Settlement's Prediction of Piles in Tropical Soil

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ABSTRACT : The prediction of deep foundation settlements remains challenging due to the scarcityofstudies about it. This paper aimed to evaluate the accuracy of the usage of load transfer methods in predicting the settlement of bored piles in granular soil in Brazil's Northeast. For this, two small piles were installed and submitted to load tests, a small amount of expansive polystyrene was placed under one of the pile's tips to evaluate the load distribuition of the elements.. For the settlement's prediction, methods based on load transfer functions, such as analytical and numerical (using RSPile and UniPile), were employed. The comparinson of the predicted values with the experimental measurements showed agreement in the elastic zone of soil's behavior for all the methods. For higher loads, discrepancies occurred. The method proposed by Massad (1992) was the most effective among the used methodologies. Using the mentioned programs, results were close to the experimental values.

KEYWORDS: Settlement prediction, Load transfer, Bored piles.

1. INTRODUCTION

In general, piles designs are based on safety criteria that account for the non-occurrence of failure of both the structural element and the soil-foundation system. However, just the safety regarding the ultimate limit state does not assure that it will perform within service limits. Thus, it is necessary to predict the settlements that the elements will suffer in order to develop a safe and economical foundations design.

Used in several methods, the knowledge of the load transfer curves of the soil allows to obtain important information for an adequate design of the foundation elements. Therefore, such information helps engineers in designing foundations not only safe but also economical. The load transfer curves use numerical methods, such as in Coyle & Reese (1966) and Cambefort Laws modified by Massad (1992), for settlements predictions of isolated piles subjected to vertical loads. Furthermore, this methodology is the basis for programs used internationally in piles designs, such as RSPile and UniPile.

Fernandes (2010) analyzed the behavior of load transfer along the lateral and the end of the pile through transfer curves, described a theoretical methodology to obtain the respective curves and, finally, presented the curves that characterize the behavior of each type of pile in a residual soil of granite. In order to simulate these curves, the author used the UniPile program.

Alves (2016) used CPT, DMT and SPT tests to predict the behavior of a bored pile with a polymer, of 1 m diameter and a 24 m length. In comparison with the results obtained in the field, using the test of static load of the instrumented pile, the predictions were mostly conservative. The load-settlements curves with averages up to 10% used Finite Elements Method and UniPile program.

Widjaja and Lilianto (2018) compared the results of load tests of a pile with 1.2 m diameter and 50.0 m length with the results obtained using RSPile. From this analysis, they observed that the settlements obtained with RSPile were significantly conservative, having a difference of 10 mm, when submitted to a load of 1600 tons.

Medeiros (2016), without having a load test for the calculation of shear stiffness, performed an analysis with the RSPile, using the constitutive method API Sand, obtaining a load x deformation curve, calculating the shear stiffness by the method of rigidity proposed by Décourt (1998), in which stiffness is the ratio between the load applied on the top of the pile and its settlement.

Perez (2014) analyzed the behavior of instrumented bored piles with 5 m length and with three different diameters (0.25 m, 0.30 m and 0.40 m) subjected to slow static load tests. That was done in order to obtain the data of the load transfer mechanism along the pile's length, and generate the curves of the first and second Law of Cambefort.

Nogueira (2004) analyzed the behavior of micropiles with 0.4 m in diameter and 12 m in length subjected to compression efforts by performing compressive load tests. The applicability of the Laws of Cambefort was proven, since, for small displacements, between 2 and 5 mm, there was full mobilization of the skin friction, and for the base, as far as it was possible to observe, greater displacements were necessary.

Due to the importance of the studies related to prediction of settlements, it is justified the validation of the predictions in piles embedded in granular soil profiles typical of those that occur in Fortaleza - CE, with the use of analytical methods and program using load transfer curves, comparing them with the values obtained experimentally.

2. SETTLEMENTS IN BORED PILES UNDER AXIAL LOADS

The load-transfer mechanism can be represented as a result of the equilibrium between the forces applied to the superstructure and the strength forces throughout the soil-foundation system, in other words, the normal force acting on the section of the pile is absorbed by the soil, along the depth (NOGUEIRA, 2004). Uniform shear stresses along the shaft and normal stresses at the pile's tip can be assumed to represent the resistance to the load applied on the top of the foundation, as shown in Figure 1(a). For the mentioned case, an axial load diagram along the shaft corresponding to the constant lateral friction is shown in Figure 1(b). For a situation in which the lateral friction is not uniform, the lateral friction diagrams and the corresponding axial load are schematically illustrated in Figure 1(c).

Another way of presenting this phenomenon is by the charge transfer functions, which are formed by the curves t-z and q-z. In these functions, the soil is considered along the shaft as a discreet set of springs (Winkler model). Thus, the load transfer between the shaft and the soil is represented by lateral springs, which the strength is equivalent to the shear strength of the soil. The load-displacement plot represents the t-z curve for a given depth. On the other hand, the vertical spring at the bottom of the pile represents the transfer of load between the end and soil below the pile. Therefore, the q-z curve is the graphic of transmitted forces by the tip and its displacement (FERNANDES, 2010).

