Some contributions to the analysis of piled-rafts made up by self-drilling piles founded in a tropical soil of Brazil

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ABSTRACT

This work presents the mechanical behavior of a piled-groups and piled-rafts constructed with self-drilling piles and founded in the “porous” clayey soil of the Federal District in Brazil. At a first moment, characterization test, field tests (Standard Penetration Test and Flat Dilatometer Test) and triaxial tests were applied to the soil deposit to obtain its mechanical behavior. The hypoplastic constitutive model that considers soil structure was tested to assess if the model can describe the clay behavior. Moreover, the mechanical behavior of the piled group and of the piled-raft foundation was validated by means of static load tests performed in the experimental field of a private foundation company in the region. By using the collected data, finite element simulations (FEM) were performed using the Abaqus software. The chosen model was implemented to describe the behavior of the soil in the Federal District. Simulated results allowed the establishment of the ultimate load of the foundation systems; the stresses generated and settlement mechanisms; and the contribution of the raft in the piled-raft systems. The results and their direct comparison allowed the generation of knowledge of academic and practical interest to those involved with the practical design, the construction or the investigation of such foundation systems.

KEYWORDS: piled-raft, self drilling pile, finite element simulations, tropical soil.

INTRODUCTION

Brasilia is located in a highly weathered residual tropical soil (lateritic & latosol on pedogenetic concepts). This soil presents high content of alumina and iron due to the leaching of the upper layers. As a consequence, the cemented structure is highly unstable and can produce serious changes in terms of volume (i.e. collapse) due to changes in both moisture and state of stresses. This material is known as "porous clay" by local geotechnical studies. Therefore, a greater understanding of the construction of pile group and piled-raft foundation systems on this particular soil conditions is needed – especially because the porous clay represents more than 80% of the total surface of the Federal District of Brazil, where the city of Brasília is located.

In addition, it is important to understand the behavior of deep foundations in the soil of Brasilia, in particular for the self-drilling “Alluvial Anker” (as locally named) type. This type of solution has recently started to be widely used in the city of Brasilia, given time and financial reasons (Barbosa, 2009). Nevertheless, there are few studies regarding this subject.

To approach the problem, several load tests were performed in this type of pile. The load tests performed included the variation of the number of piles, as well as tests with and without the contribution of the group top raft (block). Subsequently to the load testing and the characterization of the soil, finite element simulations were performed, and a hypoplastic model was applied as constitutive model in the porous clay layer. From these simulations, it was possible to obtain ultimate loads, working loads and the percentage contribution of each component of the tested foundation systems. The achieved results allowed to carry out several numerical analyses similar to those performed by Whitaker (1957), Cooke (1986), Mandolini et al. (2005) and Cartaxo (2011). Considering as reference the studies above, some modification for the Brasilia's porous clay soil was also performed.

As a contribution of this work, an advanced constitutive model – with hypoplastic structure – was used to obtain correlations of a practical application (design) and to obtain a more comprehensive understanding of the mechanical behavior of pile groups and piled-rafts in the local clayey deposits. At present, models of this type are not applied to the design of foundations, as initially they were not created for geotechnical applications (Helwany, 2007), and therefore could not reproduce the characteristics of soils found in the different regions of the world. This is therefore a pioneer work in regard, in relation to use such model to accurately describe the behavior of tropical soils when loaded by piled systems.

THE MECHANICAL BEHAVIOR OF POROUS CLAY IN THE FEDERAL DISTRICT – BRAZIL

For this research were made two load tests in the Solotrat´s experimental site (Figure 1a). This field is located in the outskirts of Brasilia, at Guará town. The Address of the site according to the current Brasilia’s nomenclature is quadrant SMAS, grouping A1, allotment 06, Guará-DF. The approximate coordinates are 15°48'59" S and 47°57'58" W and the mean elevation is 1084 m. Figure 1b shows the location of the experimental field. One characteristic of this site is the presence of the Brasilia´s porous, collapsible and soft clays.

