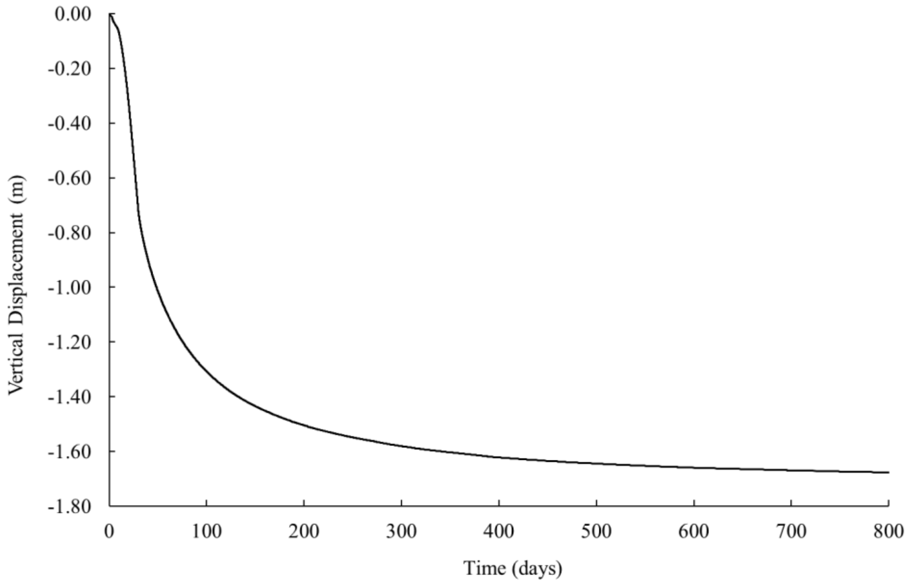

Figure 5 – Settlement Curve for Embankment Construction.

Another important feature to note is the expected drainage capability of the stone column. In figure 6 the curve for dissipation of pore water pressure inside the column shows that there was no considerable excess of pore pressure. From Figure 6 it is observed that the closer of column the lower is the excess of pore pressure.