Within this context, in perfectly rigid piles based on soil layers with constant resistance along the depth, the mobilized lateral resistance is constant along the shaft, since the vertical displacement of the element is equal for all its sections, which does not represent the real behavior. Therefore, for a better representation of the mobilization of the lateral resistance along its length, a model that represents the variation of the axial stiffness of the pile and the discretization of the layers of soil must be used.

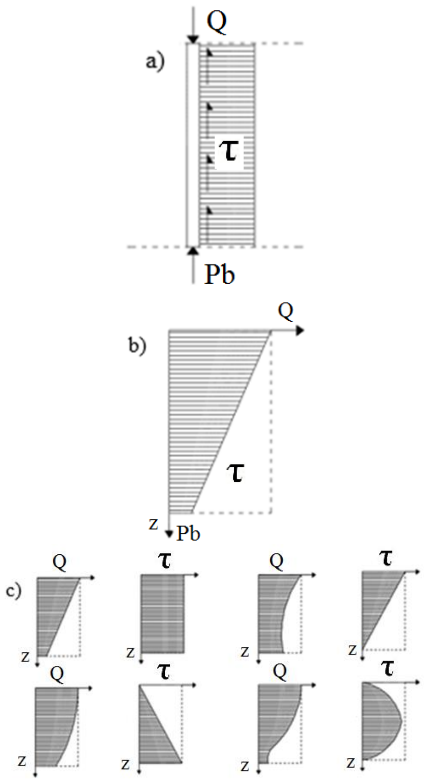

Figure 1 Load transfer mechanism: (a) Piles loads and stresses, (b) Charge-depth diagram, and (c) Diagrams for non-constant lateral friction (Source: Adapted from Vesic, 1977)

Thus, to determine the load transfer curves, t-z and q-z, instrumented load tests can be used. However, the instrumentation can be very costly or impossible due to the installation method of the pile. Thus, in the lack of this test, the literature proposes the use of dimensionless curves for the foundation design (API, 2007; ISO, 2007).

On the other hand, the load transfer phenomenon can be analyzed by the numerical method known as the Laws of Camberfort, proposed by Cambefort (1964) and simplified by Cassan (1978). These laws propose that the soil-foundation system follows the rigid elastic-plastic type relationships, both for the skin friction and for the base reactions of the piles. Thus, soil failure will occur through progressive failures, in other words, the plastification of the soil will not necessarily occur simultaneously at all points on the skin and the tip of the pile. Using the first law, the maximum displacement for lateral friction fully mobilized is calculated, this value varying from 0.1% to 0.4% of the pile diameter. By applying the second law, it is observed that the maximum displacement, in the failure, in the base generally exceeds 5% of the pile diameter, assuming values of dozens of millimeters (MASSAD, 2008).

By using these load transfer models, it is possible to predict settlement by numerical methods, such as: Coyle & Reese's (1966) method and the Cambefort Laws modified by Massad (1992). Seed & Reese (1957) introduced the load transfer functions used in this method, but, more recently, theoretical approaches have been used to evaluate the t-z and q-z curves. For the Cambefort Laws modified by Massad (1992), the development of the load-settlement curve requires well-defined stretches during the loading and unloading stages of the pile, obtained through boundary points.

With the diffusion of the use of computers, there has been a progressive use of programs and spreadsheets in foundations designs. There are several commercial programs for predicting isolated piles' settlements through load transfer methods, including RSPile and UniPile. RSPile, developed by Rocscience, uses the Finite Elements Method to analyze the behavior of piles submitted to vertical and horizontal loads. The UniPile, developed by the engineers Bengt Fellenius and Pierre Goudreault, allows the design of piles and groups of piles applying the unified method of Fellenius.

3. MATERIALS AND METHODS

3.1 Place of Study

The research site is the Experimental Field of Geotechnics and Foundations of the Federal University of Ceará, located in Fortaleza, Brazil, with an area of about 900 m², near the Physical Education Department of the UFC, as shown in Figure 2(a). Figure 2(b) shows the place where the standard penetration test was performed and the deformed and undeformed samples were collected.

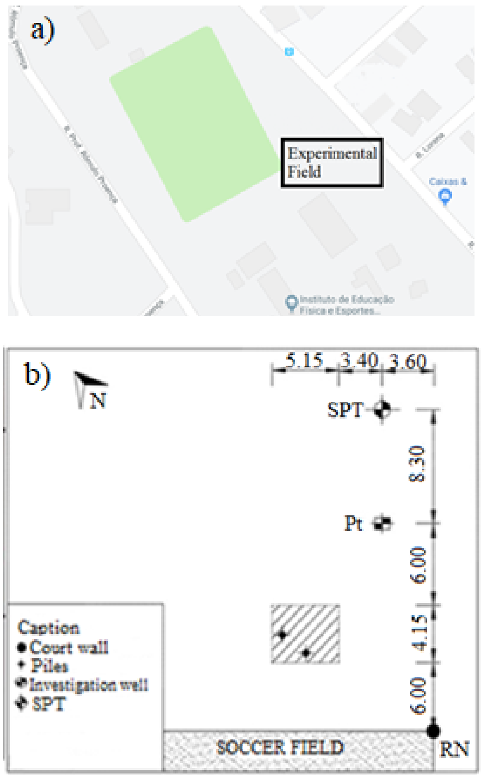

Figure 2 (a) Site map of study, and (b) Overview and points of sample collection and standards penetration tests

3.2 Data Collection

The data were collected from standard penetration tests and static load tests performed by Bonan (2017). The two piles are bored and isolated, both 1.5 m long and 0.1 m in diameter. One of them was built with the tip laid on the ground and the second was built with the polyethylene placed under its end.

