NUMERICAL EVALUATION OF DEEP FOUNDATIONS IN TROPICAL SOILS OF THE FEDERAL DISTRICT OF BRAZIL BY MEANS OF A SEMI-ANALYTICAL MATHEMATICAL PROCEDURE

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Abstract. This paper focuses on the numerical simulation of deep foundations via existing industrial application software denominated GEO4. This software computes the load-displacement curve of the pile's head, plus the distribution of normal and shear forces along the pile's shaft. It was assessed herein on a pioneer research basis, since it was never tested against experimental results from deep foundations of Brazil – in particular founded on "non classical" soil deposits. It also presents experimental results from field loading tests carried out with distinct foundations founded in tropical stratified soils of the Federal District of Brazil. The curves of pile load versus displacement, and load transfer along the pile's shaft are presented, discussed, and numerically simulated by a semi-analytical procedure, together with the description of the geotechnical characteristics of three distinct chosen sites of this District. Values of skin friction and base pressure are additionally presented and compared to numerical predictions. It is concluded that this software has successfully aided the analysis, demonstrating its large potential for future application in Brasília as well as other regions. The analyses also demonstrated that differences in backanalyzed results are prone to occur, given distinct pile construction methodologies, user considerations and distinct geological & topographical characteristics among different sites.

1. INTRODUCTION

The pre-designed Brazilian capital Brasília, located in the Federal District of Brazil, was designed to house only the main Governmental administrative institutions and its public employees. Nevertheless, it has increased (and is still expanding) four times more than what was initially forecasted, advancing through distinct (geological) zones of this same district, and allowing the use of distinct techniques for deep foundation deployment and design.

Hence, in 1995 the University of Brasília started a major research project in the foundation area, in order to enhance the knowledge on the behavior of the typical deep foundations that are founded in the stratified, tropical subsoil of the Federal District. It was decided to carry out horizontal and vertical field loading tests on distinct locally used foundation types. These foundations had full-scale dimensions and were placed within the University of Brasília campus, at the experimental research site of the Geotechnical Post-Graduation Program. The geotechnical profile of this campus is composed by the typical unsaturated “porous” clay of Brasília. In addition to that, field loading tests on full-scale instrumented foundations outside campus were also carried out. In all cases, the tests counted with the cooperation of local engineering companies and University staff.

A large effort was also undertaken to evaluate design techniques adopted to devise the foundations within the Federal District. Current techniques, as well as more advanced ones (numerical approaches) have also been evaluated and still are under scrutiny by researchers from the Foundation Group of the University of Brasília. In order to accomplish this goal, several software programs were acquired and tested against field loading test results from foundations tested to failure (or close to). Some of these foundations were fully instrumented, as those presented herein, enabling the knowledge of the load transfer distribution along depth and throughout loading level till estimated geotechnical failure. Therefore, the main objective of this paper is to present the summary of some of the findings of aforementioned research project of the Post-Graduation Program of the University of Brasília, in particular related to a new software program devised to forecast the load-displacement and load transfer curves of vertically loaded piles founded on stratified soil materials.

In summary, the paper presents experimental results from three field loading tests carried out with distinct deep foundations. It discusses the results and presents the numerical backanalyses carried out to simulate the load-displacement and load-transfer mechanism of the studied piles. A mechanically bored pile founded in the University of Brasília campus (UnB research site) is analyzed together with instrumented piles from two distinct engineering sites within this same District. One of them is a continuous flight auger type pile located in the North Wing of Brasília (212N site), close to the University Campus, and founded in the same geological material. The other is a mechanically bored pile excavated with bentonite mud, located in a site around 25 km from the University Campus (Taguatinga site), and founded in a foliated and stratified material which originates from slate decomposition.

The experimental curves of pile load versus displacement, structural load transfer along pile’s shaft, and average skin friction and base pressure versus loading level, are presented and compared to numerical predictions from a semi-analytical procedure. This procedure is coded in the industrial software denominated GEO4, which is commercially available. This software simulates the pile behavior, once few (geotechnical and structural) input data is given. The backanalyzed parameters and experimental data are also cross-compared among the distinct studied sites, serving as preliminary guidance values for usage in actual design with piles of similar characteristics as those presented herein.

2. GENERAL SITE CHARACTERISTICS

The Federal District has a total area of 5814 km^2 and is limited in the north by the 15°30’ parallel and in the south by the 16°03’ parallel. It is presented in Figure 1 as a dot within the Brazilian Map. This figure also presents the location of the three studied sites of this paper, denominated as UnB, 212N and Taguatinga sites.

The first site, UnB, has already been extensively studied and presented in literature (Cunha et al. 1999, Cunha et al. 2001). Distinct deep foundations were constructed and vertically and horizontally loaded in this site, together with the deployment of advanced in situ tests (cone and dilatometer penetration tests, standard penetration tests with torque measurements and others) and soil suction measurements. Block samples were retrieved and taken to the laboratory for triaxial, oedometer, direct shear, and standard characterization tests. Details of the laboratory results are presented in Table 1, and some of this data is still under scrutiny by other researchers (as Mota 2002).

The second site, 212N, is related to a building site where a residential block of apartments (6 floors) are to be built. In this particular site 401 continuous flight auger piles with distinct diameters and lengths have already been constructed. In order to gather more design information, the engineering contractor has agreed to carry out a vertical field loading test in one of the piles, as well as to fully instrument it with strain gauges at distinct depth levels. Details of these experiments are given in Cunha et al. (2002) and will be briefly presented herein.

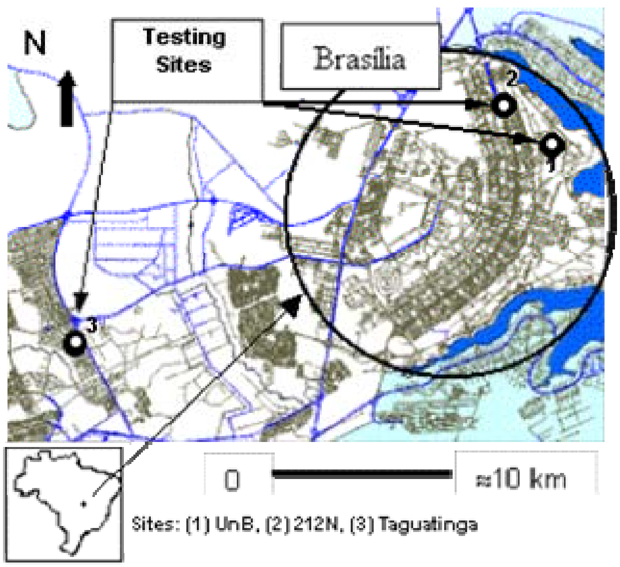

Figure 1. Location of the Studied Sites.