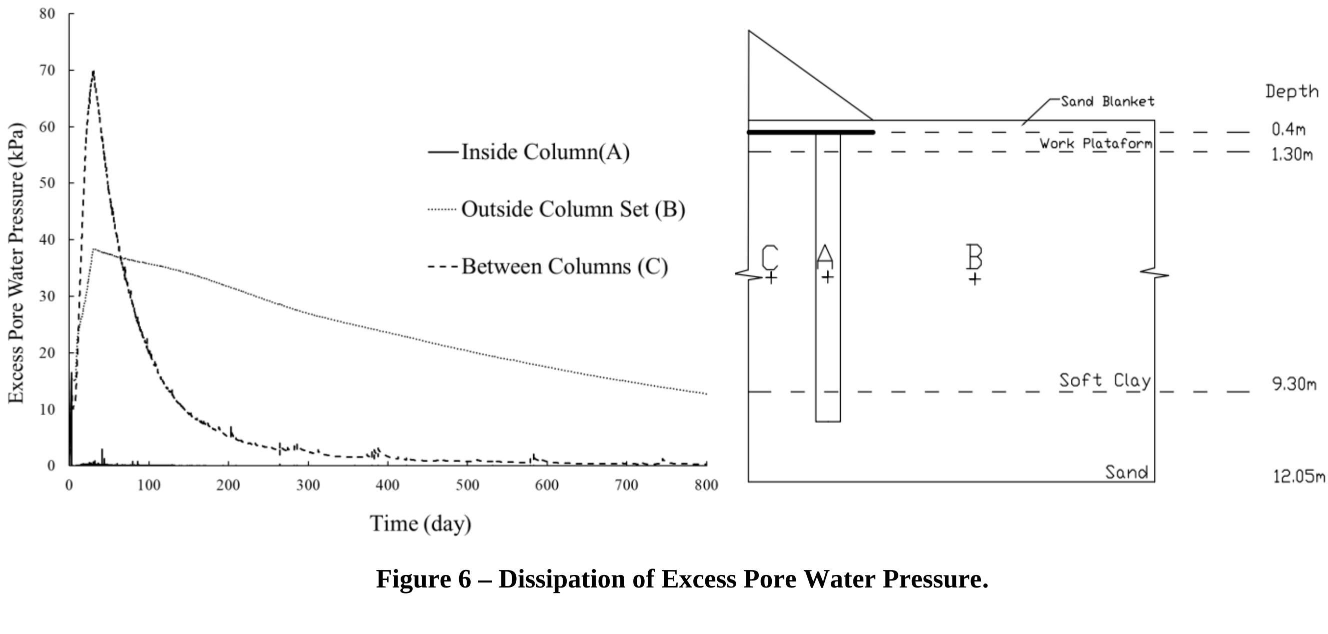

Figure 6 – Dissipation of Excess Pore Water Pressure.

In the same way, Figure 7 shows that no pore pressure was generated inside columns in both stages – after construction and after stabilization. Figure and 7(b) also indicates that the column installation accelerates the drainage and reduce the excess of pore pressure in the soft soil. After construction the excess of pore pressure is high inside soft soil (Figure 7a) and at the end of construction the pore pressure is dissipated. Comparing the area outside the column setting (unimproved area) and the area between the columns, it is possible to observe a faster drainage, dissipation of pore pressure excess and the improvement of soft soil.