The standard penetration test was performed until a depth of approximately 7.5 m. In this test, it was used a 65 kg hammer falling from a height of 75 cm and the number of blows needed to drill the standard sampler every meter in three intervals of 15 cm was counted, the N-value being the sum of the two last intervals.

The load tests were performed statically, with the quick application of load in ten stages, applying seven stages of loading and three stages of unloading. The settlement readings were performed by using two diametrically opposite strain gauges in the time intervals of 0, 1, 2, 5 and 10 min, or until their stabilization.

3.3 Samples Collection

In order to perform the necessary characterization tests of the studied soil, deformed and undisturbed samples were collected 6 m away from the piles studied. The deformed samples, weighing about 6 kgf, were collected from three different depths: 20 cm, 60 cm and 110 cm; packed in hermetically sealed plastic bags and used in geotechnical characterization tests. The undeformed sample cubes, measuring 20 cm of edge, were coated with paraffin to maintain the natural characteristics and placed in wooden boxes with sawdust to aid in transportation. The undeformed blocks were removed from the depths of 60 cm and 110 cm.

3.4 Laboratory Tests

The laboratory tests followed the procedures of the relevant technical standards in Brazil. The following tests were performed: grain size analysis, Atterberg limits, soil natural humidity, grain density and compaction test.

Following the recommendations of ASTM D3080 (ASTM, 2004), the direct shear test was performed with 50 cm³ samples, preserving the natural moisture and natural apparent specific mass of the samples. In the shear box, plates and porous stones were arranged above and below the molded specimens. The box with the specimen was subjected to flood saturation for 3 hours. The vertical stresses used for consolidation were: 50 kPa, 100 kPa and 200 kPa. In this step, the vertical displacements were measured until the stabilization of the readings, characterized by a total deformation less than 10% of that suffered in the previous loading stage. The specimens were subjected to horizontal loading at a constant velocity of 0.06 mm/min. Finally, consolidated and drained triaxial compression tests were performed on specimens with a cylindrical shape of 0.64 cm in diameter and 6.5 cm in height. In the test, a static triaxial compression equipment was used, with compressed air, in which manual valves control the pressure applied in the test body and the press is electronic. The saturation was complete when the parameter B of Skempton (1954) was equal to 1.00. The consolidation was performed with hydrostatic confinement tension of 50, 100 and 200 kPa. Finally, the failure of the specimens was achieved with the application of successive increments of axial load ($\Delta\sigma$) through the vertical displacement of the press at a constant speed of 0.03 mm/min.

3.5 Predictions of Piles' Settlements and Comparison with the Static Load Test

The evaluation of the settlements predictions was performed from the estimation of the settlements using the numerical methods of Coyle & Reese (1966) and Cambefort Laws modified by Massad (1992) and programs in which the predictions were made for each load stage of the static load tests performed, in order to compare the predictions made with the experimentally recorded.

4. GEOTECHNICAL CHARACTERIZATION OF SOIL

4.1 Characterization Tests

The grain size curves obtained from the grain size analysis performed with the deformed samples obtained at depths of 20 cm, 60 cm, and 110 cm are presented in Figure 3. According to the Unified Soil Classification System, the 3 samples tested are classified as SM, a silty sand, poorly graded. In addition, it was observed that the most superficial soil layer presents a percentage of cohesive soil slightly lower than the deeper layers.

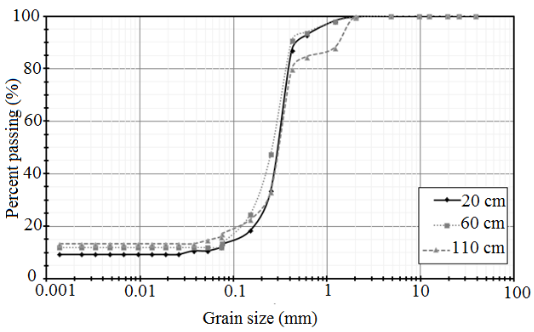

Figure 3 Grain size curve of the soil studied

The tests results of natural moisture and real density are presented in Table 1. It is observed a gradual increase in moisture with depth. However, the relative density almost did not vary with depth. Compaction tests performed for the normal Proctor energy allowed the construction of dry density curves (γ_d) versus moisture (w), as shown in Figure 4. The compaction curves are quite flat, typical of granular soils. The maximum dry densities were increasing with depth and ranged from 17.5 to 20 kN/m³. In addition, the optimal humidity is difficult to define and around 9 to 10%.