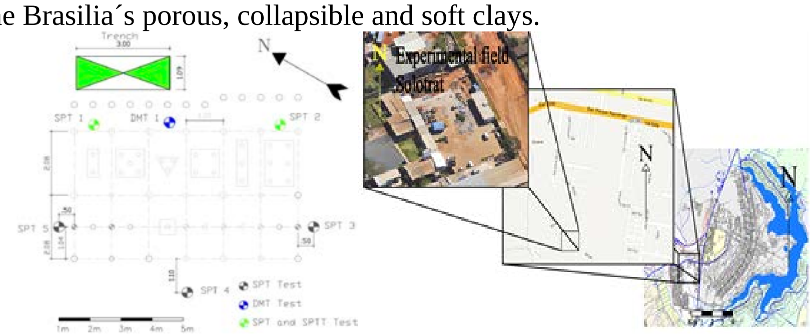
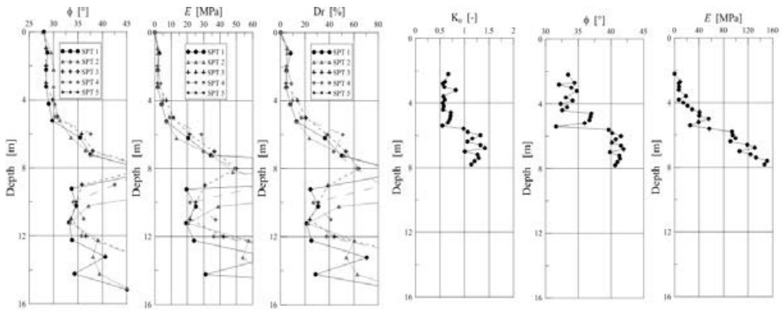


Figure 1: a. Location of load tests and field tests; b. Location of experimental site

Field tests and Laboratory tests

In order to characterize the soils of the experimental site, six field tests were carried out using a spacing of 2.40 m and depths varying from 8m to 15 m. Five boreholes were made through common dynamic penetration tests (SPT tests), very used in Brazil. Simultaneously, SPT and SPT-T tests were done. One flat dilatometer test (DMT) sounding was further performed.

By means SPT tests (Figure 2a) and one dilatometer test (Figure 2b) were done, which allowed the knowledge of both the approximate geotechnical parameters and the state variables (Figure 2)



(a) SPT test *(b) DMT test*

Figure 2: Parameters and state variables in field test

By means of SPT, SPT-T and DMT tests, as well as visual assessment of soil samples, five layers are recognized. The first layer ranges from 0 to 5.0 m (Layer 1), presents a red color classified as sandy clay, has soft consistency and the presence of plasticity. The water table is located at 5.0 m depth. The second layer ranges from 5.0 m to 8.0 m (Layer 2), showing brown color described as sandy silt, with slightly plastic and medium consistency. The third layer ranges from 8.0 to 9.0 m (Layer 3), has a white color described as sandy silt, with hard consistency, not plastic. Above 9.0 m depth, a dark brown silt clay soil was found, presenting medium consistency, slightly plastic (Layer 5). From 14.5 m to the end of the borehole, a yellow color sandy silt was found, with hard consistency and slightly plastic (Rigid layer).

From the field tests and the research developed by Araki (1997), Perez (1997) and Mota (2003) (another local soil with similar characteristics), approximate parameters of the site's profile were obtained for a Mohr-Coulomb model. The results of the parameter are shown in Table 1. These results will later be used in finite element simulations (FEM).

Table 1: Parameters obtained from the experimental field (Mohr-Coulomb elastoplastic model)

Parameter	Symbol	Unit	Value
First layer			
Friction Angle	ϕ	º	29
Elasticity Modulus	E	MPa	9
Cohesion	c	kPa	14*
Poisson's Ratio	μ	-	0.35
Second layer			
Friction Angle	ϕ	º	35
Elasticity Modulus	E	MPa	38
Cohesion	c	kPa	20**
Poisson's Ratio	μ	-	0.29
Third layer			
Friction Angle	ϕ	º	39
Elasticity Modulus	E	MPa	60
Cohesion	c	kPa	50***
Poisson's Ratio	μ	-	0.27
Fourth layer			
Friction Angle	ϕ	º	35
Elasticity Modulus	E	MPa	43
Cohesion	c	kPa	28***
Poisson's Ratio	μ	-	0.29

Observations: * From the work of Araki (1997); ** From the work of Perez (1997); *** From the work of Mota (2003).

In order to characterize soil behavior, eight triaxial tests were performed. The following conditions were applied: velocity changes and relaxation. This procedure is done to observe what features the constitutive model should have. With these tests, the constitutive model calibration was carried out, and the performance of the model was evaluated. These tests were developed on one sample block of 3 m depth ("porous clay" layer).

Two anisotropic consolidations in triaxial chamber were carried out. The first consolidation was performed with a stress ratio of η=0.3, and relaxation (stress change without volume change), as shown in Figure 3. Figure 3 shows the phenomenon of stress relaxation, starting at a mean effective stress of p=335 kPa and ending with a mean effective stress of p=205 kPa (this is the point at which stress loss no longer occurs with time). After relaxation, consolidation continues up to a mean effective stress of p=535 kPa. From this point on, an unload takes place up to p=5 kPa and then goes back to a load of a stress of p=580 kPa (Figure 3). In addition, this figure shows a η=0.3 test in a reconstituted sample.

