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Modelling and Sizing of a Floor Reinforced by Ballasted Columns Intended to Support a Tank

Cheikh Diallo Diène, Madièye Fall, Souka Bidzha Harlin Sylvaire

Laboratory of Modelling and Soil Mechanics, Electrical and Building School (ESEBAT), Dakar, Senegal
Email: cheikhddiene@gmail.com

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Abstract

This work aims to study the modeling and sizing of a floor reinforced by ballasted columns. We are studying the system of reinforcement by ballasted columns because this technique is able to replace deep foundations that are technically difficult to realize and their cost is higher. The modelling and dimensioning of foundations on a ballasted column will be an important contribution to the state of the art of this method because it will highlight the mode of transfer of loads, and will expose the induced deformations by also allowing to verification criteria of bearing capacity and allowable settlement according to geometric information of the model. The columns on a substrate located at 9 m have a length of 9 m and a diameter of 40 cm and were obtained by incorporating ballast of granular class 0/31.5 of internal friction angle of 38˚ and a density weight of 21 kN/m^3. The choice of this method is based on the geotechnical characteristics of the initial soil. Thus, identification and characterization tests were carried out to estimate the bearing capacity and the settlement giving respectively 125 kPa and 57 cm. These results show the ground does not have sufficient mechanical properties to withstand the loads transmitted by the tank. By adopting the reinforcement of the soil with ballasted columns, numerical calculations show that after applying a load equal to 265.1 KPa, 20 cm vertical settlement and 17 cm horizontal displacement were obtained. This is in the tolerable deformation range for our tank, namely, less than 20 cm. Analytically, in addition to reducing settlement, ballasted columns, Due to their high stiffness, they have effectively contributed to the increase of the permissible soil stress up to 257 kPa.

Keywords

Reinforcement, Ballasted Columns, Reservoir, Geotechnical Modeling, Plaxis 2D

1. Introduction

The population is increasingly noticing premature degradation of the works, sometimes leading to disorders. These disorders are due to the low resistance of the soil and its high compressibility, which can lead to significant subsidence and deformation [1]. To avoid basing on these compressible soils, the engineer is led to opt for deep foundations consisting of driving down piles to be based on more rigid lithologies located in depth [2].

Soil reinforcement is an alternative to the expensive and technically difficult to build deep foundations [3].

The emphasis on soil improvement needs has been increasing since the 1960s, and research on reinforced soils has been gaining interest since the 1980s. Despite the information gathered from experimental investigations, it turns out that theoretical research has allowed, at different levels, practitioners with additional knowledge to understand, in particular, the functioning mechanisms of reinforced soils and another source of validation of experimental results [4].

The field of soil reinforcement has recent developments and literature. For example, Gens *et al.* proposed an interface element formulation for the analysis of soil-reinforcement interaction in 1989 [5] and on the same wake, Michalowski, Radoslaw L. studied soil reinforcement for seismic design of geotechnical structures in 1998 [6].

There are several reinforcement techniques, each of which requires a prior verification of the feasibility of its execution in the geotechnical conditions of the project. The first is the quantification of predicted performance for improved soil. However, reasonable lead times and the cost of the improvement operation must be considered, avoiding that it is disproportionate to the cost of another solution conceivable foundation.

However, the predominant criterion of choice remains the granulometry of the initial soil. One of the most used techniques is the reinforcement with ballasted columns recommended for works with large support surfaces (works on floors, reservoirs, embankments…) transmitting a relatively moderate vertical stress (less than or equal to 120 kPa). This results in a nearly uniform and allowable settlement [7]. Indeed, Al Saoudi *et al.* studied soft soil improved by stone columns and/or ballast layer in 2015 revealing a bearing improvement ratio of 2 and a settlement reduction ratio of 17% [8]. Further Bouziane, and Abdelkhalek, through their paper published in 2022 show that the material type and spacing between columns in a triangular or square configuration can greatly affect the reinforcement efficiency [9].

The most commonly used calculation method is the homogenization method which is practical but does not consider constraints developed inside the columns thus ignoring the soil-column interaction. What leads to a one-dimensional calculation of the settlement (oedometric method) does not reflect the reality of the observed phenomena.

The objective of this paper is to show the improvement of the lift, and the reduction of the settlement by the incorporation of ballasted columns while considering ground-columns.

To achieve this goal, we will use the geotechnical characteristics of the initial soil to prove the need for improvement and, in turn, determine the new characteristics after the incorporation of the columns. In order to make the interaction between ground and in order to contribute to the understanding of this complex phenomenon, numerical finite element studies will be carried out under the Plaxis software.