Table 1 Natural Humidity

Depth (cm)	Natural Humidity (%)	Relative density
20	4,9	2,51
60	11,3	2,54
110	12,4	2,58

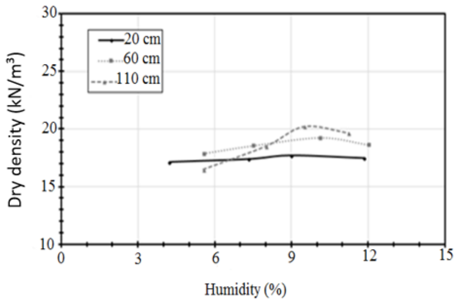

Figure 4 Compaction curves

4.2 Special Tests

Direct shear tests were performed on specimens obtained from undisturbed samples taken from depths of 60 cm and 110 cm, as previously reported. The tests were performed until a maximum horizontal displacement of 5 mm. Thus, for the normal stresses of 50, 100, 200 kN/m², shear stress versus horizontal displacement and volumetric variation versus horizontal displacement curves, and the corresponding resistance envelope were plotted. Thus, Figure 5 shows the results of direct shear tests performed to depth of 60 cm and Figure 6 to 110 cm depth.

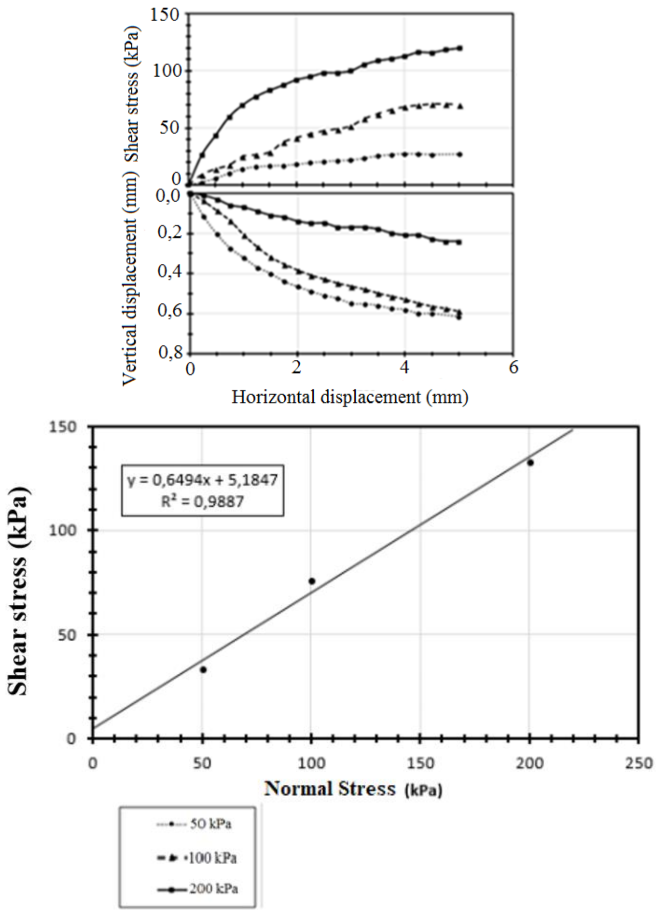

Figure 5 Direct shear test results for the 60 cm undisturbed sample

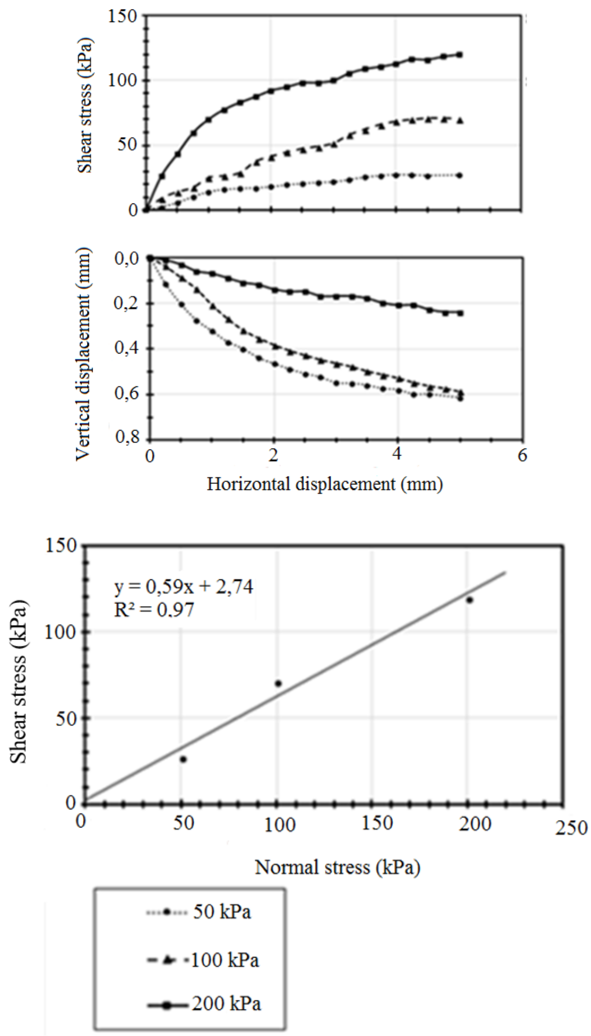

Figure 6 Direct shear tests results for the 110 cm undisturbed sample

Analyzing the envelopes of Figures 5 and 6, it is observed that there isn't almost any change of soil strength with depth. For depth of 60 cm, the friction angle and cohesion are, respectively, 33° and 5.2 kPa, and 31° and 2.74 kPa for depth of 110 cm. In addition, it is observed that these curves present maximum values without peak and reduction of the height of the specimens with the shear, typical results of loose sands.