Table 1. General Geotechnical Parameters of the UnB Site (Cunha et al. 1999).

Parameter	Unit	Range
Sand percentage	%	12-27
Silt percentage	%	8-36
Clay percentage	%	80-37
Dry unit weight	kN/m^3	10-17
Natural unit weight	kN/m^3	17-19
Moisture content	%	20-34
Degree of saturation	%	50-86
Void ratio	--	1.0-2.0
Liquid limit	%	25-78
Plastic limit	%	20-34
Plasticity index	%	5-44
Drained cohesion[a]	kPa	10-34
Drained Friction angle[a]	degrees	26-34
Young's Modulus[b]	MPa	1-8
Coefficient of Collapse	%	0-12
Coeff. Earth Press (K0)[c]	--	0.44-0.54
Coeff.Permeability	cm/s	10^{-6}-10^{-3}

OBSERVATIONS:
a- Triaxial CK0D tests-Inundated Soil and at natural humidity;
b- Triaxial CK0D tests: Soil at natural humidity-50% failure deviator stress;
c- Triaxial K0 tests: Soil at natural humidity.

In geologic terms it is common the occurrence of extensive areas (more than 80 % of the total District area) covered by a weathered laterite of the tertiary-quaternary age. This "latosol" has been extensively subjected to a leaching process and it presents a variable thickness, varying from few centimeters to around 40 meters. It is common for both UnB and 212N sites, and is basically a red residual soil developed in humid, tropical and subtropical regions of good drainage. It is leached of silica and contains concentrations particularly of iron oxides and hydroxides and aluminum hydroxides (US Bureau of Mines 1996). It also has a predominance of the clay mineral caulinite and, in localized points of the Federal District, as in the Taguatinga site, it overlays a saprolitic/residual soil with a strong anisotropic mechanical behavior and high standard penetration resistance, which is originated from a weathered, folded and foliate slate, a typical parent rock of the region (Cunha and Camapum de Carvalho 1997).

The surficial latosol has a dark reddish coloration, and displays a much lower resistance and a much higher permeability than the bottom saprolitic/residual soil from slate, as we can observe in both Figures 2 and 3 by the N_{SPT} readings (respectively for UnB and 212N sites). Figure 2 also displays the Marchetti dilatometer P0 and P1 pressure readings.

These figures also serve to show the variability of the geotechnical profile from one site to the other (2 km approximately apart). As demonstred by these figures, this variability is reflected on the field $N_{(SPT)}$ results. It will be also reflected in the geotechnical parameters obtained from the numerical backanalysis – to be discussed later.

Hence, the studied latosol at both UnB and 212N sites constitute into ”collapsible” sandy clay with traces of silt, with a high void ratio and coefficient of collapse. Its coefficient of permeability is also high for typical clays, being close to those found for fine to silty sands. This soil is the so-called “porous” clay of Brasília, which major geotechnical parameters were displayed in Table 1. The variability of this table is given to the distinct rates of leaching and weathering that took place along the depth.

It is herein assumed that the soil of 212N site, geologically from the same origin, has similar range of geotechnical parameters as those presented in Table 1. Some variability, however, exists.

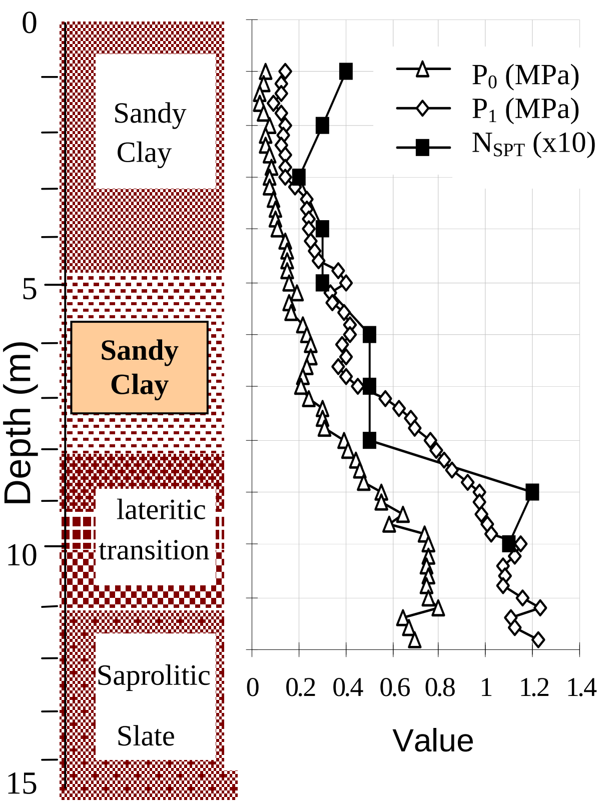

Figure 2. Geotechnical Profile of the UnB Site.

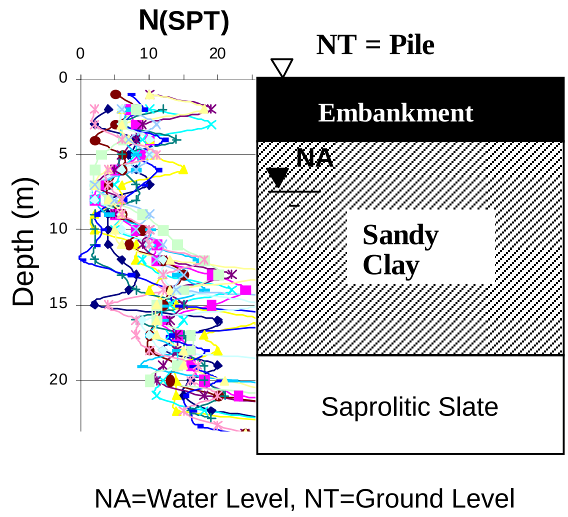

Figure 3. Geotechnical Profile of 212N Site.