2. Material and Methods

2.1. Geotechnical Parameters Description

The projected structure is a circular flour storage tank transmitting uniform surface loads that push us to choose a deck as a foundation system. The 20m diameter flush to 1m, transmits a uniform load of 150 kPa corresponding to a permissible settlement of 20 cm.

The geotechnical survey of the site showed a stratigraphy of the soil made up of:

- From a layer of clay, 9 m thick of density $\gamma = 17$ kN/m^3, an elasticity module Es = 3600 kPa and an undrained cohesion Cu = 58 kPa.
- A calcareous layer, similar to a rigid substrate.

The results of the pressure-measuring test are shown in **Figure 1**.

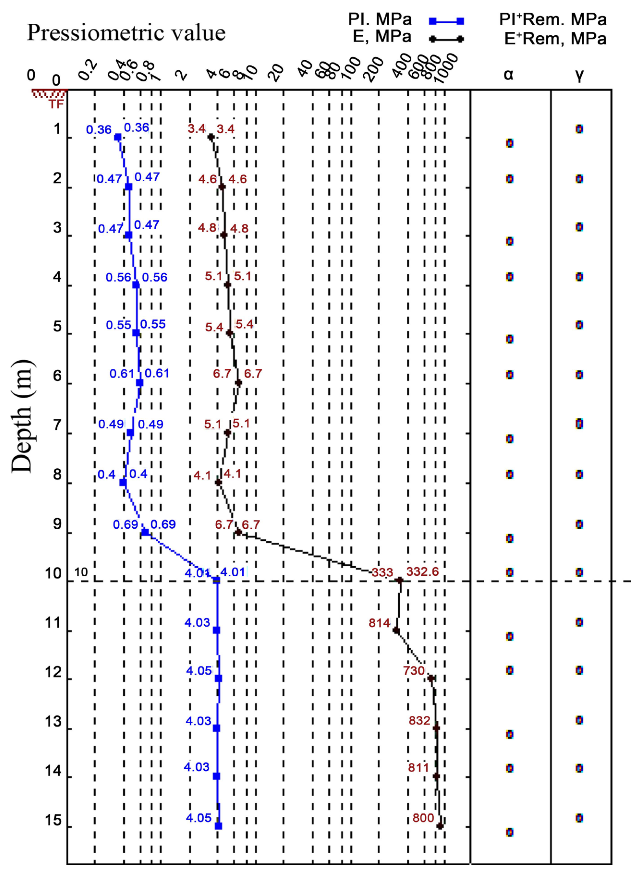

Figure 1. Results of the pressure-measuring test on the base floor.

The load capacity calculation using the Terzaghi [10] approach gives a value of 125 kPa, which is significantly lower than the load transmitted by the container. The estimated settlement corresponds to a value of 57 cm which is much higher than the permissible settlement of 20 cm. In order to increase the load-bearing capacity and reduce the settlement of the tank to a value admissible (which allows to guarantee its stability during service), a reinforcement by ballasted columns was decided.

The technique of improvement by ballasted columns is equivalent to incorporating into an initial soil, whose mechanical characteristics (C, ϕ, P_{LM} or peak resistance) and deformability (E, Pressiometric Module) are low, a grained material called ballast compacted using a vibrating needle (**Figure 2**).

Ballast columns are generally distributed in the form of a regular mesh under structures usually designed on a [11].

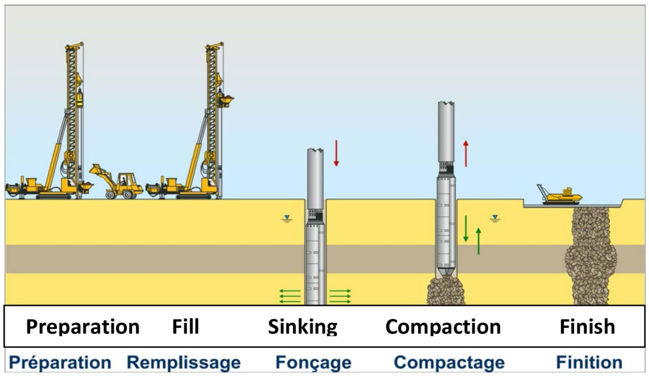

Figure 2. Ballast column placement process [11].

The assumptions regarding the choice of deformation modulus, diameter and length of the columns (**Figure 3**) depend in part on the chosen implementation material and its performance [12]. However, in the case of non-floating columns, the length depends on the depth of the substrate.