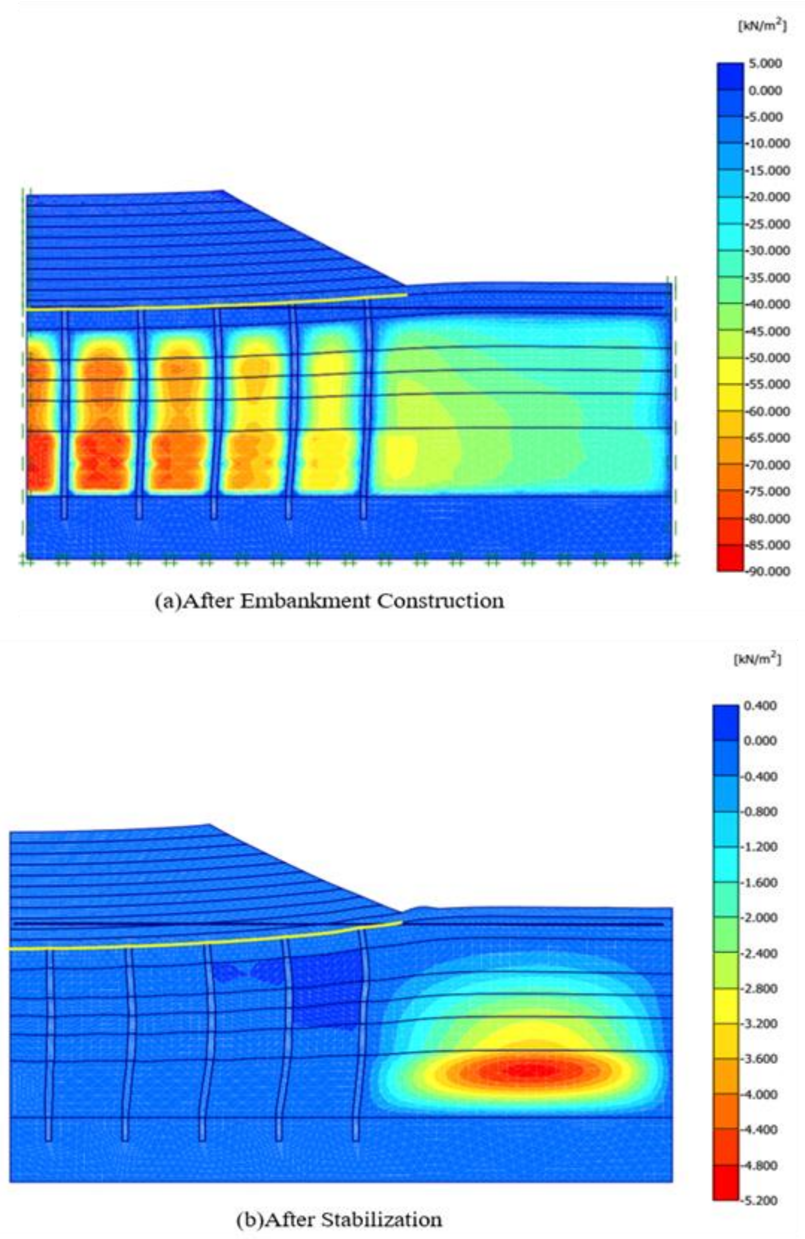

Figure 7 – Excess Pore Water Pressure Shadings.

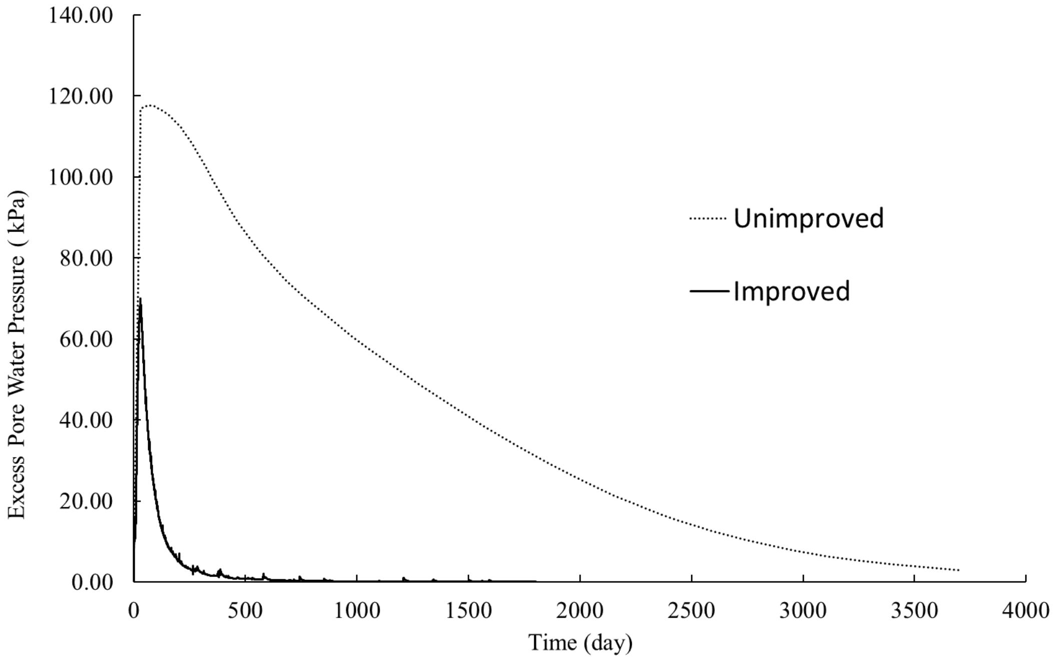

Figure 8 – Dissipation of Excess Pore Pressure in Unimproved and Improved models.

Figure 8 illustrates how column installation can reduce substantially the time needed for dissipation of pore pressure. The graph shows the curve for points of same coordinates in both cases. Comparing them, it is possible to observe that around 200 days the minimum value of pore pressure is reached for the improved system. For the unimproved system, the time needed is around 3700 days. Hence, the reduction factor is 18.5.

The Figure 8 shows the stress concentration factor n ($=\Delta\sigma_{v,c}/\Delta\sigma_{v,s}$) for a stone column located at the middle point of embankment. According to Figure 8, the stress concentration factor is different for different stages of the analysis. Initially, after the embankment construction n=2.6, indicating an expected concentration of stress at the stone column. In the same way, after stabilization n increases to 3.6. This behavior is related to the settlement of soft soil due the consolidation process, so the value of n is time-dependent.