The ground level at 212N, similar to the pile head level of this same site (denominated as NT in Figure 3), is not the same topographical level as the ground level of the UnB site (level 0 in Figure 2). In the 212N site, the soil has been excavated by 4 m for the construction of the subsoil floor level of the building. This building on the other hand, was placed on top of an old embankment fill located in this same area, with thickness of 4 m. Another difference is the fact that in the UnB site no water level was found.

The other studied engineering site, denominated Taguatinga, is located around 25 km from Brasília, according to Figure 1.

In Taguatinga site the local slate is formed via metamorphic and geologic processes, serving as hard strata, or bearing layer, of most of the bored / precast deep foundations of this region. The saprolitic and residual soils originated from this rock, abundant in the region, preserve some of the foliation and folding characteristics of the parent rock. This soil stratum is undoubtedly anisotropic in terms of deformational and mechanical characteristics. This behavior can be accentuated by non homogeneous construction techniques employed in the same working site.

Cunha and Camapum de Carvalho (1997) have already observed that the residual soil of slate has an extreme anisotropic behavior when sheared in distinct directions to the foliation plane in a conventional direct shear test apparatus. For instance, Tables 2 and 3 present the results obtained by these authors from samples taken from a nearby site with similar geological features as those of Taguatinga. This soil can be classified as CL in the Unified Classification System, with low to medium plasticity and strong anisotropic behavior.

Thus, local foundation design projects are generally done with the recommendation of further field loading tests to verify the design assumptions. This was the case herein, which has led to the participation of the University of Brasília in the interpretation of the field loading tests of this particular site. Figure 4 presents the geotechnical profile and $N_{(SPT)}$ results of the site.

In Taguatinga two bored foundations were instrumented and field loaded. However, only the results of one of them (PC2 in Figure 4), served as basis of evaluation of the bearing capacity and settlement of the site foundations, as well as to define the percentage of load distributed along foundation's shaft and base. Details of these experiments are given in Cunha et al. (2003) and will be briefly presented herein.

3. INSTRUMENTATION AND FIELD LOAD TESTS

All the field loading tests were done in accordance to the recommendations put forward by the Brazilian NBR 12131 (1996) standard, and they consisted of slow maintained tests.

The loading tests were performed in loading intervals of approximately 20 % of the working load, hence with an average value of 27, 120 and 320 kN, respectively for the piles at the UnB, 212N and Taguatinga sites.

This loading sequence was increased up to the geotechnical failure (or close) of these piles. The load was then kept constant for 12 hours to check for creep effects, and released thereafter in approximate five unloading intervals.

The load tests adopted a reaction frame and bored "reaction" piles some meters apart, in a distance of at least three pile diameters, as schematically displayed in Figure 5.

Table 2. Characterization Tests in Residual Slate Sample. (Cunha and C.Carvalho, 1997).

Property	Avg. Value (%)
Clay Fraction	30
Silt Fraction	67
Sand Fraction	3
Plastic Limit	20
Liquid Limit	44
Plasticity Index	24

Table 3. Direct Shear Tests in Residual Slate Sample. (Cunha and C.Carvalho, 1997).

Foliation Angle (x^o)	Cohesion (kPa)	Friction (degrees)
Horizontal	13.6	19.0
Inclined	29.8	10.0
Vertical	0.4	27.0

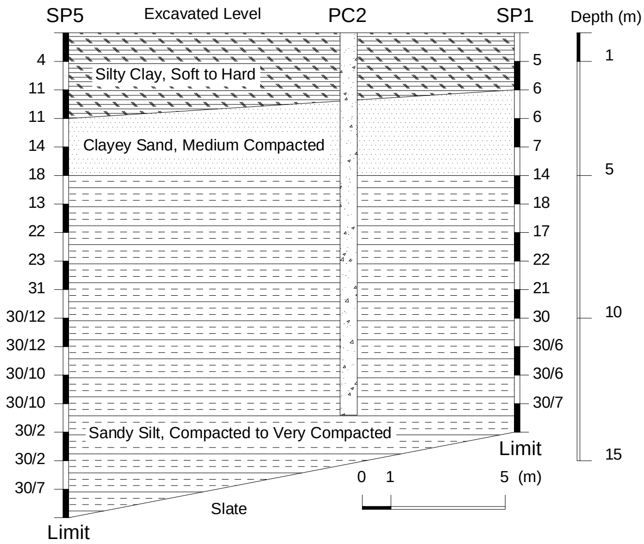

Figure 4. Geotechnical Profile of Taguatinga Site. (Cunha et al. 2003).

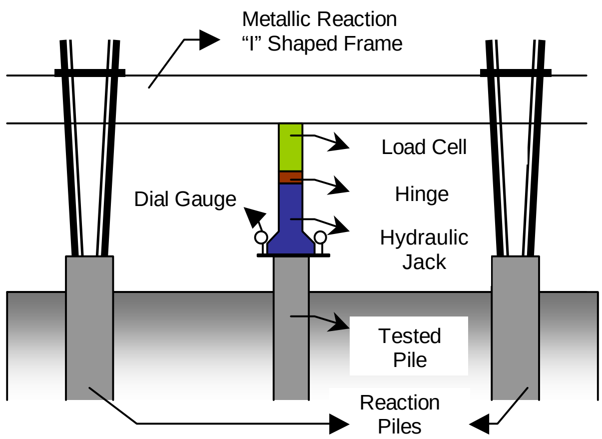

Figure 5. Schematic Drawing of the Field Load Testing System.

Both the top foundation block and the reaction frame were monitored for tilting and vertical displacements, by using six 0.01 mm precision dial gauges. A 1000, 2000 and 5000 kN hydraulic jack were respectively used in each site, together with a 100 N precision load cell to load the piles till failure condition.