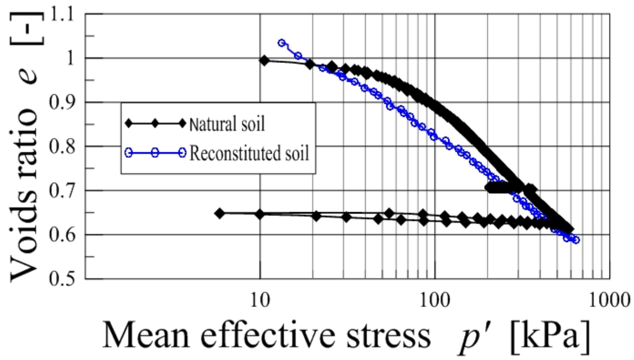


Figure 3: Triaxial compression in the plane of mean effective stress and void ratio with η =0.3 in Brasilia clay.

Six triaxial tests were made in drained and undrained conditions (Figure 4). These tests were performed to obtain soil parameters under shear.

The first three tests were developed in undrained conditions and at 110, 200 and 300 kPa confining pressures. All tests presented a constant vertical deformation rate of 0.05 mm/min. The tests allowed observing that the higher the confining pressure, the higher the peak deviator stress at a same unitary strain (Figure 4c). This is a typical behavior in soil mechanics (Whitlow, 2000; Helwany, 2007). Another factor is that the tests read the critical state line and continues through it. This leads to a resistance gain with strain (Figure 4a). This is an atypical behavior of the clays, as illustrated by the work of Roscoe et al. (1963).

Three other tests were performed at an effective stress of 110, 200 and 300 kPa, but under drained conditions (test CID). They applied velocity changes of vertical strain in two tests (110 and 200 kPa), as shown in Figure 4b. Such tests had the objective to verify the effect of velocity in the soil. It was demonstrated that the three tests reached the critical state line (Figure 4a). Figure 4b has a low influence of shear velocity on the soil. Therefore, it has a low viscous effect in the soil.

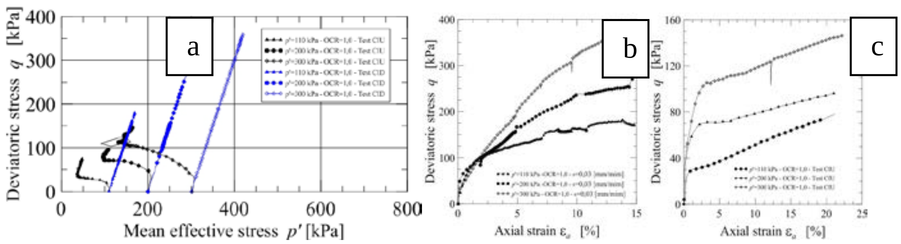


Figure 4: a**.** Stress path in drained and undrained conditions; b. Strain and stress path in drained conditions; c. Strain and stress path in undrained conditions.

LOAD TESTS ON “ALLUVIAL ANKER” PILE TYPES

This section presents the procedures used to obtain the mechanical behavior of the "Alluvial Anker" pile in the soil of Brasilia. An experimental field was built (Figure 5), located in the surroundings of Brasilia, more precisely in the city of Guará. The main characteristic of this site is the presence of the Brasilia's porous, collapsible and soft clays underneath it, where load tests on pile group and piled-rafts were carried out. These tests resulted in a settlement-load curve.

This is obtained for each pile group and piled-raft. The next step of this procedure presents the topics considered for experimental field construction and the results obtained.

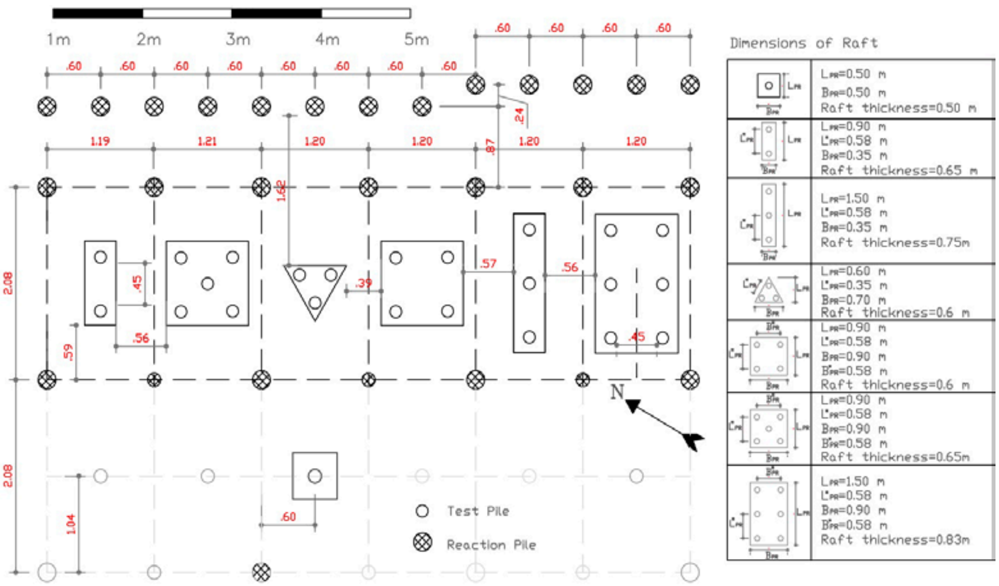


Figure 5: Location of the pile group and raft dimensions in the experimental field (build 50 Alluvial Anker type piles, 23 of these piles with 13 cm of diameter and 8 m in length (groups piles) and the rest respectively with 17 cm of diameter and 12 m (reactions piles)