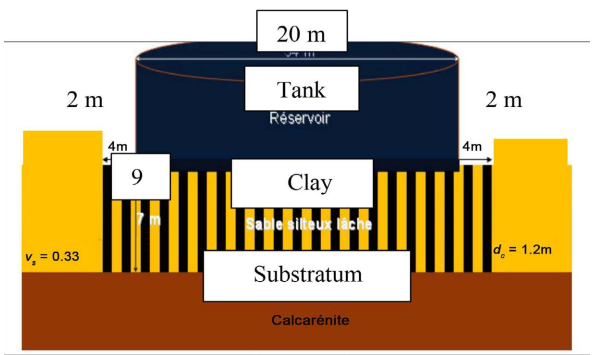

Figure 3. Reservoir on clay soil reinforced by ballast columns.

The choice of a method of improvement is always accompanied by an objective expressed in terms of incorporation rate, the number of landfills and bearing capacity is determined by the characteristics of the soil in place. This requires the carrying out of surveys, identification tests and soil characterization.

The geotechnical study program is summarized as follows:

- The identification was possible thanks to the observation of samples obtained by carotted samples.
- The calculation of bearing capacity by method C and ϕ requires the performance of the direct shear test and determination of the density weight.
- The calculation of settlement requires the odometric test.

In parallel, the pressure-measuring test [13] and the penetrating test [14] were carried out to allow in situ estimation of bearing capacity and compaction.

2.2. Method of Sizing and Checking Ballast Columns

The method of sizing used is that of Priebe (1976-1995) [15] which introduces the notion of overall improvement of the mechanical characteristics of the treated medium and corrections inherent to the relative compressibility (soil-column) and the effect of depth. Priebe was based on the principle of the unit cell with the assimilation of elastic deformations of the soil surrounding the column to that of a thick tube, The same drained characteristics as the compressible soil.

Once the dimensioning is validated, it will be necessary to check the internal loads corresponding to:

- the lateral expansion rupture by

$$q_{re} = \sigma_r * \tan^2\left(\frac{\pi}{4} + \frac{\phi'c}{2}\right) \tag{1}$$

- Punching rupture by

$$q_{rp} = 9 \cdot C_u + L_c \cdot \left(2 \cdot \frac{C_u}{R_c} - \gamma_{col}\right) \tag{2}$$

where,

C_u represents the undrained cohesion of the soil.

L_c: length of the column.

R_c: average radius of the column.

σ_r : represents the radial embrace.

Within the case of pressiometric test

$$\sigma_r \approx Ple^* \tag{3}$$

where,

$$\sigma_r \approx 4 \cdot Ple^* \left(s_i \phi' c \geq 38^\circ\right) \tag{4}$$

Ple^* = min (Ple^*[z]) on column height in each layer.

The vertical stress of rupture q_r in the column is equal to:

q_r = min (q_{re}; q_{rp}; 1.6 MPa).

In this case, the following relationships can be established:

$$C_u \approx \frac{Pl^*}{5.5} \text{ when } Pl^* < 0 \tag{5}$$

$$C_u \approx \frac{Pl^*}{10} + 0.025 \text{ when } Pl^* \geq 0.3 \text{ MPa} \tag{6}$$

The weighting coefficients of 2 and 1.5 are assigned in the calculation to the ULE and the SLE respectively.

The settlement shall be calculated by the method of homogenization under a uniform load [16] where ballasted columns are stopped on a more compact layer.

After the construction of the ballasted columns, the settlement of each layer *i* in the center of the work is written

$$w_i = h_i \cdot \sigma_t \, a_i \cdot E_{col} + \left\{ (1 - a_i) \cdot E_{si} \cdot \frac{1 - \nu_{si}}{1 - \nu_{si} - 2\nu_{si}^2} \right\} \tag{7}$$

The value of the constraint in the column at layer *i* (σci) can be given by:

$$\sigma_{ci} = E_{col} \cdot \frac{\sigma_t}{a_i} \cdot E_{col} + \left\{ (1 - a_i) \cdot E_{si} \cdot \frac{1 - \nu_{si}}{1 - \nu_{si} - 2\nu_{si}^2} \right\} \tag{8}$$

where:

a_i: percentage of incorporation (ratio of sections), in the layer *i* considered;

E_{col}: Young's module of the column;

E_{si}: Young's module of the layer *i* considered;

ν_{si}: Poisson coefficient of the layer *i* considered;

σ_t: mean vertical stress provided by the structure;

h_i: layer thickness *i*.