The loading tests were carried out with the soil only at its natural moisture content conditions. The following characteristics can be described for each site:

- UnB site: The field load test was carried out in July 2000 (dry season). It was tested a mechanically bored, cast-in-place pile, with 0.3 m in diameter and 8 m in length, herein defined as UnB pile. This pile was excavated by using a continuous hollow flight auger, which was introduced into the soil by rotation. The hydraulic mechanical auger was assembled in the back part of a truck specially devised for this type of work. The soil was successively removed during continuous auger introduction and withdrawn, and, after the final depth was reached, the auger was withdrawn leaving a freshly excavated hole. This hole was subsequently filled with concrete poured by using the transportable service of a local concrete company. The pile was loaded till its geotechnical failure, estimated in 270 kN by the Van der Veen (1953) method, being noted a displacement of 9.4 mm at ultimate load level. The instrumentation followed the procedures put forward by Cintra and Toshiaki (1988), where it is found the step-by-step sequence to work with strain gauges in foundations. The strain gauges of the type KFG-1-120-C11-11, with tolerance resistance of 120Ω, were adopted. They were installed in a full "Wheatstone Bridge" configuration, in order to reduce temperature effects during the load test. They were connected to a 16 mm diameter smooth surface bar, which was positioned centrally to the foundation's transversal cross section. The strain gauges were placed at distinct positions along the pile, allowing the knowledge of the load transfer mechanism during the load test, at different (head) load levels. Figure 6 presents the full load-displacement curve of this test. It was noticed that, at failure stage, less than 5% of the applied load at the pile's head was transmitted to the base of this pile;

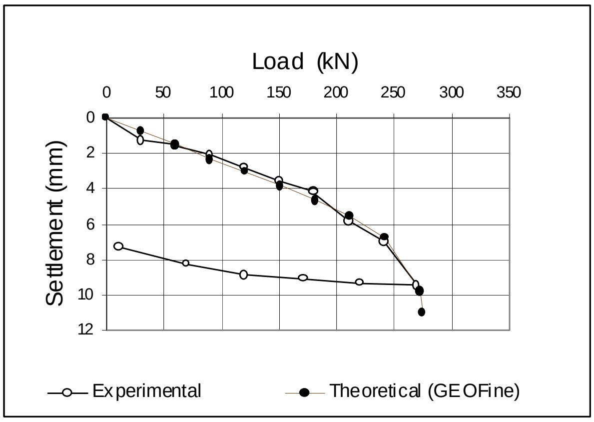

Figure 6. Load-Displacement Curve Obtained for the UnB Pile (modified after Mota 2002).

- 212N site: The field load test was carried out from July to August 2001 (dry season). It was tested a continuous flight auger (CFA), cast-in-place pile, with 0.4 m in diameter and 18.5 m in length, herein defined as 212N pile. This pile was excavated by using a continuous flight auger equipment (so far the only equipment in the Federal District for this category of pile), which was introduced into the soil by rotation. The hydraulic and computer controlled auger was assembled in the front part of a truck specially devised for this type of work. No soil was removed during auger introduction, and, after the final depth was reached, the auger was withdrawn with simultaneous soil removal and pump of concrete, leaving a finished CFA pile once the auger was totally withdrawn to surface. The pile was loaded till its geotechnical failure, estimated in 1200 kN by the Van der Veen (1953) method, but the reaction system failed when the load was at 1100 kN level. At this level it was noted a displacement of 23 mm, and the load-displacement curve was already in its plastic stage, indicating imminence of failure. The instrumentation also followed the procedures put forward by Cintra and Toshiaki (1988), with the use of strain gauges of the type KFG-1-120-C11-11 with tolerance resistance of 120Ω. They were installed in a ¼ "Wheatstone Bridge" configuration, since the test was carried out at night – hence, with low influence from temperature effects. They were also connected to a 16 mm diameter smooth surface bar, which was positioned centrally to the foundation's transversal cross section. The strain gauges were placed at distinct positions along the pile, allowing the knowledge of the load transfer mechanism during the load test, at different (head) load levels. Figure 7 presents the full load-displacement curve of this test. It was noticed that, at the ultimate (1100 kN) stage, 25% of the applied load at the pile's head was transmitted to the base of this pile;

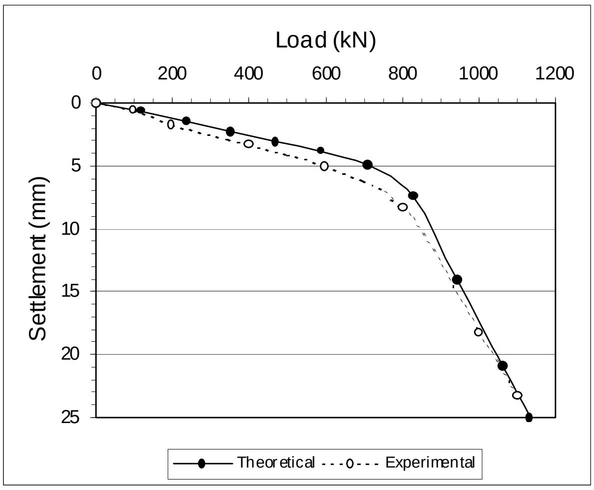

Figure 7. Load-Displacement Curve Obtained for the 212N Pile (after Cunha et al. 2002).

- Taguatinga site: In this site two fully instrumented large-scale foundations were field loaded in August and September 2001 (dry season). They were constructed with similar equipment as adopted for the UnB site and had a diameter of 60 cm. They were bored with bentonite mud (poured after the hole was excavated), and cast *in situ* with plastic concrete. They had distinct lengths and were located in different positions of the same construction site, in order to simulate the foundation's behavior at contrasting geological and geotechnical sections of this same area. However, only one of them, herein denominated as Taguatinga pile (PC2 of Figure 4), have guided the foundation's contractor in the improvement of the field construction methodology. This foundation had a length of 13.4 m, was located centrally to a heavy loaded pile group of the site, and was instrumented in a similar way as for the UnB pile. It was loaded till a value close to its geotechnical failure, estimated in 3200 kN by the Van der Veen (1953) method, being noticed a small displacement of less than 7.0 mm at the ultimate load level. Figure 8 presents the full load-displacement curve of this test. It was noticed that, throughout the testing stage, an average of 40% of the applied load at the pile's head was transmitted to the base of this pile.

