ANALYSIS OF ARAQUARI SAND BEHAVIOR THOUGHT NUMERICAL MODELING OF TRIAXIAL TESTS

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Abstract. The present paper describes the numerical evaluation of triaxial tests to assess the constitutive model of better adaptation to a set of laboratory data. The data considered is part of an experimental campaign carried out on samples of sandy composition, from the Araquari/SC experimental testing site. To model the triaxial data, an axisymmetric representation of the triaxial specimen was designed in the finite element software Plaxis. Applications of the Mohr Coulomb (MC) and the Hardening Soil Model (HS) were evaluated by a direct comparison between laboratory results and numerical predictions. Considering different confining pressures, both models were able to predict with good accuracy the maximum bearing load. The greatest discrepancy was observed in the prediction of strain behavior: as expected, the hardening soil model was more adequate to represent the non-linear behavior of the sand. As the main scope of the research developed at the Araquari Experimental Testing Site is the investigation of the complex pile-soil interaction mechanisms, after the analysis of the constitutive model that better represents the triaxial tests, the hardening model was used to predict the behavior of a reference pile.

Keywords: Araquari Sand, Plaxis, Finite Elements, Triaxial Test.

1 Introduction

The triaxial apparatus is a common and effective test used for the study and identification of stress strain behavior of soil and rocks. In a typical triaxial test a cylindrical specimen is initially confined in a pressurized cell to simulate a stress condition and then increments of vertical stresses are applied. The evolution of deformations with the applied stresses can be used to characterize modulus, and maximum values of mobilized stresses to define strength (shear strength properties of the sample). Details of classical procedures can be found on Bishop and Henkel [1] and Head [2].

When performing triaxial tests in granular material, as undisturbed samples are difficult to obtain, the consideration of different sets of specimens with different molding densities are useful to understand possible changes in field behavior. Finding the constitutive model that better represents the expected behavior is a key part on this kind of analysis. In this context, the present paper describes the numeric modelling of a set of triaxial tests presented in Neto [3], to evaluate the adaptation of two different constitutive models, using the Plaxis finite elements software. Thereby, this paper has the objective of demonstrating the accuracy of the proposed numerical analysis to represent the laboratory data. After the analysis of the constitutive model that best represents the triaxial tests of the Araquari/SC sand, a reference pile of the site was modeled to evaluate the expected field behavior.

2 Modeling triaxial tests

2.1 Constitutive Models

Soil constitutive models have advanced significantly, from the basic models that idealize soil as a linear elastic medium or a perfectly plastic linear elastic medium to sophisticated elastoplastic approaches considering hardening and softening (Rebolledo [4]). Constitutive models have the function of reproducing, interpreting, and predicting the stress strain behavior of a given material. Depending on the evaluated material, this behavior can be very distinct and difficult to represent (Dias et al. [5]). Additionally, the idealization of the constitutive model must also observe the ease of replication and the viability of the test executions used to define the input parameters.

In this context, the present paper described the use of two elastoplastic models available in the software Plaxis, the Hardening Soil Model (HS) and Mohr Coulomb Model (MC) to represent the behavior observed in a set of laboratory tests executed by Neto [3]. The main features of the related models are presented below.

2.1.1 Hardening Soil Model (HSM)

According to Teixeira [6] the HS model was developed based on the relationships described by the classical hyperbolic model of Ducan and Chang [7], but differs from it, as it is based on the theory of elasticity and considers the phenomenon of dilatancy. The basic idea of HS is the hyperbolic relationship between vertical strain (ε_1) and deviatoric stress (q) in primary triaxial loading (Schanz [8]): when subjected to primary deviator loading, the soil stiffness decreases, and simultaneously irreversible plastic strains increase. The approximation proposed by the HS model in the software Plaxis to represent this idealized behavior is presented in eq. (1).

$$\varepsilon_1 = \frac{q_a}{2E_{50}}\left[\frac{\sigma_1 - \sigma_3}{q_a - \ \sigma_1 - \sigma_3}\right] \text{ for } q < q_f. \tag{1}$$

The ultimate deviatoric stress q_f and the quantity q_a are defined as:

$$q_f = \frac{6\sin\phi}{3-\sin\phi}\ \ p + c\cot\phi \ \ ;\ q_a < \frac{q_f}{R_f}. \tag{2}$$