In the case where pressure tests are available

$$w_i = \frac{h_i \cdot \sigma_t}{a_i} \cdot E_{col} + \left\{ (1 - a_i) \cdot \frac{E_{Mi}}{\alpha_i} \right\} \tag{9}$$

The value of the constraint in the column at layer *i* (σ_{ci}) can be given by:

$$\sigma_{ci} = E_{col} \cdot \frac{\sigma_t}{a_i} \cdot E_{col} + \left\{ (1 - a_i) \cdot \frac{E_{Mi}}{\alpha_i} \right\} \tag{10}$$

With:

E_{Mi}: pressiometric module of the layer *i* considered.

α_i: Coefficient linking the Eeod of the layer *i* considered.

In order to better visualize the internal behavior of the column and its interaction with the soil, a numerical simulation is performed under the software Plaxis 2D.

2.3. Main Steps of Numerical Simulation Using Plaxis 2D

Operations on PLAXIS can be divided into three main parts which are **Pre-processing** (using the Input subroutine), **Calculations** (using the Calculations subroutine) and **Post-processing** (using the Output subroutines and Curves subroutines). First, the initial soil model (without inclusion) is defined in order to know the real behavior.

The purpose of pre-processing is to create our model on Plaxis 2D. This involves defining our problem (physical representation), followed by defining the construction procedure corresponding to the creation of the physical model with dimensions 30 × 15 m with defined boundary conditions (**Figure 4**). The model considered is an axisymmetric with a six-nodded triangular element mesh [17].

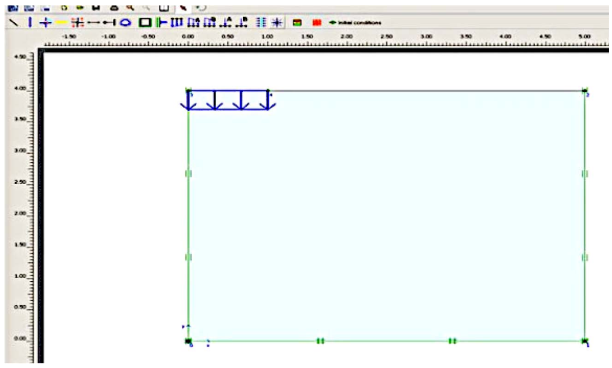

Figure 4. Plaxis input window.

When defining the construction procedure, it is up to us to establish the initial situation and the different stages of the construction.

This is followed by the definition of the material properties (**Figure 5**).

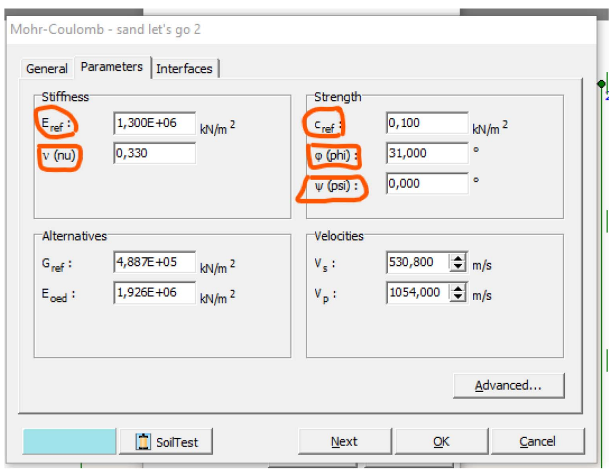

Figure 5. Material geotechnical properties sets.

In this case, the procedure is as follows:

The definition of the characteristics of the initial material and the chosen behavior law corresponding to that of Mohr-coulomb which is adapted to the available data [18].

The characteristics of our terrain model on the geometric, physical and geotechnical plane are made from the following data (**Table 1**).

Table 1. Parameters used to create our geotechnical model.

Length (m)	50
Width (m)	10
Level of the table (m)	3
γ_h (kN/m^3)	16.3
γ_{sat} (kN/m^3)	18.25
C (kPa)	58
ϕ (°)	19
E (KPa)	500
Ψ (Degree)	0

After material definition, application of loading follow. In the case of this present study, the foundation is considered to be rigid. To this end, the settlement of our foundation will be simulated as a uniform vertical downward movement [19].

In modelling consolidation, it is important to consider boundary conditions. This means specifying how the load is applied, soil drainage conditions, and other factors that influence the consolidation process. In principle, all boundaries must have one boundary condition in each direction. That is to say, when no explicit boundary condition is given to a certain boundary, the natural condition applies, which is a prescribed force equal to zero and a free displacement. To avoid the situation where the displacements of the geometry are undetermined, some points of the geometry must have prescribed displacement [17]. The simplest form of a prescribed displacement is a fixity (zero displacement), but non-zero prescribed displacement may also be given. When the geometry model is complete, the finite element model (or mesh) can be generated. Plaxis allows for a fully automatic mesh generation procedure, in which the geometry is divided into elements of the basic element type and compatible structural elements, if applicable.

