

# Acquired Knowledge on the Behavior of Deep Foundations Vertically and Horizontally Loaded in the Soil of Brasília

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**Abstract.** This paper presents and discusses a limited set of results from an extensive testing program which has been carried out since 1995 at the University of Brasília Experimental Research site. This site is underlain by the typical tropical, unsaturated and collapsible soil deposit of the Federal District, which has been thoroughly studied via an on going program of laboratory and *in situ* geotechnical tests. In this specific location, several isolated deep foundations were constructed and vertically and horizontally loaded with distinct soil moisture conditions. These foundations, and the soil deposit, do represent typical conditions that occur in other areas of the region, and have therefore been simulated under semi controlled conditions. Since 1995, a large number of research theses were involved with this particular theme, and for the first time some of their main results have been condensed and discussed in the same publication within a logical framework. The knowledge in terms of the observed site behavior, its hypothetical explanation, and some theoretically or empirically derived design variables are shown herein. General conclusions in terms of the vertical and horizontal design values are given together with experimental observations on attained displacements under distinct load levels. The influence of specific external factors on the results is studied; for instance the dissimilar behavior of piles constructed with different methods, or the influence of the weather seasons on the bearing capacity values. From this overall set of data one can have an insight into the complex physical mechanisms involved with the performance, and the difficult simulation, of deep foundations founded in tropical “non-classical” soils. It is a collection of results with value for researchers and practitioners at both regional and national levels.

**Keywords:** tropical soil, deep foundation, experimental load test, bearing capacity, displacement, acquired experience, design values.

## 1. Introduction

The pre-designed Brazilian capital Brasília, located in the Federal District of Brazil, was built in the early 60's to house the main Governmental administrative institutions and its public employees. After 50 years (celebration in 2010) it has grown, and is still expanding, considerably more than what was initially envisioned by its founders. The city borders and inner “sectors” have advanced through different (geological) zones of this same District, thus allowing the use of distinct techniques for deep foundation deployment and design during the last half century.

Given such conditions, one can conclude that foundation and *in situ* testing are two demanding research (and practical, in terms of design) topics at the Brazilian capital. Besides, given its distance from major Brazilian cities with already established foundation practice, together with the particular conditions of the regional tropical subsoil, design solutions for the area must be applied solely based on local expertise, not on foreign ones. This point is clearly exemplified, for instance, when one remembers the early stages of foundation construction in this city. At that time, most of the solutions incorporated the accumulated experience of “outside” engineering firms, which led

later on to cracking problems in few buildings by the absolute unawareness of the collapsible conditions of the Brasília “porous clay”.

Perhaps also because of that, more sound and research-based solutions and techniques have been and are since then under development (and scrutiny) by designers, contractors and researchers of the region. These latter under the support and investigative scope of the Research Group on Foundations, *In Situ* Testing and Retaining Structures, *i.e.* the “GPFees” Group ([www.geotecnia.unb.br/gpfees](http://www.geotecnia.unb.br/gpfees)) of the University of Brasília (UnB). As the name states, this group is composed by Professors, technicians and students of the Geotechnical Graduation Program of this university who are in charge, among other things, of the understanding of the problem and the development of sound based design solutions tied to questions related to these geotechnical construction works.

It can be said that the good academic-industry interaction has not only advanced the existing and the new foundation technologies of the city, but it also stimulated a pioneering use of high level *in situ* tests (such as the cone test CPT, the pressuremeter, the Marchetti Dilatometer, pile integrity tests, and others), as the design basis for the foundations constructed within the tropical soil of the region.

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The behavior of such foundations founded in the local soil of Brasília is one of the most important research topics from this Group, also because of the lack of information on the complex (soil-structure) response that takes place around and below these foundations. This knowledge is fundamental to aid in understanding and further design, as well as furnishing information on the reliability of the existing theories and on the establishment of design criteria for new foundation techniques.

Once it was understood that the necessity of acquiring knowledge was intermingled with the lack of design criteria in the region, the University of Brasília decided in 1995 to launch a major research project in this area. This was initially done in order to enhance the knowledge of the behavior *in situ* of the distinct foundation types which were founded in the predominant subsoil of the Federal District<sup>1</sup>. At such occasion it was decided to carry out horizontal and vertical field loading tests on the different types of locally used deep foundations, and to understand the behavior and apply known (analytical, empirical, numerical) theories to simulate the results. These foundations were constructed at full scale size within the University of Brasília campus, specifically at the Experimental Site of the Geotechnical Graduate Program (Mota *et al.* 2009). A large effort was also undertaken in association with local engineering companies to characterize this site, by performing advanced *in situ* tests, standard and high level laboratory tests, and other experimental techniques.

This paper therefore conveys a limited, summarized fraction, of the gathered knowledge so far, with the focus on the behavior of deep foundations under vertical and horizontal load, the estimation of their bearing capacity both at horizontal and vertical directions, the assessment of their

displacement at working loads, and the beneficial or detrimental influence of some external variables in their performance, such as their construction condition/type or the local weather seasons, among others.