As can be seen q_f is derived from c and ϕ, cohesion and friction angle, respectively. The Mohr Coulomb failure criterion is suitable for setting these parameters (Schanz et al.[8]). The failure is reached, and plastic yielding occurs when $q=q_f$. Parameter q_a is defined by considering a failure ratio R_f, that is suggested to be smaller than 1. $R_f=0.9$ is the default setting. The hyperbolic relation described is plotted in Fig. 1. Parameter E_{50}, E_i and E_{ur} are elastic modulus defined as:

- E_{50} is the confining stress-dependent stiffness modulus for the primary load:

$$E_{50} = E^{ref}_{50}\left(\frac{c\cos\phi + \sigma'_3\sin\phi}{c\cos\phi + p^{ref}\sin\phi}\right)^m. \tag{3}$$

- E_{ur} is the confining stress-dependent stiffness modulus for unloading and reloading conditions:

$$E_{ur} = E^{ref}_{ur}\left(\frac{c\cos\phi + \sigma'_3\sin\phi}{c\cos\phi + p^{ref}\sin\phi}\right)^m. \tag{4}$$

- E_i is the initial stiffness related to E_{50} by:

$$E_i = \frac{2E_{50}}{2-R_f}. \tag{5}$$

where E_{50}^{ref} is the reference secant stiffness modulus for the drained triaxial test, E_{ur}^{ref} is the reference stiffness modulus for unloading and reloading conditions (by default $E_{ur}^{ref} = 3E_{50}^{ref}$), σ^{ref} is a reference isotropic stress (100 kPa by default in the software), and m is the exponent that defines the strain dependence value of the stress state. In natural soil, the exponent m varies between 0.3 and 1.0 (Rebolledo et al. [4]).

Details of the yield surface, plastic flow rule and plastic potential functions can be seen in Schanz et al. [8].

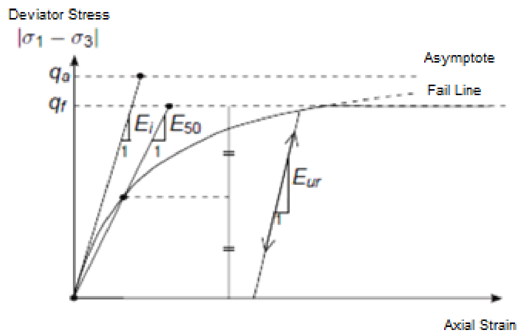


Figure 1. Stress x Strain Hyperbolic Curve (Plaxis [9])

2.1.2 Mohr-Coulomb Model (MC)

For comparison with the Hardening Soil model, the Mohr-Coulomb (MC) elastoplastic model was also considered for analysis. The MC model is a linear elastic model with yielding defined by the Mohr Coulomb failure criteria. Elastic parameters are defined by constant values of the Young´s module and Poisson´s coefficient, obtained by usual laboratory tests. For failure characterization, cohesion, friction angle and dilatancy can be considered.

2.2 Geometric model and Boundary Conditions

To simulate a triaxial test on the Plaxis Software, using either of the constitutive models, it´s necessary to define and appropriate geometry and boundary conditions. The numerical modeling of the triaxial tests was performed considering an axisymmetric geometry of the problem, with 1m x 1m extension. Figure 2 shows the geometric section and the finite element mesh, and the boundary conditions considered.

The dimensions used in the model were those recommended in literature of 1m x 1m according Surarak [10]. The applied contour conditions are also presented in Fig.2 and refer to the restriction of displacements on the lower and left side faces and load systems as illustrated.

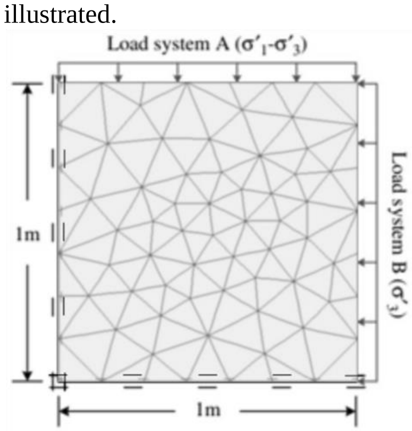


Figure 2. Finite Elements Mesh.