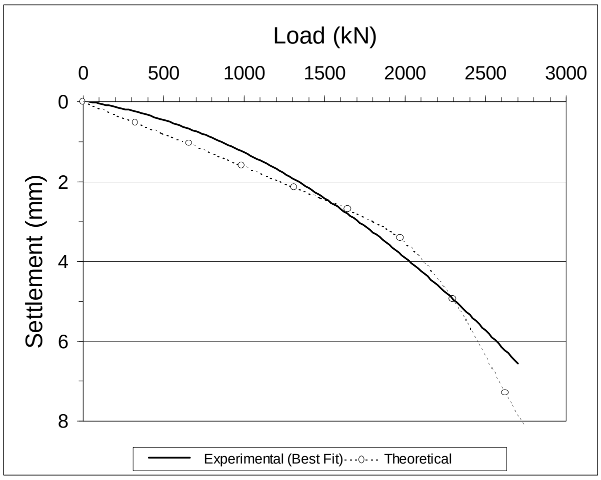

Figure 8. Load-Displacement Curve Obtained for the Taguatinga Pile.

4. NUMERICAL SOFTWARE

The numerical backanalysis of the behavior of the piles was carried out with the industrial software denominated GEO4 from the commercial company FINE Professional Civil Engineering Software Company Ltd., with headquarters in Prague/Czech Republic.

Although simple to use, this software has a high potential for usage in practical civil engineering applications – not only in the pile foundation area, but also in many others as retaining walls, shallow foundations, embankments, pavements, diaphragm walls, slope stability and so on (given distinct personalized modules for each of aforementioned technical areas).

In Brazil, in particular, it is one of the first times that this (foundation) module from the whole GEO4 software package is tested with an actual case, or civil engineering work. The whole GEO4 software package has been donated to the Geotechnical Post-Graduation Program of the University of Brasília with the objective of evaluation and testing, as well as usage in geotechnical research. The foundation module is capable of deriving the full load-displacement curve of a vertically loaded pile, as well as its load transfer mechanism (structural load along pile's depth, for each test load level). The horizontal behavior of the pile can also be simulated in this same module. Unfortunately this latter characteristic of the software was not tested herein, given difficulties involved with a lateral load test.

This module is based on a semi-analytical solution. This solution is related to the Young modulus and Poisson's ratio of the soil (Winker-Pasternak solution, as presented by Bittar and Sejnoha 1996), as well as the depth of the influence zone. After discretization of the pile on one-dimensional bar elements, the influence zone evolves around each of the nodes. The pile-soil interface is modeled in nodes using nonlinear soil springs. In case of a semi-infinite body surrounded by soil the response is given by the known Mindlin's solution. The shear behavior of pile-soil interface is described using the elastic-plastic material model with the Mohr-Coulomb yield condition. More details of each theoretical aspect of the mathematical model and numerical solution are given next.

4.1 Vertical Bearing Capacity

This module of the program also serves to determine the vertical bearing capacity of a pile – simply by derivation of the limit of the load-displacement curve. The main advantage of the program is the accessibility of soil and rock parameters, respectively. The program requires the knowledge of the angle of internal friction, cohesion, bulk weight and the Young modulus of the soil & rock. As a result, the module provides the load-displacement curve till a pre-specified limit deflection (or failure according to some standards). The pile's vertical bearing capacity is related to this limit deflection.

4.2 Theoretical Grounds

The pile is modeled using standard beam elements, while the behavior of surrounding soil, in terms of load distribution, is described by a known fundamental solution of a layered soil structure. In case of a semi-infinite body the solution is known as Mindlin's solution. The solution is improved by incorporating the structural strength of soil in a similar way as used for modeling settlement of spread footings. The influence of underground water is incorporated using the Archimedes law. The shear behavior of pile-soil interface is described using the elastic-plastic material model with the Mohr-Coulomb yield condition. The unknown cinematically admissible displacement follows from the equilibrium condition in the vertical direction. The material nonlinearity is reflected by using variable secant modules for the soil "springs", each of them along the pile's shaft and one at base.

4.3 Solution Procedure

The following steps are followed during a standard analysis of a vertically loaded pile immersed in a layered, elasto-plastic, soil material:

a) The pile structure is modeled as a member composed of several elements. The number of elements is then determined from the length to pile diameter ratio, for which the program derives the solution of the shear stiffness of the soil surrounding the pile. The element length should be about 2.5 times larger then the pile diameter. Nevertheless, the program automatically assumes at least ten elements to avoid spurious results. The shear stiffness of the soil is, however, still based on the l/d ratio of 2.5;

b) Each element is supported at its bottom end by a spring. The spring stiffness is derived by employing parameters of elastic subsoil C1, C2 and modified Bessel's functions (Winkler-Pasternak model). Values of C1 and C2 are determined from the Young modulus and Poisson´s coefficient of the soil. The depth of the influence zone, which affects the values of C1 and C2, is variable and changes with pile deformation (settlement). For zero deformation it is set equal to one time the pile diameter, whereas at the onset of geotechnical pile failure the influence zone is set to 2.5 times the pile diameter. It shall be mentioned that the reliability of the values C1 and C2 depends on a good assessment of the soil deformation parameters;

c) For each pile element the program determines the maximum value of shear force transmitted to the shaft via skin friction. This is done with the use of the traditional Mohr Coulomb failure criteria, plus the lateral stress at each pile element. This stress is found via geostatic stress distribution multiplied by a coefficient of lateral earth pressure K, which is user inputted according to the pile's construction methodology;