“Alluvial Anker” self-drilling pile

Generally, the Alluvial Anker pile (common name for this pile type in the city of Brasília) has a nominal diameter of 13 cm, and is drilled by rotation with a steel pipe that has a cutting tip. This pipe is placed in a drilled machine specially designed for this work. A coolant to the cutting edge is continuously injected. Fluids used can be water-cement or water, depending on the type of soil. After finishing the process of drilling, an injection of a significant amount of water–cement is conducted (if the coolant is water). Figure 6 shows its construction process.

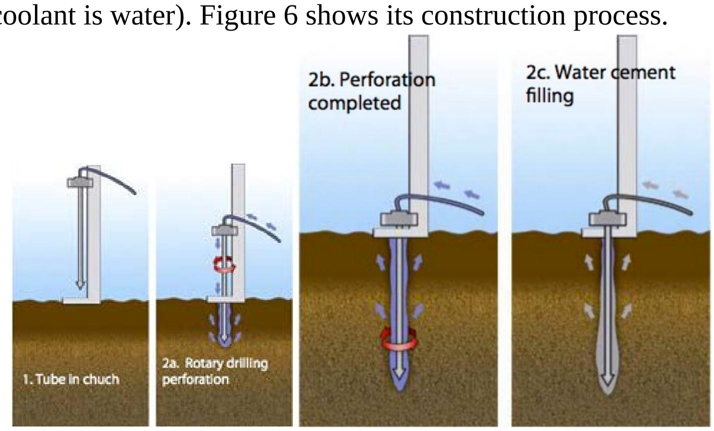


Figure 6: Alluvial Anker pile construction process (Barbosa, 2009)

Load Test Procedure

The load test was carried out pursuant to the NBR Brazilian standard for slow static load tests by following the next steps: a. equal increments by no more than 20% of the pile workload (ultimate load divided by a factor of safety of 2.0); b. load stabilization for at least 30 min, and reading of the displacement transducer of duration of 2, 4, 8, 15, 30 min and thereafter in increments of time of 30 min until they stabilize the readings, according to the Brazilian standard criterion of deformation (the difference between consecutive readings of settlement at the same step to a maximum of 5%); c. application of a total load to the pile greater than double of the workload.

To perform load tests, the loads were applied by a hydraulic jack of 2000 kN. Load measurement was performed by using a load cell of 1000 kN with 1 kN of sensitivity. To measure settlements, four mechanical deflectometers were installed into the base plate of the jack (Figure 7a). Four mechanical deflectometers were installed in the reaction piles. This was done to help monitoring movement and eccentricity in the test. Figure 7b shows the complete schematic feature of the vertical load test.

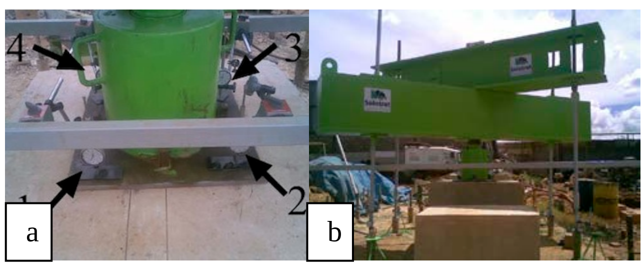


Figure 7: a. Mechanical deflectometers (1, 2, 3 and 4) installed in the base plate on hydraulic pump for the load test; b. Static piled-raft load

Considering the items above, twelve load tests were performed in two different configurations: piled-raft and pile group.

For the piled-raft, eight load steps of 200 kN were carried out for a total of 1800 kN (depending on the number of piles). For tests on the pile group, at a first moment, an excavation of 12 cm was performed beneath the raft and a process similar to the piled-raft system was adopted in static load test. It was plotted in terms of settlement and load curves with the criterion of the ultimate load given by the NBR 6122 Brazilian standards. This procedure was applied for each piled-raft (P-R) and pile group systems (P-G), as shows Figures 8a and 8b. For load tests that did not reach the ultimate load, an extrapolation was performed by using the method of Van der Veen (1953).