With the undisturbed samples of the same depths, triaxial compression tests were also performed. The results obtained with triaxial (CD) compression tests for both depths are presented in Figures 7 and 8.

As in the direct shear test, the observed behavior was compatible with loose granular soils. For both samples, the calculated tangent modulus of elasticity has a value of 20 MPa for the stress range of interest of the present research, that is, σ_c equal to 50 kPa.

From the shear failure envelopes, a friction angle of 31.6° and a cohesive intercept of 7 kPa were obtained for samples taken at 60 cm and a friction angle of 33.5° and a cohesive intercept equal to 0 for the samples removed at 110 cm. Thus, by comparing the values of the parameters at depths of 60 cm and 110 cm, there is a slight increase in the angle of friction with depth and a decrease in cohesion.

Finally, by comparing the envelopes obtained from the direct shear tests and the triaxial compression tests, it is observed coherent values found in relation to the Lambe and Whitman (1969) indications, which state that the value of the friction angle obtained in direct shear tests, generally, exceeds that obtained in the triaxial test by up to 2 degrees.

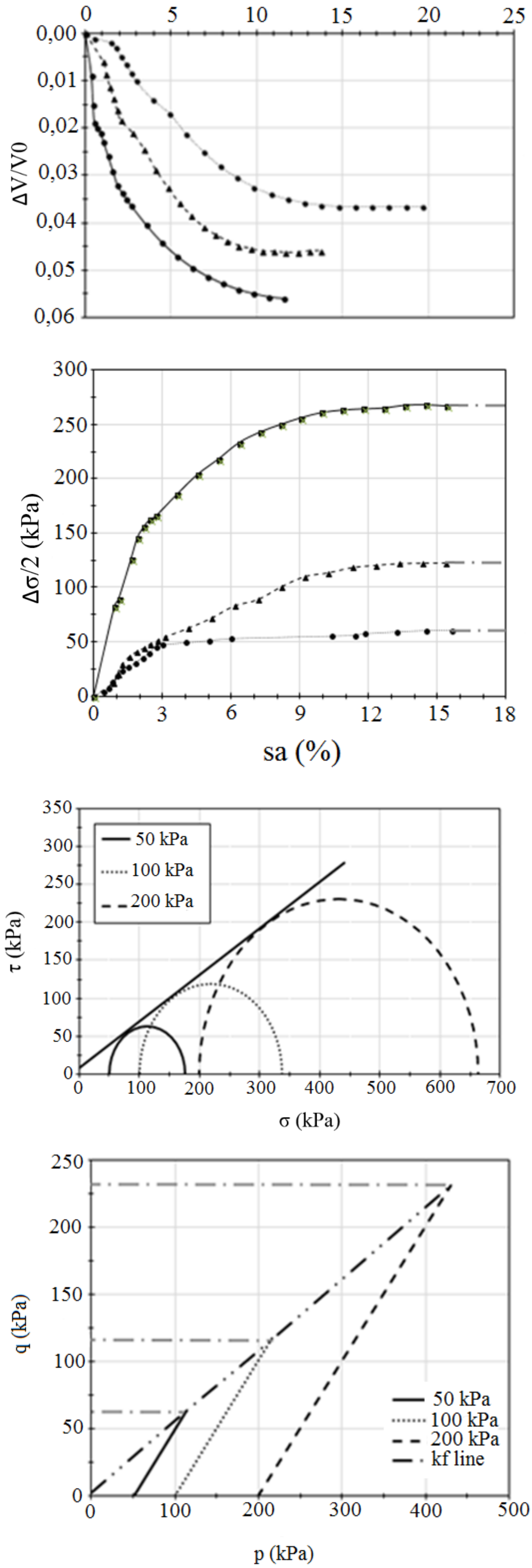

Figure 7 Triaxial compression tests results for undisturbed sample from 60 cm

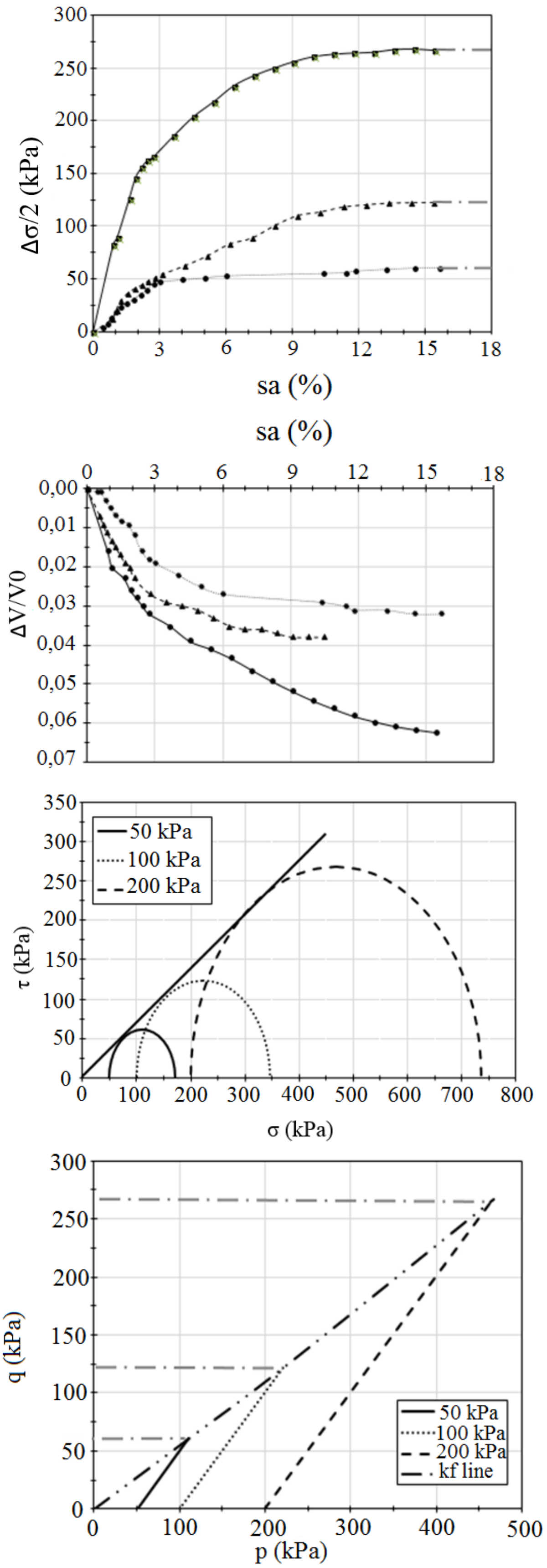

Figure 8 Triaxial compression tests results for undisturbed sample from 100 cm

4.3 Field Tests

Figure 9 shows the results of standard penetration tests and static load tests performed. Up to 4.0 m depth, the N-value ranges from 12 to 18, and that, for greater depths, the value of this parameter decreases to approximately 4 blows. The stratigraphy is composed of a thin layer of silty sand followed by a sandy silt, with the compactness varying from soft to medium compact. The water level (WL) was identified at depth of approximately 7.4 m.