The following step is the calculation phase which contains all the elements to define and initiate a finite element calculation. It is to choose the project for which the calculations will be defined (**Figure 6**). Each calculation step is characterized by a series of iterations to reduce the solution balance errors.

Then the Output subroutine contains several very important menus such as the deformation menu that allows the visualization of the different deformations through options like Total Displacements, Total Deformations. The Constraints (stresses) menu It allows us to visualize the total and effective stresses in our model (**Figure 7**).

Finally, the Curves subroutine is used to display stress-strain, load-displacement or time-displacement curves, stress paths or strain of selected points in geometry. These curves represent the evolution during the different phases of calculation, which gives an overview of the global and local behavior of the soil. The

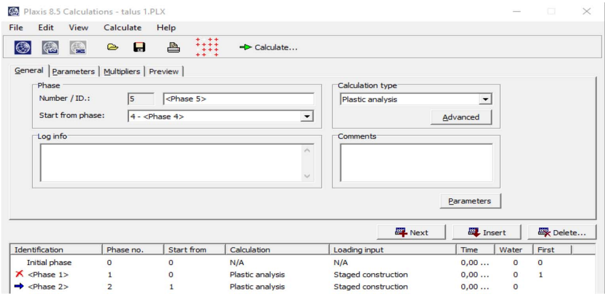

Figure 6. Plaxis calculation window.

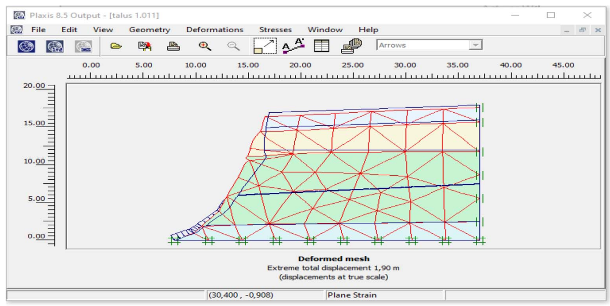

Figure 7. Plaxis output window.

points whose curves are generated must be selected in the calculation program before starting the calculation process.

3. Results and Discussions

3.1. Results of the Simulation

The settlement representing a uniform vertical displacement to the base will be simulated by the "Prescribed displacement" option (Figure 8).

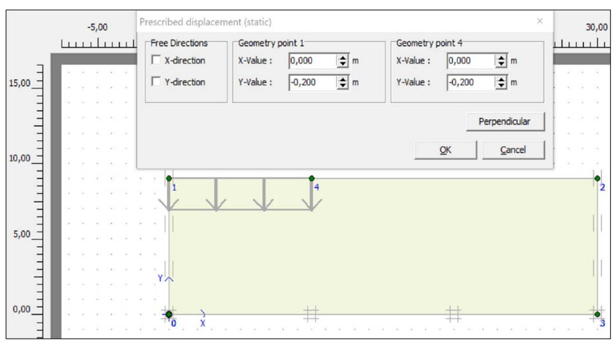

Figure 8. Value (20 cm) of the prescribed displacement.

The idea of fixing a vertical displacement of 20 cm is to start from the allowable base to obtain the minimum stress that can induce this settlement

(Figure 9) and compare it to the load applied by the storerver.

To achieve a 20 cm settlement on our model, a force of 2168.1 kN was applied. In 2D plaxis, for axisymmetric models the force Y is expressed as a unit of force

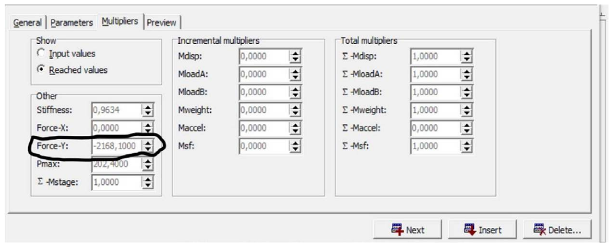

Figure 9. Determination of the force required to achieve a vertical displacement of 20 cm.

per radiant (**Figure 5**). Therefore, in order π to obtain the total force necessary to produce a displacement of 20 cm under our foundation, it is necessary to multiply the force obtained on plaxis by 2, so that:

- Force Y totale = 2168.1*2π = 13622.57 KN.
- The stress under the foundation = 13622.57/102π = 43.36 kN/m^2.

The stress obtained by simulation being lower than the load provided by the net (150 kPa), it is clear that our reservoir induces unacceptable deformations in this soil.