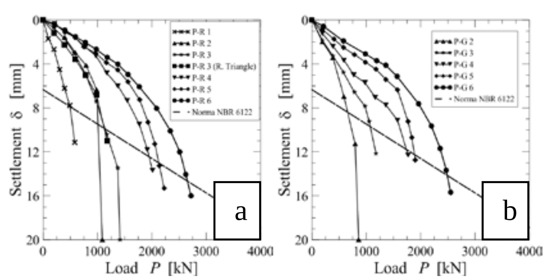


Figure 8: a. Results in static piled-raft load tests; b. Results in static pile group load tests

CONSTITUTIVE MODELS IN FEM

The selected constitutive models to simulate the stress-strain behavior of the tested soil were of the hypoplastic and of the elastoplastic types.

Hypoplastic model with structure

The model used to simulate the first soil layer was based on an approach of hypoplasticity with structure. Such model was chosen as it properly represents the typical characteristics of the superficial soft (porous and collapsible) clay of Brasilia.

At first, the hypoplastic model was developed for granular soils with the assumption of nonlinear incremental deformations. The first models were proposed by Von Wolffersdor (1996) and subsequently modified by Herle and Kolymbas; however, an extension to fine soils was needed, as proposed by Masín (2006), which was modified for a soil with structure.

The research done by Leroueil and Vaughan (1990), Cotecchia and Chandler (2000), among others, show the differences in the behavior of reconstituted and natural soils, explained as a loss of structure that natural soils do present. Several researchers have developed constitutive formulations taking into account the effect of structure in the soil. Among such proposals, one finds those of Gens and Nova (1993), Liu and Carter (2003), Masín (2006), among others. The proposal made by Masín was chosen given its facility in implementation and reasonably good accuracy.

The model is represented by Equation 1, in which $\dot{\mathrm{T}}$ is the objective stress rate, **D** is the Euler's stretching tensor, **L** and **N** are the fourth and second order (Masín, 2006) variables. The model is written as an increase of a nonlinear function of time to correlate stresses and strains. The modification made to the basic model in order to include the influence of structure consists of the incorporation of a structure degradation law by means of the proposal made by Baudet and Stallebras (2004). This proposal consists of the incorporation of a larger size limit state surface (SBS) by altering Hvorslev´s equivalent stress by a scalar (*s*) value, as illustrated in Figure 9.

The modification adds 3 new parameters (*s*, k, *A*). The first parameter is the initial value of the state variable of a structure factor or sensitivity (*s*) as shown in equation 2 (law of degradation). The other factors in the equation are the (s_f) factor, which is the limit to a stable state (Masin, 2006), with a value of 1 (Figure 9), and the initial value of (*s*) which evolves until (s_f). The factor (*k*), which is a parameter that controls the degradation of the structure; λ^* is the slope of the virgin isotropic compressibility line considered in a double natural logarithm chart; Besides, ε_d (Equation 3) is a "damage" strain, which depends on the volumetric and shear strain rates. It can be multiplied to an (*A*) factor, which controls the rate of shear strain variation, and varies with values $0<A<0.5$ (Masin, 2006).

The other five parameters of the model can be obtained from a natural or a reconstituted sample. That means, with triaxial isotropic consolidation tests, 3 parameters can be reached (λ^*, κ^*, *N*), as it similarly happens with the Cam-Clay model. Nevertheless, these parameters are different from those from this latter model as they are obtained in a double logarithm plane. The (*r*) parameter is achieved from undrained triaxial tests as the ratio between the undrained bulk modulus and the shear modulus. The angle (φ_c) is analogous to (*M*), which is the slope of the critical state line of the Cam-Clay model.

$$\dot{\mathrm{T}} = \mathrm{L}:\mathrm{D} + \mathrm{N}\,|\,\mathrm{D}\,| \tag{1}$$

$$\dot{s} = -\frac{\kappa}{\lambda^*}(s - s_f)\dot{\varepsilon}^d \tag{2}$$

$$\dot{\varepsilon}^d = \sqrt{\dot{\varepsilon}_v^2 + \frac{A}{1-A}\dot{\varepsilon}_s^2} \tag{3}$$

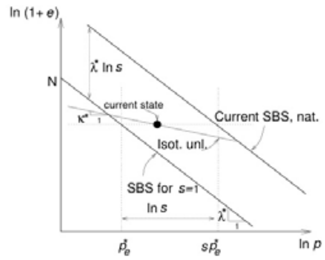


Figure 9: Idealization of isotropic compression behavior of natural and reconstituted soils (Masín, 2006)

The model was implemented and validated with the "Incremental Driver" program from Niemunis (2008). Then, simulations of the laboratory tests were performed to check the present premises. In Figure 10a, prediction of triaxial compressions is shown, whereas Figures 10b and 10c shows simulations of the triaxial tests on the natural clay of Brasilia. The soil parameters used in those simulations are therefore presented in Table 2.