In Figure 9(b), it can be observed that the maximum load applied to the load tests performed on both piles was 68.67 kN and that they showed physical failure in both tests. In the pile with end's resistance (soil under its tip), the maximum settlement was 7.12 mm and the residual one was 6.83 mm. At the pile without the end's resistance (polystyrene under its tip), the maximum and residual values measured were equal to 9.59 mm. Finally, it is observed that the pile with base is mobilized almost exclusively by lateral friction and can therefore be classified as a floating pile.

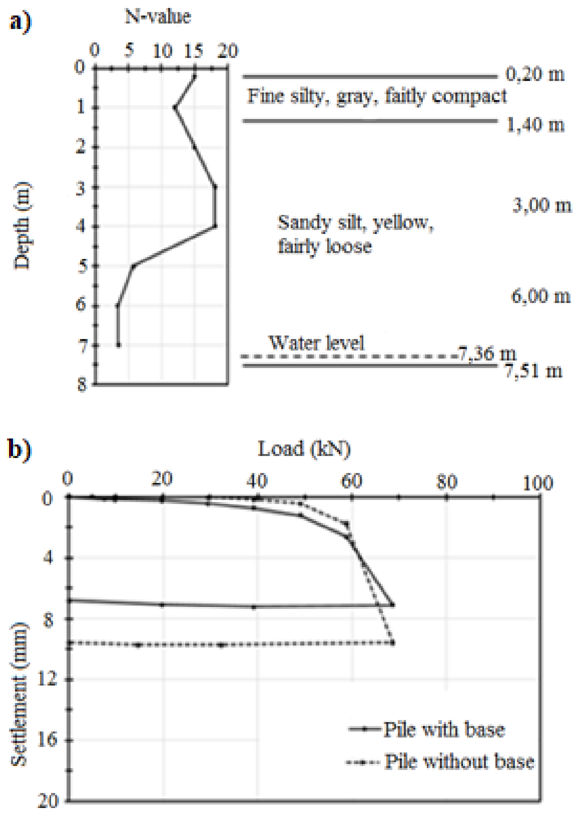

Figure 9 Field test results: (a) standard penetration test, and (b) static load test

5. RESULTS AND DISCUSSIONS

The numerical methods used to predict piles settlements were Coyle & Reese (1966) and the Cambefort Laws modified by Massad (1992). The load applied to the pile in analysis is of 20 kN and its modulus of elasticity, determined from the proposal of Cintra & Aoki (2010), was estimated in 18 GPa. For the application of the Coyle & Reese method (1966), it was used the input parameters obtained for each of the 4 forms indicated in the elements and the methodology of obtaining the soil modulus of elasticity and load transfer curve are presented in Figure 10.

From the described methodology, the displacements of the pile analyzed were estimated for loads up to 60 kN and their values were compared with the results of the static load tests (Figure 9(b)). In Figure 11, it can be observed that the methodology using parameters from the triaxial tests was the closest to the experimental curve. On the other hand, the most divergent prediction resulted from the direct shear test. For the load of 20 kN, the prediction obtained from the N-value presented result with relative convergence, shown to be an alternative to be used.

