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Verification of a plane strain and three-dimensional model for the analysis of stone columns group

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Abstract

The effect of using stone columns on settlement rate of soft clay soils is considered a subject with little studies. This paper presents implementation of 2D and 3D numerical model to predict settlement rate of soft clay soils. The numerical models were compared with observations of large-scale field project. The project consisted of implementation of 0.90m end-bearing stone columns with spacing of 2.90m and average the field behavior especially for settlement values, settlement rates, and pore water pressure. The 2D model is considered conservative at the early stage of loading and accurately predict the field observations for long-term behavior. The final results show that 3D model is more realistic than the 2D model, but it requires more time and computational efforts.

Keywords: Stone columns; Settlement; Excess pore water pressure; soft clay

1. Introduction

Soft soil construction has many problems in the geotechnical engineering field due high sensitivity, low shear strength, settlement, and compressibility. An excessive settlement occurs during and after construction due to soil's high compressibility. Several techniques are used in the past decades to control and minimize the amount of settlement. Use of stone columns to decrease the settlement, increase settlement rate, and improve the bearing capacity of soft soils was first used in the 1830s and has been widely used since the 1950s [1]. Popular installation methods for granular columns are known as replacement and displacement methods.

Priebe [2] analyzed a group of stone columns considering the compressibility effect of granular columns materials, and developed design charts to estimate the settlement for both single and strip footing. Han and Ye [3] estimate the rate of consolidation for soil supported on stone columns. It has been noticed that during the consolidation phase the stress on stone columns increases with increasing time, but the stress on soil decreases. With the end of the consolidation period, the stress concentration ratio was steady. Han and Ye [4] used experimental and theoretical approach to prove that the use of stone columns can accelerate the rate of consolidation of soft clay by providing drainage path and reducing the stresses on the soil. The researchers proposed a modified coefficient of consolidation that account for the effect of the stone column-soil modular ratio or stress concentration ratio.

Deb et al. [5] focused on the effect of using encased stone columns with geosynthetic. They presented a theoretical solution to estimate the settlement of soil due to encasing stone columns with geosynthetic materials. Venda et al. [6] presented a numerical model to estimate the settlement and pore water pressure when using Deep Mixing Columns (DMC). The deep mixing columns acting like drains and suction water when columns permeability has greater value than surrounding soil and controlling the flow. The stiffness and permeability of DMC considered important parameters to control the consolidation time.

In contrast to most of the previous researchers, the current study focuses on addressing the effect of using stone columns on consolidation rate of treated soft clay soils by verifying 3D and 2D numerical models against field measurements. The verified models can be used to address other factors that are hard to be considered in field-scale tests on consolidation rate.

2. Test Embankment

2.1. Model Geometry

The current study of the embankment supported on soft clay soil improved with stone columns located in Brazil. The embankment constructed in a large area with 10m soft soil thickness. The embankment high (compacted drainage layer and fill) was 5.35 m. Grid of vibro-stone columns (VSCs) executed in (10 × 10) columns in a square shape below the embankment, as presented in Figure 1. Stone columns average diameter (D) was 0.9m. the installation method was the dry bottom-feed technique. The stone columns are compacted, the quality of compaction depended on the hole volume which filled with gravel material. The spacing of stone columns (S) was 2.9m from center to center, and the area replacement ratio (ARR) was about 7.56%. The geometry of this case study based on a project of a test embankment area work as a minor part of a bigger embankment project and created for validation of column spacing [7][8][9].

Figure 1 presented a cross-sectional of soil-embankment area. The soft clay layers are about 10 meters in thickness divided to 5 distinct profiles and lays above a sand layer. Above the clay layer, a working platform was constructed to give identical proper operational conditions for vibrating substitution machinery. To obtain a better pore pressure dissipation result, piezometers (PZ) were used. In order to estimate an accurate settlement result, settlement plates (SP) were used.

In addition, two mono-axial geogrids with stiffness (J) 300KN/m added (orthogonally to each other) between the blanket of sand and the working platform for increasing the stability and ensure uniform distribution of stress above the stone column-soil interface.