This translates into the deformed mesh model below (**Figure 10**).

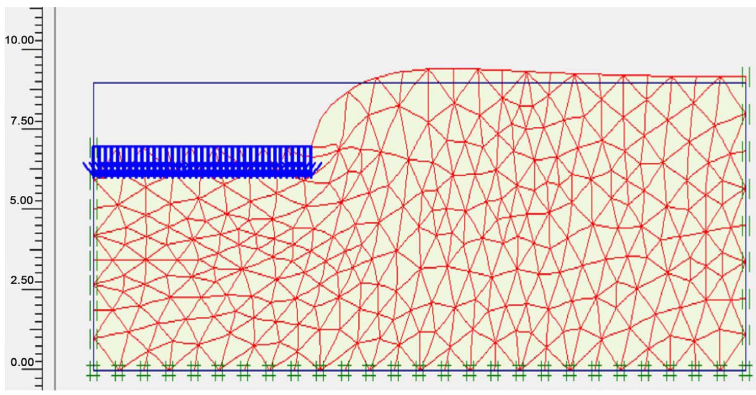

Figure 10. Mesh deformed after application of a stress of 43.36 KN/m^2.

We observe deformations at the level of the ground underlying our structure (**Figure 11**).

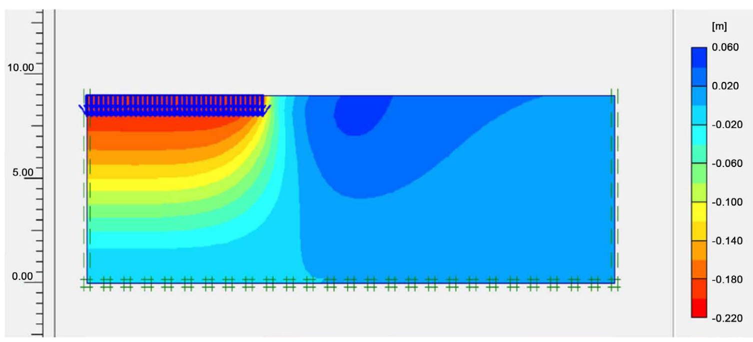

Figure 11. Vertical displacement induced by a stress of 43.36 kN/m^2.

It is therefore up to us to determine the settlement produced by the application of a stress equal to 150 kPa. As before, the Prescribed displacement option is adopted on Plaxis 2D to simulate the settlement. We will prescribe a uniform displacement of 5 cm in an increasing manner on Plaxis 2D, until a displacement corresponding to the application of a stress equal to 150 kPa is reached.

The vertical deformation model obtained is shown in Figure 12.

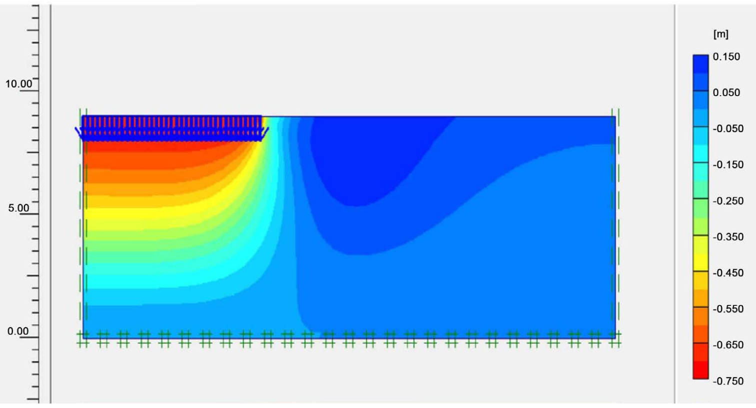

Figure 12. Vertical displacement induced by a stress of 150 KN/m^2.

Figure 12 allows to identify vertical displacements corresponding to a stress of 150 kPa such that the first under the structure which are oriented downwards are bumps which have a value of 70 cm. The second ones on the adjacent side of the structure are smaller displacements corresponding to soil uplifts.

The simulation shows that the soil in place is not suitable to support this tank. What will happen with the ballasted column improvement solution chosen? To answer this question, we present the results of the simulation obtained after improvement (Figure 13).

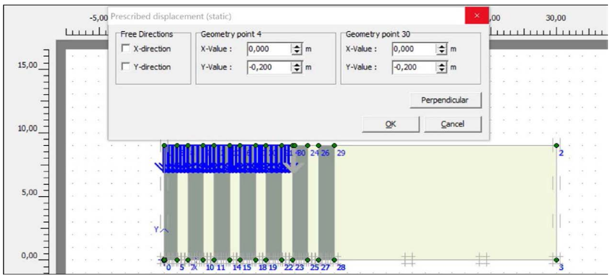

Figure 13. Floor reinforced by ballasted columns with a prescript movement of 20 cm.