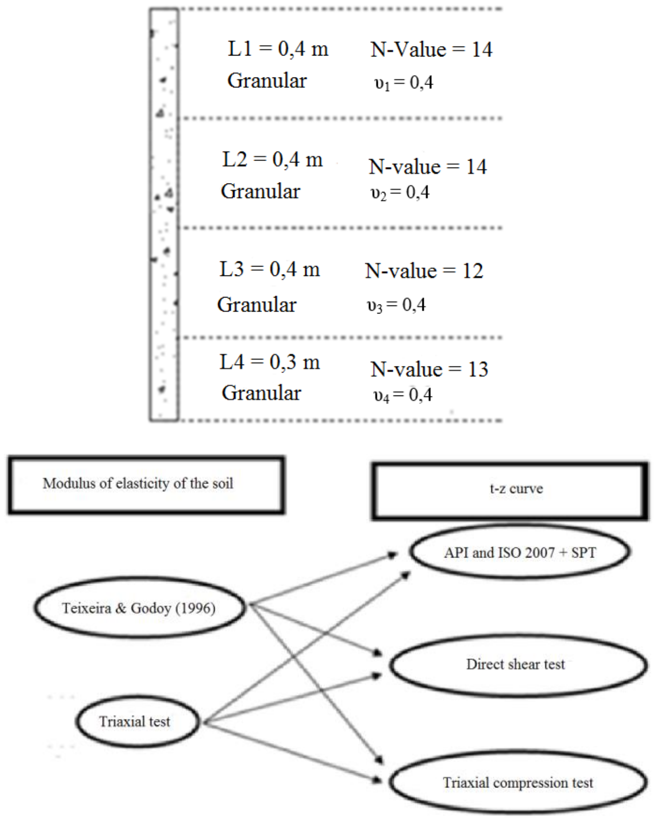

Figure 10 Coyle & Reese: input parameter of elements and adopted methodology

In Figure 12, it was also compared the predictions made with the described methodology and the experimental measurements. It is observed that, using the proposal of Massad (1992), there is a significant convergence up to 23 kN.

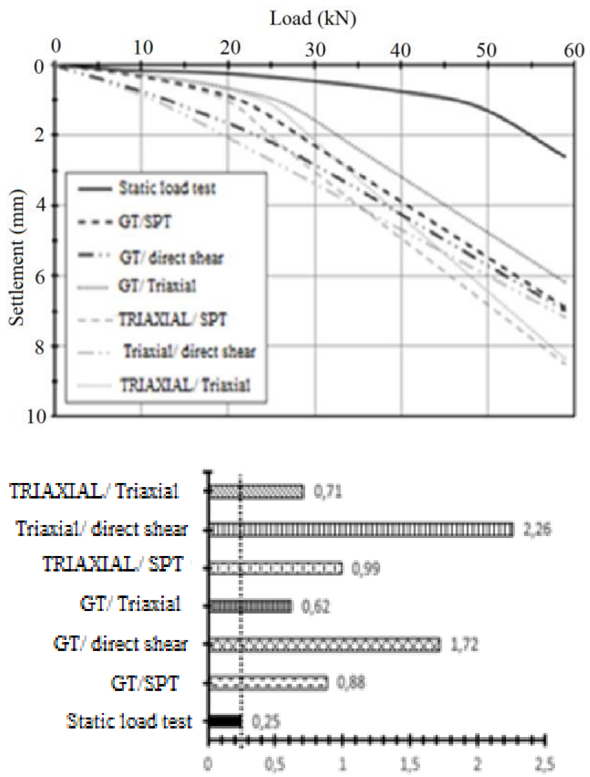

Figure 11 Comparison of the repression estimates made from the Coyle & Reese (1966) method: for the load range up to 60 kN and for the workload

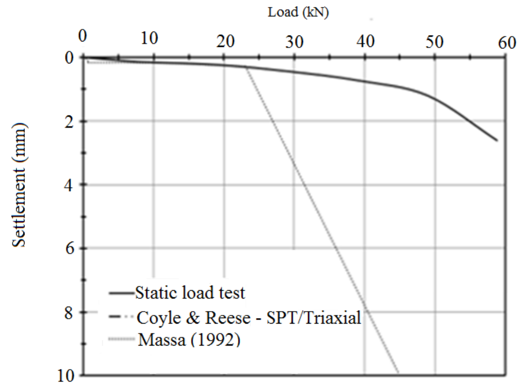

Figure 12 Results of Massad (1992)

The prediction of the load-settlement curve of the pile in analysis based on the use of computer programs was made using commercial programs RSPile and UniPile. In order to use RSPile, the soil was divided into two layers and from this, elastic modulus was estimated through Teixeira & Godoy (1996) correlation, so that the values of 29 MPa and 38 MPa were obtained for the lower and upper layers, respectively. For natural specific weight, it used 20 kN/m³ in both layers. For the load capacity factor (Nq), the value of 41.4 was used, as suggested by Bowles (1968) for a soil with a 35° angle of friction. In the predictions made with UniPile, it was used the stress distribution of Boussinesq and the resistance method proposed by Meyerhof, with the maximum shear stress along the shaft and the maximum peak stress as a function of the N-value. The load transfer curves, suggested by API and ISO (2007), were introduced in the custom function type. It is worth mentioning that, in the prediction, the effect of the residual load was not taken into account.