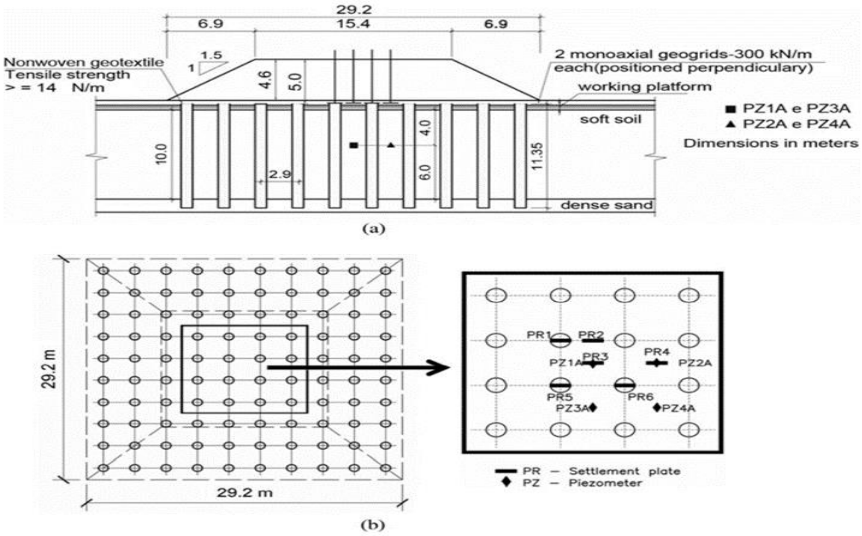

Fig 1. Cross-sectional profile and plan of the soil-embankment

2.2. Soil Parameters

The properties of the embankment, stone columns, and sand materials are presented in Table 1 that is determined from laboratory soil tests, and experience [10], [11]. Table 2 shows the soft clay soil geotechnical properties, five layers of soft clay extent below the embankment between the working platform and sand layer as shown in Figure 1. Previous related studies investigated in the stockyard (Lima,[12]; Almeida et al.,[11]; Lima et al.,[13]) and a similar comparison done between experimental measurements and the Finite element analysis (FEA). The comparison verified that the parameters of stone columns presented in Table 1 are consistent. The changing of the stone columns friction angle accounts for the strength curvature envelope obtained by the triaxial test (large diameter specimens) data presented in Gebrenegus et al.,[14].

The properties of soil layers estimated from the site are shown in Table 1. Soft soil geotechnical properties were estimated by laboratory tests on five undisturbed samples within various depths along. By the recommendations of Ladd and DeGroot [15] The soil samples estimated using the Shelby tubes and the Brazilian standard NBR 9820/ 1997. The odometer tests executed on these samples by stages of loads applied until it reaches 400 kPa and later unloaded until reached 25 kPa in order to estimate the controlling parameters γ', γ, C_s, C_c, C_v, e_o, and OCR. The internal friction angle (φ') and effective cohesion (c') of the soft soil have been applied according to previous studies that have been investigated by Almeida et al. [16] and Futai et al.[16][17]. The major value of at rest earth pressure coefficient (Ko) was estimated by using φ' and OCR.

TABLE 1. Granular Materials Properties. (Pires,2021)

Material /Property	Embankment	Sand Blanket	Work Platform	Sand	Stone Column
γ (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
kv (m/day)	1	32.4	0.864	1.728	86.4
kh (m/day)	1	32.4	0.864	1.728	86.4

TABLE 2. Soft Soil Properties (pires,2021)

Material /Property	1 st layer	2 st layer	3 st layer	4 st layer	5 st layer
γ (kN/m³)	15	13	14	13	13
γsat (kN/m³)	15.48	13.90	14.41	13.6	13.54
Cc [-]	0.42	1.36	0.8	1.79	1.91
Cs [-]	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
kv (m/day)	1.695E-04	1.119E-04	1.108E-04	1.78E-04	2.967E-05
kh (m/day)	4.24E-04	2.797E-04	2.769E-04	4.45E-04	7.416E-05

Note: γ = the density of soil; γ_{sat} = saturated density of soil; E = Young´s modulus; ν = Poisson coefficient; Cc = compressibility coefficient; Cs swelling coefficient; e_o = initial void ratio; c' = effective cohesion; ϕ' = effective friction angle; k_v = vertical permeability; k_h = horizontal permeability.

3. Numerical Analysis

Two- and three-dimensional analyses (2D, 3D) were performed to simulate the field conditions using Plaxis finite element analysis software. The model geometry consists of an embankment with 4.95m high, 0.4m compacted sand thickness working as a drain layer, and five layers of soft clay soil with 10 m thickness laying under the platform layer as shown in Figure 1.

For granular materials, the adopted constitutive model was Mohr-Coulomb model and for soft clay soils, the Soft Soil Model (SSM) was applied. In this way, soil consolidation can be perfectly simulated and modeled as a time-dependent variable. Additionally, in this case study model, stiffness is a stress-dependent variable and the failure criteria followed by Mohr-Coulomb formulation.