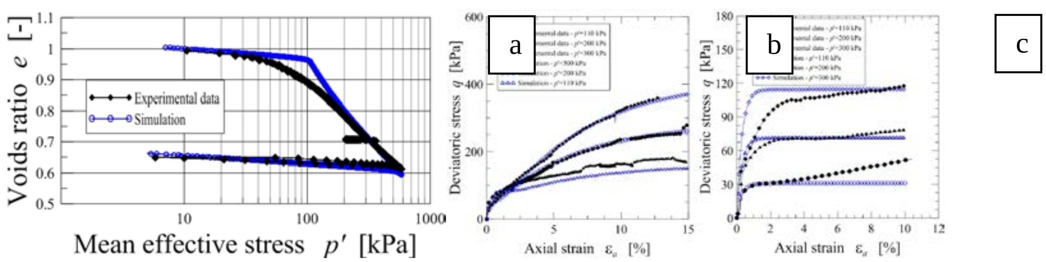


Figure 10: a. Anisotropic triaxial compression tests with η=0.3 b. Simulated and real stress paths in drained condition; c. Simulated and real stress paths in undrained condition

Table 2: Model parameters with simulations

κ	λ	ϕ_c	N	A	r	s	k
0.0022	0.060	31	2.13	0.4	0.35	1.5	2.5

Elastoplastic model

The second model was an elastoplastic model that responds well to the Mohr-Coulomb failure criteria. This model was implemented for the finite element simulations of the subsequent (to the first) layers. This was done since it has already been shown before (Mota 2003, and Anjos, 2006, among others) that the simple Mohr Coulomb model can reasonably well represent the deeper layers of this same stratum. Besides, due to the fact that this model has only four parameters and that all of them have a physical explanation, it has a great popularity among geotechnical practicioners.

SIMULATIONS PERFORMED BY THE FINITE ELEMENT METHOD (FEM)

An axisymmetric analysis was made using a finite element model in the ABAQUS program. The model has four layers, as specified in the analysis of Figure 11a. The first layer is modeled as a hypoplastic structured clay (the model's parameter is in Table 2). The other layers were modeled according to the Mohr-Coulomb Model, as previously cited (parameters in Table 1).

The parameterization model was a function of the pile length and diameter. The geometry of the numerical model consisted of one and a half times the pile length along the depth and 30 times the real diameter of the pile (19 cm) on the lateral boundaries (Helwany 2007) (Figure 11b). The real pile diameter was based in field observations from an exhumation test.

The elements selected for the mesh were of the C3D8R (Continuous, 3 dimensions, 8 node, and reduced integration) type for the piles, and of the C3D8 type (Continuous, 3 dimensions and 8 node) for the dry soil. For the saturated soil, the element assumed was of the C3D8P type (Continuous, 3 dimensions, 8 nodes and pore pressure measurement).

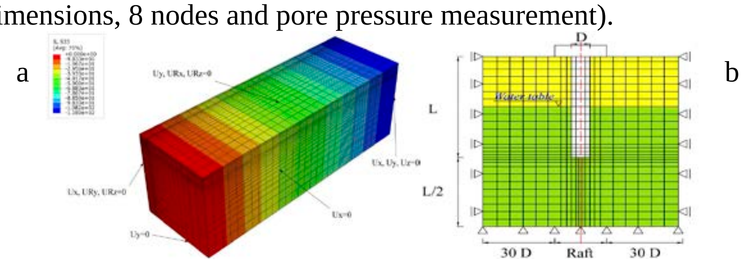


Figure 11: a. Example of piled-raft of the four piles, stress geostatic and initial conditions (Ui=Displacements in x, y and z and URi=Rotations in x, y and z); b. Parameterization and initial conditions

Load Test Simulations

With the purpose to understand the behavior of the PR and PG systems, FEM simulations were performed for both experimental distinc0t cases. Results for each of them are respectively shown through Figures 12a to 12b. Experimental data is also represented along these figures to indicate the accuracy of the modelization & adopted simplifications.

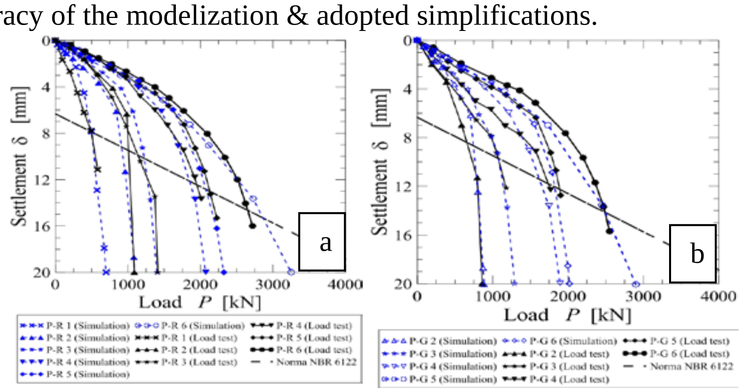


Figure 12: a. Comparison between simulations and load testing in piled-rafts; b. Comparison between simulations and load testing in pile group

Analysis and Discussion of Results

The most widely used methodology for the calculation of a piled-group is the sum of the bearing loading of the individual piles, including group efficiency (Equation 4). The efficiency factor obtained for each of the tested groups, obtained via finite element simulations and experimental load tests, are presented in Tables 3 and 4.