d) With the knowledge of both spring stiffness and limit force value (via maximum shear force) the program starts to incrementally load the pile, with the given force(s) applied at the pile's head. Forces developed in individual springs of all elements are computed at each increment. These forces are then compared with the maximum shear force (Tlim) estimated in the previous step, for each of the pile elements. If the spring force exceeds Tlim, then the stiffness of this spring is reduced in such a way that, for a given deformation, the spring force equals Tlim. The exceeding force for this particular load increment is then redistributed into the remaining springs. Each load increment is iterated until the force developed in every spring is less than Tlim. The gradual softening of the individual springs leads to a final nonlinear load-displacement curve for the loaded pile, if geotechnical failure starts to take place during this numerical simulation (i.e. if the pile, soil and loading conditions are such that allow soil plastification during the simulation). Evidently, at a certain (high) load level all springs will lose capability of increasing its force, and the pile will start to penetrate into the soil with support solely given by the base (heel) spring. It is also worthy to mention that, in mathematical terms, there is no restriction on the magnitude of the force to be developed in the base spring (as assumed by GEO4), although this will not hold true for actual deep foundations cases;

e) The program then provides the load-displacement curve of a vertically loaded pile. By default, it is derived for the maximum allowable displacement of 25 mm. The user, however, can change this default value. Apart from this curve, the program presents the distribution of normal shear forces along the pile, at each loading level. The program also enables the visualization of the relationship between skin friction and displacement at any pile element.

5. RESULTS AND DISCUSSION

The results will be presented and discussed in terms of comparison between backanalyzed geotechnical parameters and experimental data, in terms of load transfer curves, and in terms of numerical predictions of average skin friction and base pressure along loading level.

5.1 Geotechnical Parameters

The backanalysis consisted of selecting, by trial and error, input geotechnical parameters for the foundation GEO4 module, thus allowing this software to derive, with engineering precision, the load-displacement curve of the pile (in relation to experimental results from the sites). The predicted curves are well compared to the experimental results, as can be seen in Figures 6, 7 and 8, and in practical terms can be considered as a very good representation of the experimental loading curves. In order to tentatively obtain these curves some criteria was put in mind:

1) To be reasonably representative of the geological nature of the natural soil deposits studied herein – which means to be in the range of known values from other sources, as laboratory or in situ testing results, or backanalyzed parameters from other programs (as the values presented by Cunha et al. 2001);

2) To be considered as approximate values, or "estimated guess" of the real values, given all simplifications built in the numerical and experimental analyses;

3) To take into consideration the natural spatial variability of the deposit, in special related to ground level differences and geology between the sites;

4) To take into consideration the natural variability of the deposit, given its tropical and residual origin. This latter aspect has already been exemplified, via dilatometer test results, for the UnB site (see Mota et al. 2000).

Hence, Table 4 presents the assessment of the backanalyzed geotechnical parameters from the three testing sites. It shall be mentioned that more research effort still has to be put on this analysis (and are under way by Mota 2002). Nonetheless, in practical terms, as it will be seen, they already serve to carry out simulation analyses for piles founded in soil deposits similar to those studied herein.

Table 4. Backanalyzed Geotechnical Parameters from All Sites via GEO4 Software.

Sublayer Type	Depth (m)	ϕ' (deg)	c´ (kPa)	E (MPa)	K	$N^{*}_{(SPT)}$
UnB Site:						
Clay I	0-3	27	13	5	0.6	3
Clay II	3-8	27	14	13	0.6	4
Clay III	8-12	27	52	19	0.6	4-15
Rock	> 12 (Non Deformable)					
212N Site:						
Embankment	0-4	25	0	25	0.4	9
Clay A	4-8	27	15	20	0.4	9-5
Clay B	8-15	27	5	40	0.4	5-15
Clay C	15-25	27	5	100	0.4	15-20
Rock	> 25 (Non Deformable)					
Taguatinga Site:						
SClay	0-2.5	27	15	20	0.7	6
CSand	2.5-5	30	10	60	0.7	10
SSilt I	5-10	30	20	120	0.7	16-40
SSilt II	10-15	40	30	600	0.7	50+
Rock	> 15 (Non Deformable)					

OBSERVATIONS: (*) Average estimated values based on few, non-statistical, results
ϕ' = Effective Friction Angle; c´ = Effective Cohesion; E = Young Modulus; γ = Apparent Unit Weight;
ν_{soil} = Poisson Coefficient = 0.3; K = Coeff. of Earth Pressure; γ_{soil} = 16.5 kN/m^3
Structural Parameters: E = 20000 MPa (UnB, 212N) and 30000 MPa (Taguatinga), $\gamma_{concrete}$ = 25 kN/m^3

It is worthy to mention that a parametric analysis was already presented by Cunha et al. (2002) with the results of the 212N pile backanalysis. In this case, a forecast of the geotechnical behavior of piles with distinct values of diameter and length was carried out, and cross-compared, for this site.

The difference of results obtained from both UnB and 212N sites is somehow related, as explained before, to the spatial variability of the soil deposit, ground level differences, pile construction method differences (mechanical bored against continuous flight auger pile), and user considerations (the UnB site backanalysis was carried out by Mota 2002, and accepted herein without any modifications). Nevertheless, both analyses present geotechnical values that are within the range of obtained parameters for the Brasília porous clay deposit, and within the range obtained via numerical backanalyses in this same material (Table 1 and Cunha et al. 2001). Some observations, however, are required to further clarify this aspect to the reader.

The effective cohesion values obtained for 212N site are much lower than those from UnB site (and lower than the range expressed in Table 1). This is given by the fact that a ground water level has been found in the former site, hence considerably reducing the matric suction of the soil material. The cohesion values from the UnB site are, therefore, related to unsaturated soil conditions, found all year round in this particular site.

The Young moduli from 212N site are higher than those from UnB site. Indeed, part of the differences is related to ground level differences from both sites. For instance, Clays I to III from UnB site are mostly related to Clay A (plus a small part of Clay B) from 212N site, if one consider that the only difference between both sites is the topographic level. Under such aspect, the moduli differences are not high, since the highest E values from the 212N site are related to depths well beyond 15 m, where the average N_{SPT} blow counts linearly increases above 15 – depths and $N_{(SPT)}$ resistances that are not found in the UnB site. The differences also indicate that a further refinement of the 212N pile backanalysis would be required.

Table 4 also presents the results from the Taguatinga pile, where a distinct (from UnB and 212N sites) geology and type of pile is considered. Thus, the observed differences between the geotechnical backanalyzed parameters can be accounted for that, plus user considerations, i.e., slight variations on the assumptions assumed for each of the sites/piles during the analyses. As user consideration one may, for instance, consider the value adopted for the structural E, based on subjective approach as the fck and site quality of the concrete, and the coefficient of earth pressure K, based on subjective approach from laboratory/in situ data for the site combined with the effect of pile installation.