The same logic of the displacement of 20 cm prescribed in the case of the floor in place was adopted. To obtain a settlement of 20 cm on our model reinforced by the ballasted flaps, a force of 13,255 kN was applied (Figure 13). In 2D plaxis, for axisymmetric models the force Y is expressed as unit of force per radiant. Therefore, to obtain the total force necessary to produce a displacement of 20 cm under our foundation, it is necessary to multiply the force obtained on plaxis by 2π, such as:

- Force Y totale = 13255 * 2π = 83283.62 KN.
- The stress under the foundation = $83283.62/10^2\pi$ = 265.1 KN/m^2.

The corresponding deformed mesh is presented in **Figure 14**.

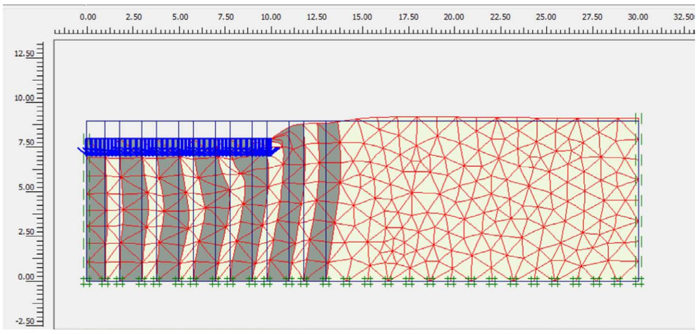

Figure 14. Deformed mesh after application of a stress of 265.1 KN/m^2 on the reinforced ground.

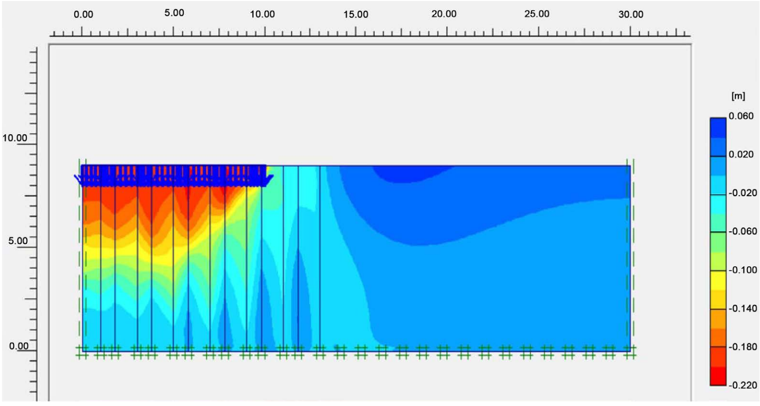

Figure 15. Vertical displacement induced by a stress of 265.1 KN/m^2.

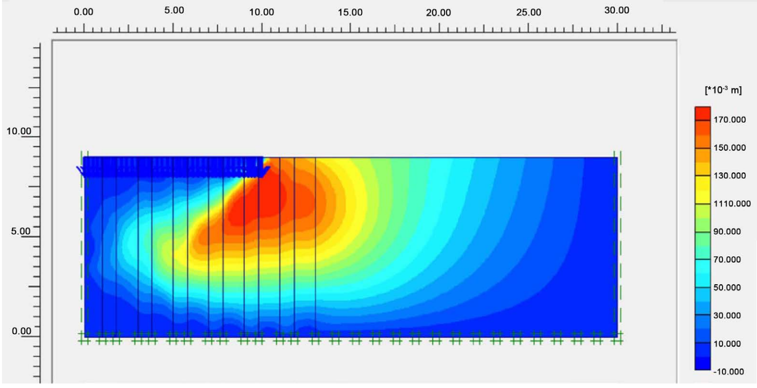

Figure 16. Horizontal displacement induced by a stress of 265.1 KN/m^2.

After applying a load equal to 265.1 kPa, a vertical settlement of 20 cm (**Figure 15**) and a horizontal displacement of 17 cm were obtained (**Figure 16**), therefore, when a stress of 150 kPa on the reinforced ground, the deformations will remain within the tolerable range, that is to say, less than 20 cm (**Figure 16**).