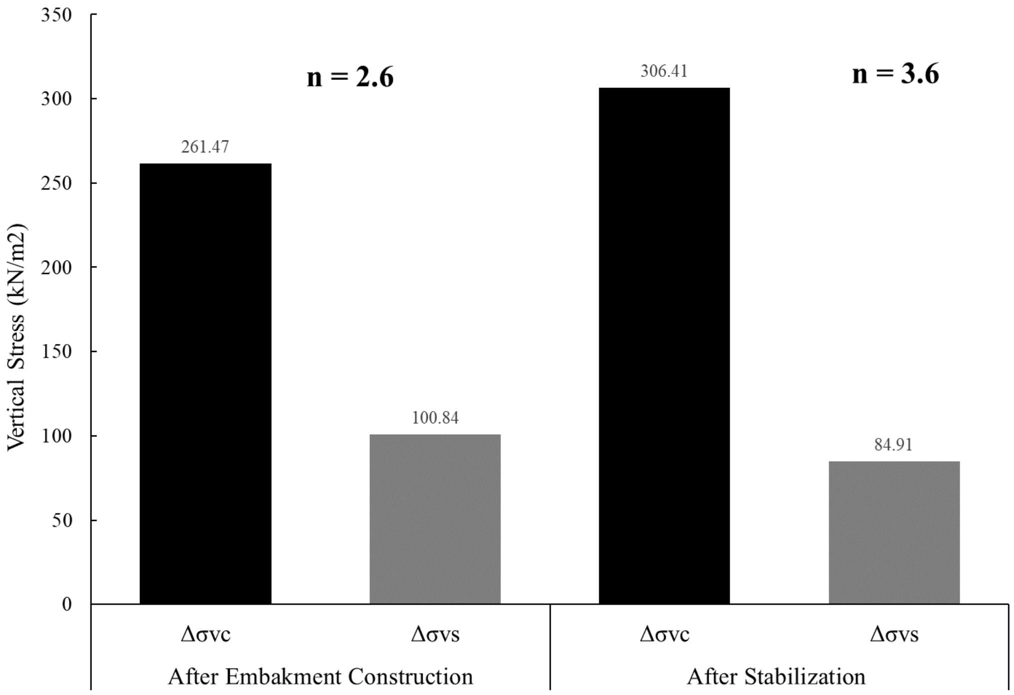

Figure 8 - Stress Concentration Factor.

The last analysis is referred to the final safety factor after the embankment construction. Plaxis generates additional displacement during the phi-c calculation. While these displacements do not have a physical interpretation, and are not relevant, it gives a sense of the failure mechanism present in the studied case.

The factor of safety is estimated in two moments: just after the end of the embankment construction and after the consolidation period needed for stabilization. In the first moment, the factor of safety is 1.79 and in the second moment is 2.4.

In this way, is possible to observe that the safety factor increase as the excess of pore pressure decrease. So, the drainage provided by the stone columns leads to an increase in the overall safety factor against foundation failure.

6 CONCLUSIONS

This paper presented a numerical simulation of a test embankment construction over a soft soil profile. Its major aim as mentioned before was to give insights about the behavior of this structure and evaluate its feasibility.

A factor of safety of 1.79 was found just after the final construction stage – its most critical phase- indicating that there is a guaranty of its stability. It is worth noting also that this safety factor is greater than the usual adopted for engineering project according to Brazilian regulations.

The stone columns served its purpose to reduce settlements and reduce the time required for the excess pore pressure dissipation.

The transformation of the condition 3D to 2D can be used to analyze a tridimensional problem using a 2D tool.

The stress concentration factor ($n=\Delta\sigma_{v,c}/\Delta\sigma_{v,s}$) is not constant along construction period and post construction period, contrary the value of n is time-dependent. The value of n increase with the time, i.e., with the pore-pressure dissipation.

Dissipation of excess of pore-pressure is immediately inside the column and between columns the time required for dissipation is smaller than in points far away for columns.

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