Taguatinga analyses also show parameters (ϕ and c´) that are consistent with expected values, either for the superficial porous clay or for the subjacent sandy silt strata (residual slate). It should be recognized, however, that the strength parameters for the residual slate are slightly higher than experimental values measured with samples from a nearby site (see Table 3), and, again, this is indicative that a further refinement of this first analysis would be desirable. The very high backanalyzed young moduli from the residual slate are indicative of the high stiffness of the material around the base of this pile. Indeed, the extremely high $N_{(SPT)}$´s, as presented in Table 4, and the very low settlements obtained during loading test, as presented in Figure 8, indicate that this pile was founded in a saprolite of slate rather than in a weathered soil. As it will be discussed later, this aspect is important for the analysis and has led to a large difference between backanalyzed base pressures and experimental estimated pressures – again, indicating that a further refinement of the analysis would be desirable.

5.2 Load Transfer Curves

The necessity of a further refinement for both 212N and Taguatinga analyses is also evident when comparing the load transfer curves of these same sites, as respectively done in Figures 9 and 10.

For instance, Figure 9 presents the numerical results obtained with the parameters of Table 4 for the load transfer curves of 212N pile. The numerical curves are directly compared to experimental results in this same figure. Although the comparison is fair, in special for the deepest sections of the pile, it can be considered valid as a first approximation of the real pile behavior. It is observed that, for the ultimate loading condition (1100 kN), ≈37% of the applied load at the pile’s head was transmitted to the base of this pile (numerical prediction). This value is higher than the value that has been experimentally measured at this same loading stage (as given before, equal to 25%). Besides, for the working condition (600 kN), the software has estimated that ≈6% of the applied load at the pile’s head would be transmitted to the base of this pile. This value, in this case, is lower than the value that was experimentally measured at this same loading stage (as presented by Cunha et al. 2002, equal to 12%). Nevertheless, the comparison, at ultimate condition, of the average unit skin friction between numerical (32 kPa) and experimental (39 kPa) values was reasonably good. For comparison purposes, the average experimental unit skin friction of the UnB pile, also at failure condition, was 36 kPa.

Hence, although refinement is required, the results can be already considered suitable from a practical point of view, if one accepts as valid and within tolerable range (in engineering terms) the errors involved in this first series of analyses.

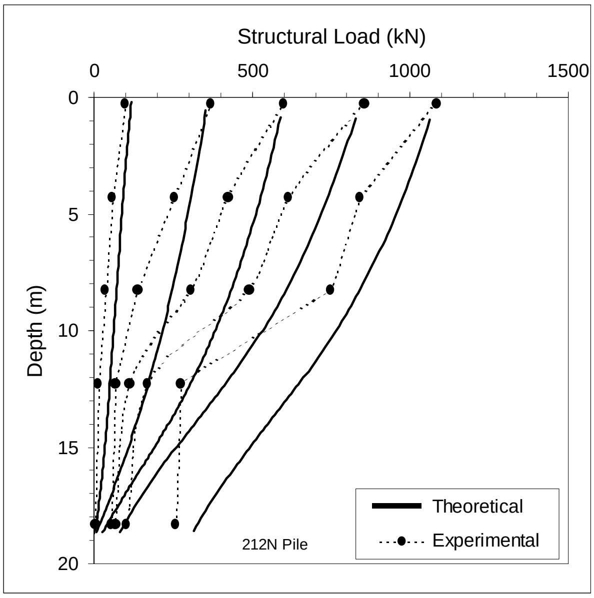

Figure 9. Load Transfer Curve – 212N Pile.

Figure 10 presents the comparison of load transfer curves for the Taguatinga pile. It indicates that a fairly good result was obtained, and a better (than 212N pile) agreement was achieved. Similarly as before, one can affirm that the analyses can be already considered as valid from a practical point of view. Indeed, the comparison of average unit skin friction and percentage of base load between numerical and experimental values is good. It is observed that, for the failure condition (3200 kN), ≈42% of the applied load at the pile's head was transmitted to the base of this pile (numerical prediction). This value is very close to the value that has been experimentally measured throughout the test (as given before, in the range of 40%). At working condition (1600 kN) however, the software has estimated that ≈11% of the applied load at the pile's head would be transmitted to the base of this pile. Also at this latter condition the comparison of average unit skin friction between numerical (≈56 kPa) and experimental (≈40 kPa) values was reasonably good. The discrepancies are again related to the fact that the pile was founded in a very hard soil stratum (saprolite), which was not perfectly simulated in this first analysis.

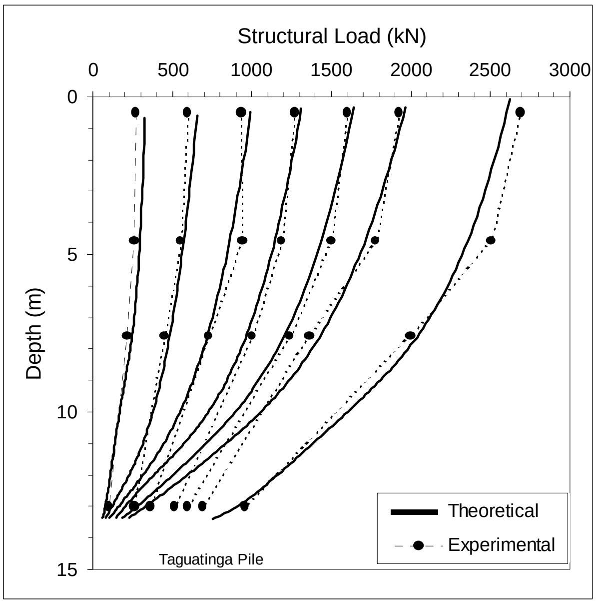

Figure 10. Load Transfer Curve - Taguatinga Pile.