3 Araquari Experimental Testing Site

Originating from a partnership of several institutions such as the Federal University of Paraná (UFPR), The Federal University of Rio Grande do Sul (UFRGS) and the State University of Santa Catarina (UDESC), the Araquari experimental field has as its main objective to study the complex pile-soil interaction mechanisms. The soil of the Araquari experimental field is predominantly composed of sand (with less representative silty and clayey layers). In the experimental field, six (6) instrumented piles, ET01 to ET06, were executed, and loading tests were carried out. A more comprehensive description of the experimental field can be found in Brochero [11], Nienov [12], Lavalle [13], Sestrem [14] and Neto [3]. In the present work a set of triaxial tests performed on samples collected near pile ET06 were used as reference. These results are part of the experimental campaign of Neto [3]. A general soil profile for the related region can be seen in Fig. 4.

4 Simulations

4.1 Applying the Hardening Soil Model (HS)

Using data from Neto [3] with the necessary changes to perform the simulations within Plaxis comes Tab.1 of input data, where each row of the table represents a laboratory test with different confining pressure. Parameters presented on Tab.1 and results on Fig. 3 were derived from samples collected at 22m depth. Similar results were obtained from modeling the data from 5, 8.5 and 12 m depths collected samples, see Pacheco [15].

The parameters considered were set as follow: the applied confining pressure were used according to the original data of Neto [3]; the friction angle was reinterpreted; cohesion was set as a minimal value to avoid numerical instabilities; and the Elastic Modulus of 50% ($E_{50\%}$) was calculated from the stress strain graphs presented in Neto [3] and the other elasticity modules were automatically calculated by the software. Default parameters were set as described in 2.1.1.

Results are presented in Fig.3a along with the laboratory data for a direct comparison. From the related figure it possible to observe that in general the model can predict the stress strain non-linear behavior of the developed laboratory results with good agreement. Considering strength values interpretation, the peak deviation stress values (σ_1-σ_3) of the simulations were 139 kPa, 283 kPa and 438 kPa for the confining pressures of 50 kPa, 100 kPa and 150 kPa, respectively, while for the laboratory test the values were 144 kPa, 276 kPa and 442 kPa for the same confining stresses, translating into 3.5%, 2.5% and 0.9% variation respectively.

4.2 Applying the Mohr-Coulomb Model (MC)

Parameters considered when modeling MC behavior were also displayed on Tab.1. In this case only constant values of $E=E_{50}$ and Poisson coefficients (ν) were considered. The Poisson coefficient (ν) was set as a standard value of 0.3, this value is within the range for the simulation of tests with loading as described in Plaxis [9]. The results of the simulations and laboratory data are presented together in Fig.3b, aiming for a direct comparison. The peak deviation stress values (σ_1-σ_3) were 143 kPa, 285 kPa and 428 kPa for the confinement stresses of 50 kPa, 100 kPa and 150 kPa, respectively. The results of the laboratory testing for the same confining stresses were 144 kPa, 276 kPa and 442 kPa.

The greatest discrepancy was observed between the simulation and the laboratory result obtained for the confinement stresses of 100 kPa and 150 kPa. In these stresses, the difference between numerical and laboratory results was 3%. For the confinement stress of 50 kPa the difference was only 1%. Despite the low discrepancies observed in the results obtained by the MC and tests results of strength, it is evident that the non-linear behavior of the stress-strain curves of the laboratory tests is not captured by the model, therefore highlighting the importance of the choice of a constitutive model that is closest to the soil model under study: for Araquari sand, the HS model proved to be more suitable.

Table 1. Input parameter for HS and MC

Depth (m)	Confining Stress	Friction Angle (°)	Cohesive Coefficient	E_{50} (kPa)	E_{oad} (kPa)	E_{ur} (kPa)	ν(nu)
22	50 kPa	36	0,1	3500	3500	10500	0,3
22	100 kPa	36	0,1	5750	5750	17250	0,3
22	150 kPa	36	0,1	9375	9375	28125	0,3