Potential limitations and assumptions of this simulation study include the challenge of converting stone column grids into 2D structures for modeling. Different approaches like enhanced soil parameters, embedded beam elements, and equivalent column methods are available for modeling stone columns, but selecting the most suitable approach can be complex due to limited literature guidance. Additionally, the software assumes specific material models like the Mohr-Coulomb model for soil analysis, which may not fully capture the complex behavior of soft soils under varying conditions. Furthermore, the accuracy of the analysis heavily relies on the input soil parameters obtained from laboratory tests, which can introduce uncertainties in the simulation results.

3.2. Sizing and Verification

The reservoir has a diameter of 20 meters and block measures 22 meters. The mesh under the chosen grid is a regular square-shaped mesh and characterized by a reference mesh $A_{ref} = 3.61\ m^2$ ($1.9*1.9\ m^2$) according to the CFMS recommendations [8].

Either A the surface of the right-of-way of the row and n the number of columns under this surface such that:

$A = (22/2)^2 \times \pi = 379.94\ m^2$;

n = 379.94/3.61 = 106 columns.

The effective breaking stress q_{re} by lateral expansion is:

$$q_{re} = 0.375 \cdot \tan^2\left(\frac{180}{4} + \frac{19}{2}\right)$$

$$q_{re} = 0.737\ \text{MPa} = 737\ \text{kPa}$$

The Constraint at SLE is:

Qa,ELS = 737/2 = 369 KPa

The Constraint at ULE is:

Qa,ELU = 737/1.5 = 491 KPa

For settlement calculation, the homogenization method is chosen (**Table 2**):

Table 2. Parameters for the calculation of improved soil compaction.

E_{col} (kPa)	60,000
E_{sol} (kPa)	500
H (m)	9
A (m^2)	0.58

$$S = \frac{E_{col} \cdot H}{a \cdot E_{col} + \left\{(1-a) \cdot E_{soli}\right\}}$$

The settlement obtained by the homogenization method is of the order of 15 cm. The homogenization method is also used for the load-bearing capacity (**Table 3**).

Table 3. Parameters used for the calculation of the stress applied to ballasted columns.

E_{col} (KPa)	60,000
E_{sol} (KPa)	500
σ_t (KPa)	150
A (m^2)	0,58

$$\sigma_c = \frac{E_{col} \cdot \sigma_t}{a \cdot E_{col} + \left\{(1-a) \cdot E_{sol}\right\}}$$

$\sigma_c = 257$ KPa.

4. Conclusions

The main problems related to soils are generally manifested by a low bearing capacity, deformations (absolute or differential settlement) important under static loads, or dynamic especially for soft clay soils. The choice of type of foundation depends on the load provided by the structure and the bearing capacity of the soil. If the soil under the planned structure is excessively settled, either deep foundations or soil treatment by improving its mechanical characteristics.

It is in this context that this paper is part of the study of strengthening a soft soil (soft clay), intended to receive the slatted of a reservoir, by ballasted columns. The objective was to calculate and verify the permissible stress and settlement of the slab on ballasted columns by an analytical and numerical approach.

This project was a comprehensive and integrated exploration of soil reinforcement techniques with a particular focus on ballasted columns. It is a pragmatic approach, combining theoretical knowledge with practical applications. The multidisciplinary approach, combining geology, geotechnics, modelling and sizing, reflects the intrinsic complexity of real-world geotechnical projects.

The analytical calculation using pressiometric test values revealed that the loads brought by the structure caused a settlement of the order of 57 cm.

For the numerical study, the software (PLAXIS 2D) allowed us to modelize the initial soil supporting the tank. From this situation, it appeared that a transmitted load of 150 kPa induces a vertical settlement equal to 70 cm largely over the permissible value of 20 cm.

In light of the results obtained after using two different methods, we can easily say that an improvement of the initial soil was essential for the implementation of our reservoir.

The numerical analysis after soil reinforcement by ballasted columns modelized using Plaxis 2D allowed us to determine the provisional settling of the columns after the construction of the tank. It is shown that for a load of 265.1 kPa, we observed a vertical settlement of 20 cm and a horizontal displacement of 17 cm. Thus, we can infer that the application of a stress of 150 KPa on our reinforced soil will keep us in the range of tolerable deformations by our reservoir, that is, less than 20 cm.

For analytical situations after reinforcement, the calculation consists of a verification of the constraints inside the columns compared with the allowable constraints to SLE and ULE cases. This revealed a favorable situation because of the fact that the obtained constraints are inside the limits. Given the high lateral expansion value obtained in the presence of ballasted columns, it would be prudent to ensure a sufficient lateral grip to prevent lateral expansion failure.

For a thorough study of this document, we propose in perspective a comparative study with the other reinforcement methods as vertical drains or rigid inclusion.

Conflicts of Interest

The authors declare no conflicts of interest regarding the publication of this paper.

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