5.3 Skin Friction and Base Pressure

In both 212N and Taguatinga cases, it has been observed that small discrepancies between experimental and numerical predictions were found. In fact, such discrepancies were higher in terms of base pressure rather than average skin friction. This can be readily seen by the comparison of results expressed in both Figures 11 and 12.

The good agreement for the skin friction is directly related to the fact that a good adjustment was done in terms of strength parameters for the soil springs along the pile's shaft. Indeed, as commented before, the strength parameters in both sites were consistent with measured experimental data, i.e., within acceptable differences.

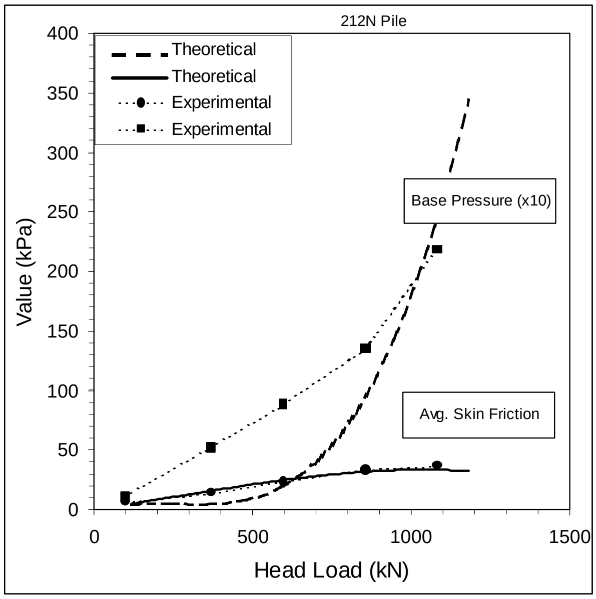

Figure 11. Comparative Results for the 212N Pile.

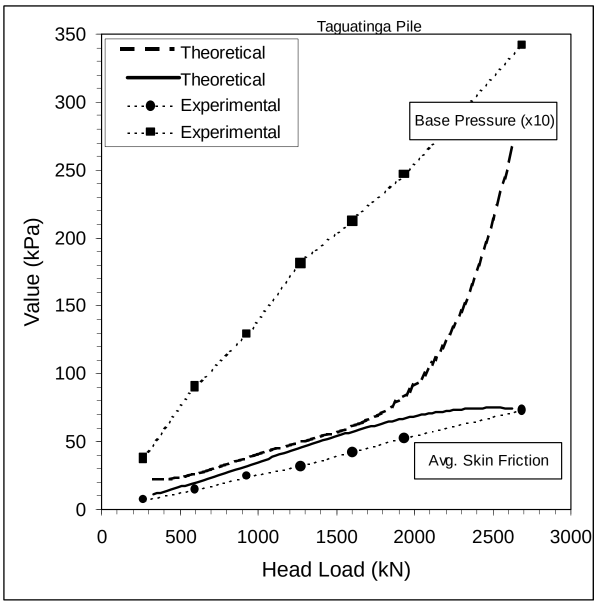

Figure 12. Comparative Results for the Taguatinga Pile.

The lack of agreement for the base pressure is directly related to the fact that the analyses lacked to properly model the last layer, or the foundation layer, where extremely high values of unit pressures took place and were (indirectly) measured. In numerical terms, it is assigned a unique and particular soil spring for this last layer. Since it is evident from Figures 11 and 12 (in particular the latter one) that the numerically predicted base pressures were substantially lower than the measured values, one can conclude that the stiffness of this spring was not properly incorporated in the analyses. A fine readjustment of the stiffnesses for both situations, increasing towards more representative values, would certainly improve the matching quality of the load transfer curves of Figures 9 and 10, in special at the deepest pile sections.

Nevertheless, as stated before, the results already suffice for practical design applications. This is so given the good matching quality of the load-settlement curves (Figures 6 to 8) and the reasonable good response of the numerical predictions in terms of load transfer mechanism (Figures 9 and 10) and average skin friction (Figures 11 and 12).

The experienced gained with these analyses has demonstrated that a deep strata, stiffer than the overlying layers and leveled to the pile's base, shall be incorporated in the numerical backanalysis. This holds particularly true for the Taguatinga site, where the pile was founded on a hard saprolite medium. It was also learned the importance of having instrumented piles at the site. Although such providence may increase construction costs, the assistance of the instrumentation in guiding and benchmarking the backanalysis is immeasurable.

6. CONCLUSIONS

This paper emphasized the use of a numerical methodology to derive backanalyzed geotechnical parameters that are useful for design applications in the civil engineering foundation area. Once calibrated, this methodology can be used in a further parametric analysis for the studied site, verifying the adequacy of distinct pile geometries in order to optimize the foundation design. The mathematical model involved in this methodology is based on a well established semi-analytical solution coded on an industrial software program, which can be readily available.

The use of this software to successfully simulate instrumented field loaded, large scale, bored foundations, has validated the high versatility and potential that this particular program has for usage in practice. This also holds, in particular, for the difficult (non classical) case histories considered herein, which consisted of foundations founded on the tropical and stratified soils of the Federal District of Brazil.

Hence, given all the previous discussion it is possible to conclude that:

1. The obtained values from Table 4 are a reasonable representation of the correct magnitude of the values to be used in design (practical applications / parametric analyses) in similar sites, when adopting the same pile construction technique and numerical methodology. Nevertheless, it is also concluded that a refinement of the analyses would be desirable to lessen the differences;
2. The software has proven to represent reasonably well the behavior of vertically piles loaded on tropical stratified deposits of the Federal District, and has the potential for usage in practical applications. Differences between the results can be accounted for distinct geological, topographical and pile construction conditions;
3. To consider this numerical technique in design it is desirable to initially calibrate the software in a backanalysis stage, using large scale field load testing results. This will account for the geology and pile construction methodology of the particular site;
4. The instrumentation of the loaded pile considerably enriches the design process, by guiding and benchmarking the backanalysis.

It shall be finally pointed out that, given the reduced number of foundations and the limited spatial size of the studied area, it is evident that more studies are still necessary. Nevertheless, the methodology of data interpretation, the gained experience, and the site results, can already be seen of practical interest for those involved with foundation design in Brasília and elsewhere.

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