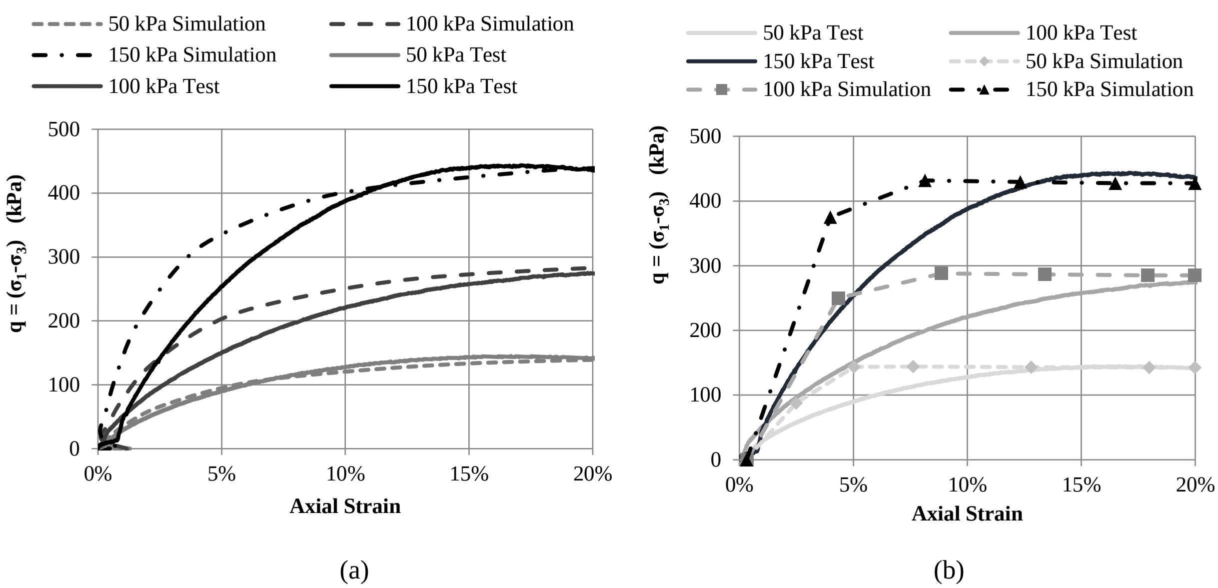


Figure 3. Stress x Strain: (a) HS and (b) MC

4.3 Concrete Pile Model

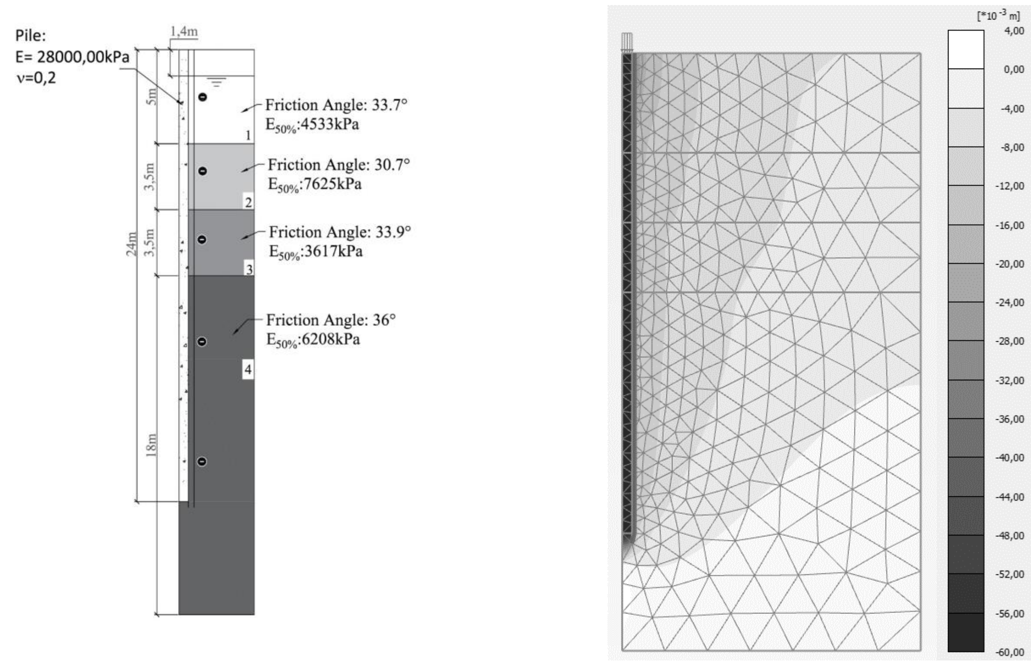
A geometric model of a pile inserted in a soil profile defined for the region near station 6 (ET-06) of Araquari field was modeled in software Plaxis. The pile was modeled as linear elastic and the soil near the station was represented by the HS model. As can be seen in Fig. 4, four (4) soil sandy layers were modeled based in the representation presented in Neto [3]. The input data required for the pile modeling and the results obtained are presented in Fig. 4, Fig. 5, and Tab.2.