It is observed from these results that the efficiency values are equal to or less than one. The average efficiency of the simulations was 0.97, and of the load tests was 0.94. The group with the lowest efficiency was the one with three piles (value of 0.87). The group that showed the highest efficiency was the one with four piles (value of 1.07). These results are important because they indicate that equation 4 can be used in practice without much error when considering a unit group efficiency factor (usually done in design). The efficiency factor tends indeed to be lower than one for pile group in clays, and superior than that for groups in sands (Whitaker, 1957).

$$P_{PG} = \eta^* \sum_{i=1}^{n_p} P_p \tag{4}$$

in which: P_{PG}= Ultimate load in piles group; η^*= Efficiency factor; n_p= Numbers of piles; P_P= Ultimate load in individual pile.

Table 3: Load per pile in the group and efficiency factors (Simulations)

Piles group	Load type	Pile Load [kN]	Efficiency [-]
One Pile	P_P	419.18	-
Tow□ Piles	P_{PG}	850	1.01
Three Piles	P_{PG}	1100	0.88
Four Piles	P_{PG}	1800	1.07
Five Piles	P_{PG}	1900	0.90
Six Piles	P_{PG}	2520	1.0

Table 4: Load per pile in the group and efficiency factors (Load test)

Piles group	Load type	Pile Load [kN]	Efficiency [-]
One Pile	P_P	–	-
Two Piles	P_{PG}	750	0.89
Three Piles	P_{PG}	1780	0.87
Four Piles	P_{PG}	1780	1.06
Five Piles	P_{PG}	1970	0.91
Six Piles	P_{PG}	2520	1.0

Another analysis carried out on the P_G systems was the calculation of the settlement rate of the group (R_S), defined as the ratio between the settlement of the group (δ_G) and the settlement of an isolated pile of similar characteristics (δ_s). This rate is calculated at working load, i.e., failure load divided by 1.5, as indicated in Equation 5 (both in the elastic distance). The calculation of the factor of the group (R_G) was performed, as (R_S) divided by the number of piles (n_p) (see Equation 6). The apparent settling rate (R) was also calculated, being a function of the number of piles, the pile length (L) and the spacing between them (s_p) (Equation 7).

All of aforementioned variables are presented in Figure 13, which is an adaptation of the work presented by Mandolini et al. (2005). It shows (N) 63 cases of measured settlement for piled foundations. This model also uses an expression for R_S ("approximate equation”) as proposed by Clancy (1993) and shown in Equation 8.

$$R_s = \frac{\delta_G}{\delta_E} \tag{5}$$

$$R_G = \frac{R_S}{n_p} \quad \text{6)}$$

$$R = \sqrt{\frac{n_p s_p}{L}} \tag{7}$$

$$\mathbf{R_s = 0.29 n_p R^{-1,35}} \tag{8}$$

From the results of the simulations, a modification was made by Mandolini et al. (2005) via Equation 9 (proposal made). This equation is shown in Figure 13, which compares the original equation to the points of the simulations presented here. Notice that it is possible to calculate the settlement of the individual piles from this result (Mandolini et al., 2005) and to modify the R_S equation for the special characteristics of the soil of Brasilia.

$$R_s = 0.13 n_p R^{-1,25} \tag{9}$$

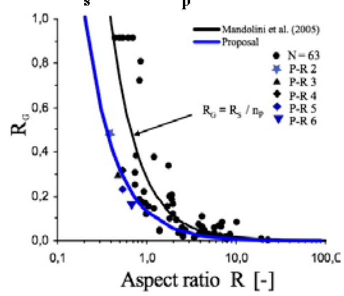


Figure 13: Relationship between (R_G) and (R) (modified from Mandolini et al., 2005)

With the simulations carried out in the P_R systems, percentages of load by each component, either pile and raft, were obtained in terms of ultimate load and workload (safety factor of 1.5), as shown in Figure 14a. From this figure, it is observed that a system with a greater number of piles decrease the percentage contribution per pile, as off course expected. It was also observed that the (summing of the) piles and the raft shared more less the same amount of load between them at working conditions, for each of the cases. This behavior has also been noticed for the experimental results, that are represented in Figure 14b. These latter results have been predicted by assuming the value of load absorbed by the raft alone, which was calculated as the piled-raft load minus the load in the pile group. It is also noticeable that, in experimental terms, the ultimate load of the raft alone and the summing of the piles has been considerably distinct in each of the tested cases (see for instance cases of PR with 2 and 3 piles). A more homogeneous (similar) condition, has nevertheless been obtained for the numerical analyses (which do not take on account the existing variability of stratigraphy in the site).