Figure 13(a) shows the predictions made from the experimental results obtained with the load test and, in Figure 13(b), a comparison of the predictions for the workload and the value obtained experimentally. From these results, it was observed that the settlements obtained were convergent, and for RSPile the values were slightly smaller and for UniPile were higher.

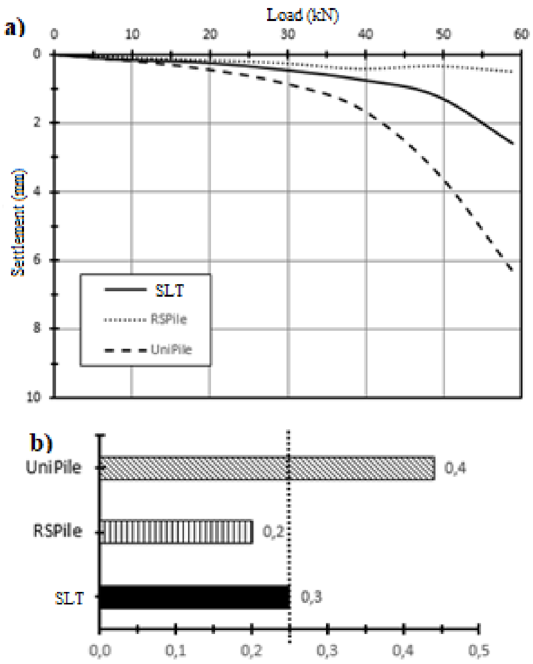

Figure 13 Prediction of settlements from RSPile and UniPile

Finally, Figure 14 shows the displacement profiles and the load-depth diagrams for the workload using the methods proposed by Coyle & Reese (1966) and the RSPile . From the results shown, it is observed that the elastic shortenings obtained by the two methodologies were convergent and assumed negligible values, a result that was in agreement with the values indicated experimentally. Moreover, it is observed that the lateral friction load loss assumed different values, varying from 7.97 kN for RSPile to 16.05 kN for the method proposed by Coyle & Reese (1966).

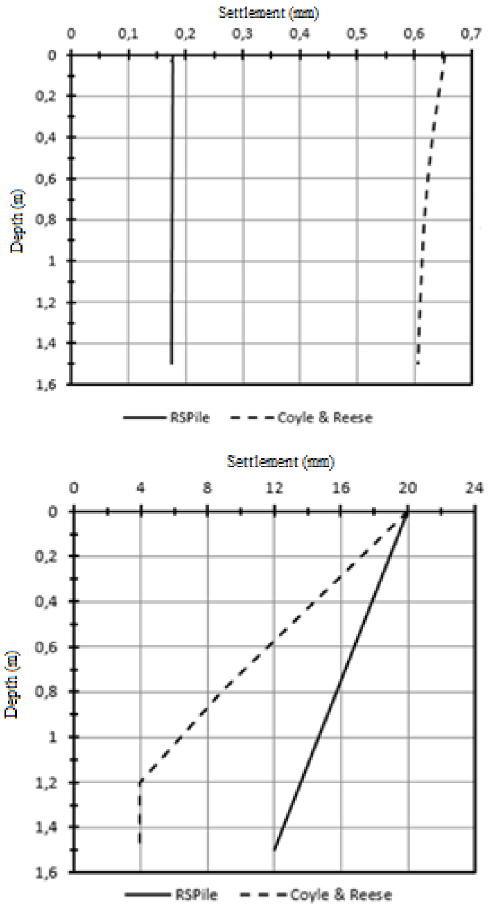

Figure 14 Estimation of load transfer for workload: displacement profile and load-depth diagram

6. CONCLUSIONS

From the results of the special resistance tests, it was observed a behavior typical from loose granular soil, with stress deviator versus volume variation, in the triaxial compression test, and shear stress versus horizontal displacement, in the direct shear test, with no peak strength and volume reduction of the specimens of both tests during the failure. Furthermore, comparing the failure envelopes obtained by these tests, we can observe straight-line shapes with a very close angle of friction and cohesion, these values are included in the range indicated in the literature for silty sands.

From the tests of static loads carried out on similar piles until the failure, with and without end's resistance, it was observed that they are only working by lateral friction.

Comparing the predictions of analytical settlements made by the method of Coyle & Reese (1966) and Massad (1992), there is a reasonable convergence of values in relation to the experimental values for the load range up to the workload. Specifically in the workload, the Coyle & Reese (1966) method adopting soil parameters from the triaxial tests provided the most convergent predictions. For the Massad method (1992), a significant approximation was also observed between the predictions and the experimental values for the load range up to 23 kN.

Regarding the predictions made from the UniPile and RSPile programs, convergence was observed between the two programs in the load range up to the work load (20 kN). Using the RSPile, the predictions were slightly lower and by UniPile the estimated values were higher than the experimental values.

It was also observed that the distribution of load along the shaft and the elastic shortening of the pile provided results consistent with the values measured experimentally.

Therefore, it was verified that, in general, for the bored piles installed in granular soil located in the Brazil's northeast, the predictions made in the elastic domain range based on the load transfer method were convergent and that the viability of the use of the referred method in the program used was proven.

7. ACKNOWLEDGMENTS

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