Pile ET-06, has a diameter of 1 meter and area of 0.79m² and was subjected to 6 increments of load until reaching 3500 kN with 5.7 mm displacement, no failure was characterized. The same load increments applied in the field were used in the numerical model to assess load capacity for the pile and displacements. Results are presented on Fig. 5.

Observing Fig.5, it is possible to verify that the result considering the original derived parameters defines failure at 2000 KN loading, with 59 mm displacement, under predicting pile capacity and over predicting settlements.

An attempt to adjust the input elastic modulus was proposed to approximate the stress-strain relationship, these results are also shown in the Fig. 5. It is observed that a 10 time's increment of the modulus was needed to provide a better approximation.

It is important to highlight that only the modulus of elasticity E_{50} was changed in this analysis because it was considered as an average value in each evaluated layer. Keeping the same analogy of average modules, better results could be found with a better discretization of the soil layers, which was not the purpose of the present work. Furthermore, discrepancies are justified due to the soil pile interaction mechanism. Studies seeking to characterize this interaction continue to be developed for the experimental field.



(a) (b)

Figure 4. Pile Model on Plaxis (a) general view (b) deformed result

Table 2. Input data for the pile on EST6

Diameter (m)	Area (m²)	P (kN)	Settlement (mm)
1,00	0,79	544,00	6,00
1,00	0,79	1025,00	11,75
1,00	0,79	1501,00	18,43
1,00	0,79	1997,00	59,67
1,00	0,79	2012,98	59,91

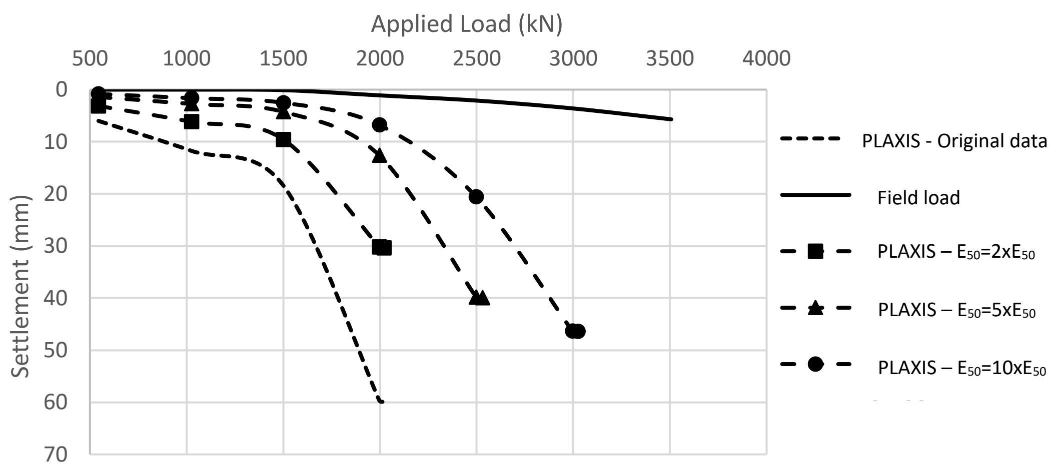


Figure 5. Load x Settlement Graph

5 Conclusions

The results of the simulations suggest that the Plaxis 8.2 program allows satisfactorily evaluation of soil behavior for both models analyzed, since both the HS and the MC presented peak deviation stress results very close to the laboratory tests results.

Among the constitutive models analyzed in the present paper, it is noted that the HS presented simulations more accurate with the laboratory tests for the non-linear behavior, thus inferring that the Hardening Soil Model is the constitutive model that best represents Araquari sand soil behavior among those analyzed.

The pile model simulated in Plaxis resulted in strains up to 54, 25 times bigger when compared to the in-situ load tests, demonstrating that the simulation was not representative of the actual soil-pile behavior. It is indicated that future works seek a better discretization of layers and characterization of interface parameters.

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