Figure 2-a shows the 2D finite element model performed in this study. The stone columns at Plane strain 2D analysis was transformed and modeled as trenches of equivalent stiffness as recommended by Han et al. [18]. Equivalent stone columns diameter is estimated to be 0.22m and used in the 2D plane strain analysis. The total elements of the finite element mesh were 5,213 triangular elements.

Figures 2.b shows the 3D finite element model with dimension of (29.2 m x 29.2m) and 21.4 m total high. Full geometry was modeled for obtaining accuracy results. The FEA mesh used 328,901 tetrahedral elements for the soil layers. The boundary conditions adopted in 3D were the same used in the 2D analysis. The measured results of settlement and excess pore water pressure of the experimental results compared with the 2D and 3D analysis

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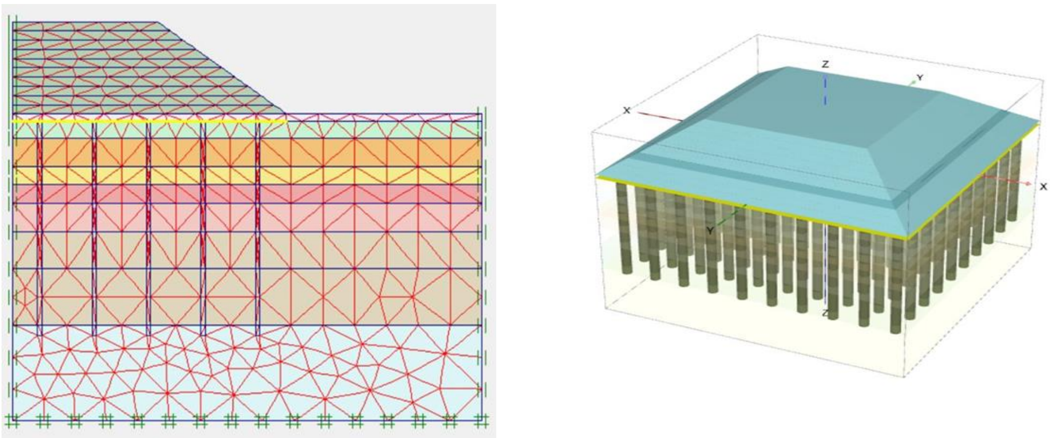

a **b**

Fig 2. Finite element model geometry: **a.** 2D plane strain model and **b.** 3D model

The boundary conditions assumed to be displacements-restrained in vertical and horizontal direction at the bottom of the model, and vertical direction only at the sides. The pore water pressure flow is bounded laterally in the soft clay layers. The loading construction and consolidation phases that adopted into 2D were the same in 3D analysis model. The phases of loading followed by stage of consolidation are conducted.

4. Results

The results of 2D and 3D analysis discussed in this section. The displacement and pore water pressure values predicted by the 2D and 3D model are compared by the observed field values to address the level of accuracy of each model.

4.1. Displacement

The results obtained from both 2D and 3D FE analysis models were compared with the field observations as shown in Figure 3 for soil improved with stone columns. After the end of consolidation, the vertical displacements accelerated with time until the limit state condition was reached. It is apparent that the 3D numerical model well agrees with the measured data during the loading stage up to reach 1400 days. Although the 2D and 3D well agreed in final settlement result, the 2D was 29 cm higher than the original settlement value until consolidation reach 550 days. The agreement of results between the 2D and 3D analysis proves that the geometric transformation equation of Han et al., [18] was satisfactory.