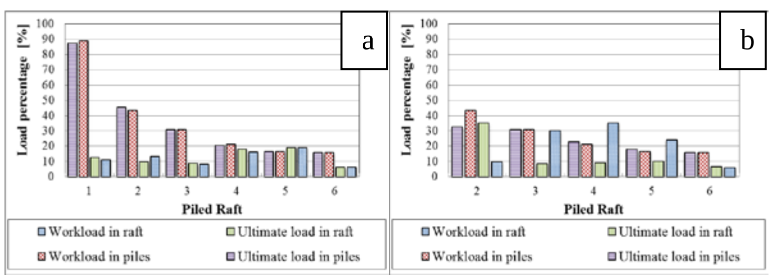


Figure 14: a. Simulated percentage of average load at piles and raft for the PR systems; b. Experimental percentage of average load at piles and raft for the PR systems

A comparative analysis was carried out between the tests performed in this paper and those conducted in centrifuge as reported by Conte et al. (2003) (for five types of piled-raft generating large settlements). This comparison takes on consideration typical non dimensional parameters used for piled rafts. That means, according to Cooke (1986), the contribution of rafts can be determined by the ratio (ζ_{PR}) (Equation 10), which is the ratio between the load capacity of a piled-raft (P_{PR}) system and the load capacity of a standard pile group (P_{PG}). The settlement and the group characteristics are somehow expressed by the R_M variable, defined in Figure 15.

$$\zeta_{PR} = \frac{P_{PR}}{P_{PG}} \tag{10}$$

The information related to this comparison is presented in Table 5, that depicts the step by step calculation of the experimental data plotted in Figure 15 for the present series of analyses.

Table 5: Data for the relationship (R_M) and (ζ_{PR})

piled-raft	A_G [m^2]	A_R [m^2]	R [-]	R_M [-]	P_{PR} [kN]	P_{GP} [kN]	ζ_{PR} [-]
two piles	0.11	0.31	0.38	1.08	920	750	1.22
three piles	0.22	0.52	0.46	1.11	1250	1100	1.13
four piles	0.33	0.81	0.53	1.29	1875	1780	1.05
five piles	0.33	0.81	0.50	1.22	2190	1970	1.11
six piles	0.67	1.35	0.66	1.32	2520	2520	1.13

OBS: Area of the pile group = A_G, raft area = A_R, relationship between apparent rate settling and areas = R_M.

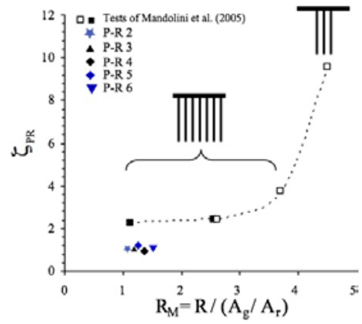


Figure 15: Relationship between (R_M) and (ζ_{PR}) (modified from Mandolini et al., 2005)

MAJOR POINTS AND CONCLUSIONS

This paper investigated the individual behavior of piled-rafts and standard groups of piles with differing characteristics concerning the number of piles through real load tests and numerical simulations on FEM under vertical load conditions. Based on the results obtained, it is possible to suggest that:

By means of an advanced constitutive model such as the hypoplastic model, it is possible to represent, in a satisfactorily manner, the behavior of the porous clay of Brasília. It is an optimal tool to obtain specific correlations for the design of foundations with self-drilling piles of the Alluvial Anker type, offering a good understanding of the behavior of this type of foundation;

Efficiencies ranging from 0.87 to 1.07 were obtained in the piled-groups. A practical value close to unit can be related to these type of foundations, thus allowing the direct use of the sum of the pile capacities to obtain the overall group's bearing load;

The piled-raft systems had bearing values that surpassed those from similar (number of piles and arrangement) piled group systems. The percentage increase varied from 32% (piled-raft of two piles) to 6% (piled-raft of six piles). An average increase of 15% can be established for the tests carried out herein. This difference in behavior is, off course, credited to the capacity gained by the contact of the raft in the piled-raft systems;

A practical relationship between Clancy and Randolph (1993) R_G (group factor) and R (aspect ratio) variables, valid for the soil of the Federal District of Brazil, has been established and can surely be used in the local practice when designing with similar foundation systems; Moreover, a another practical proposal that estimates the settlement of similar PR systems at working loads was devised.

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