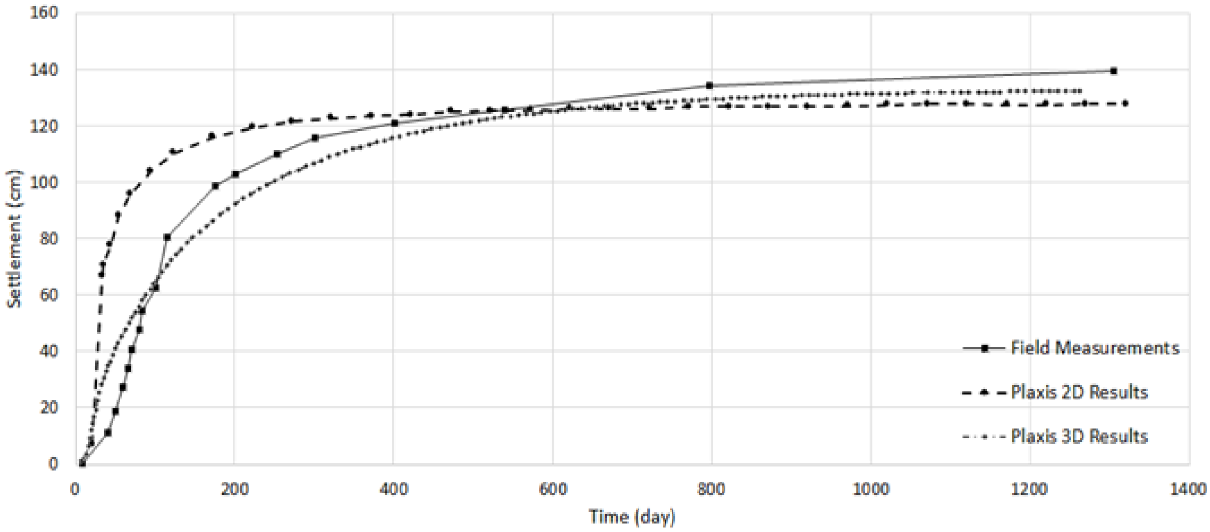

Fig 3. Verification of the vertical displacements for improved soil: Field observations, 2D, and 3D analysis

4.2. Porewater pressure

The measured pore water pressure in unimproved soil was 120 KN/m2 and it is a large value, but the result is reasonable due to soft clay large thickness. Comparing with improved soil pore water pressure, the value was 71KN/m2 due to presence of stone columns. Results of 2D, 3D numerical analysis results for improved soil are compared with the measured values in Figures 4. The behavior of measured and numerical analysis was the same in the improved soil showing that the numerical values are close to measured results.

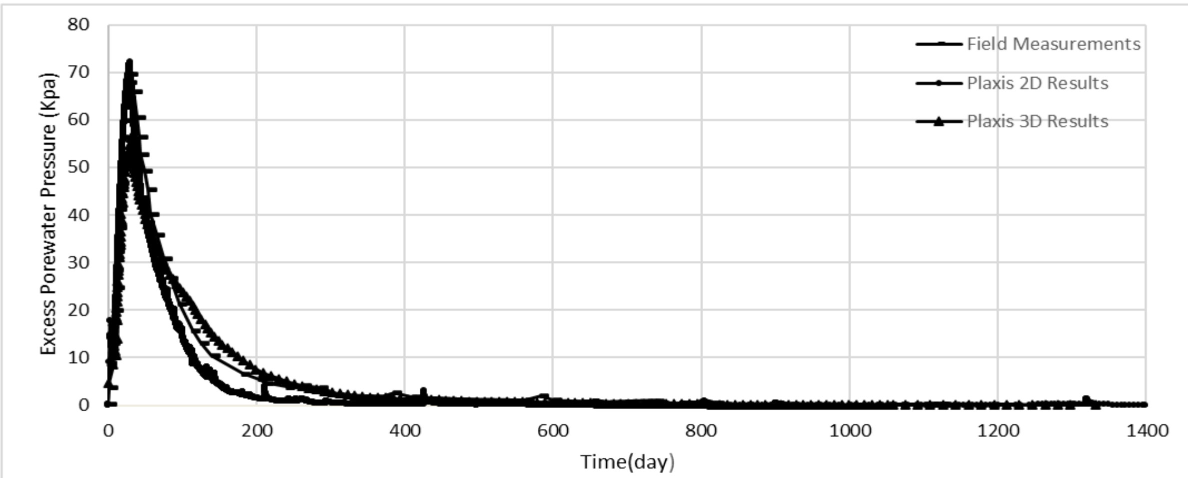

Fig 4. Verification of the excess pore-water pressure for improved soil: Field Measurements, 2D, and 3D analysis

5. CONCLUSIONS

This research paper discussed a case study of embankment supported on soft clay soil improved with stone columns. Verification analysis done with 2D and 3D finite element analyses, and the following was observed:

a- Verification results with 2D are quite like 3D, that is due to application of equivalent drainage trench as recommended by Han et al [18].

b- The settlement value of 2D and 3D analysis was almost the same, although the 2D exceeds in settlement value about 29 cm from the measured values at early stage of loading.

c- The 3D analysis is proved to be more accurate than the 2D analysis model. When considering the required computational effort to carry out the 3D numerical model, the 2D analysis might be considered for preliminary assessment.

d- It was observed that the results of pore water pressure obtained by the finite element numerical models were smaller than the measured values. Possible reason for that is the uncertainty in measuring in-situ accurate values due to the vibration movement during stone columns installation.

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