The acquired experience to be discussed herein undoubtedly serves as a start point to design projects in the region, and in others of similar characteristics. It is of value to practitioners and researchers, and it was originally published (in Portuguese) within several Dissertations and Theses from members of this same research Group. Their work will be stated within the following sections (Perez, 1997, Jardim, 1998, Lima, 2001 and Mota, 2003) together with international publications (in English) that also served to consolidate in a logical sequence the main points of information of this paper.

## 2. Experimental Research Site

The Experimental Research Site of Foundations and *In Situ* Testing has already been portrayed in several publications. Its main characteristics will be presented herein, but the interested reader can review its detailed aspects published in Mota *et al.* (2009), Anjos (2006) and Cavalcante *et al.* (2006), among others.

This site is located in the city of Brasília, which was established 50 years ago in the highly elevated (close to 1000 m) plateau of the central area of Brazil, as depicted in Fig. 1. The city of Brasília was erected in a special unit of the Federation, called the “Federal District”, a geometrically designed rectangular area of 5814 km<sup>2</sup>. According to many, this city has the shape of an airplane, and being so, one can notice that the Experimental Site is located in its upper north “wing” section, within the university campus. Figure 1 also shows this location, which is complemented

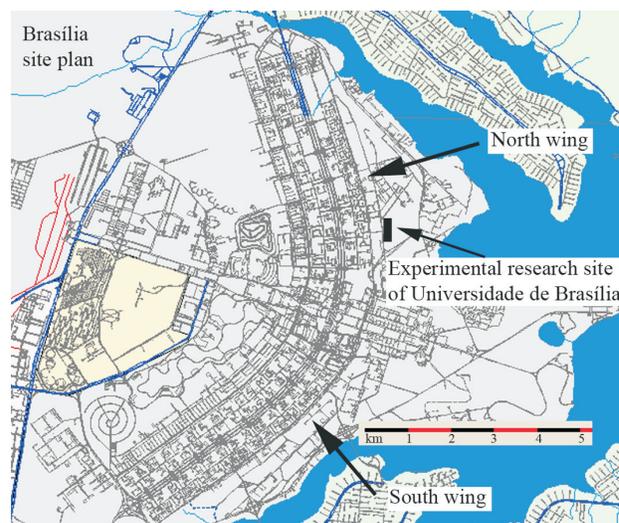


Figure 1 - Location of the city of Brasília and the Experimental Site.

<sup>1</sup> Later on the research objectives also encompassed questions related to *in situ* tests, unsaturated soil conditions and others. It should be acknowledged that, before this time, research on foundations was undertaken in this same site, although in a more limited manner.

by Fig. 2 where a more detailed section of the site, close to the Civil & Environmental Engineering Dept., is presented.

Within the Federal District it is common the occurrence of extensive areas covered by weathered latosol of the tertiary-quaternary age. This soil has a variable thickness (in the range of 10 to 30<sup>+</sup> m), which depends on several factors as the topography, the vegetal cover, and the mother rock. In localized points of this area the top latosol overlays a saprolitic/residual soil with a strong anisotropic mechanical behavior (Cunha & Camapum de Carvalho 1997) and high blow count resistance ( $N_{SPT}$ ) from the Standard Penetration Test (SPT), which is originated from a weathered, folded and foliate slate typical of the region. Given its characteristics, this is the soil horizon which bears most of the (highly loaded) end bearing piles for high-rises in Brasília.

The superficial latosol has a dark reddish coloration, and displays a much lower ( $N_{SPT}$ ) penetration resistance and much higher permeability than the bottom saprolitic/residual soil. According to Araki (1997), the high porosity and weak particle cement bonding (iron and aluminum oxides) of this soil are originated from typical physic-chemical geological processes associated to the superficial soils of

the Brazilian Central Plateau, whereas combined lixiviation and laterization processes have an important role due to well defined and extreme “wet” and “dry” seasons of the region (weather seasonality).

In the particular area occupied by the experimental research site the lateritic “porous clay”, as it is known, has a thickness of  $\approx 8$  m, followed by a transition zone overlying the saprolitic/residual soil of slate, as depicted in Fig. 3.

The figure also presents the average (arithmetic mean) values of  $N_{SPT}$  blow counts ( $N_{avg}$ ) and maximum torque measurements ( $T_{max,avg}$ ), plus respective coeff. of variations in percentage, for each meter depth at the site. This data comes from 5 SPT tests (SP1 to SP5) carried by Mota (2003) - situated in the layout figure to be shown later in this paper.

Table 1 presents the geotechnical characterization of the site, based on soil classification tests also carried out by Mota (2003) including grain size proportions both without and with a deflocculating agent. By breaking down the structure with this agent, the grain size curve of this soil shows a greater concentration of clay-size particles. In this table one can also notice the low unit weight, and high void ratio of this deposit.

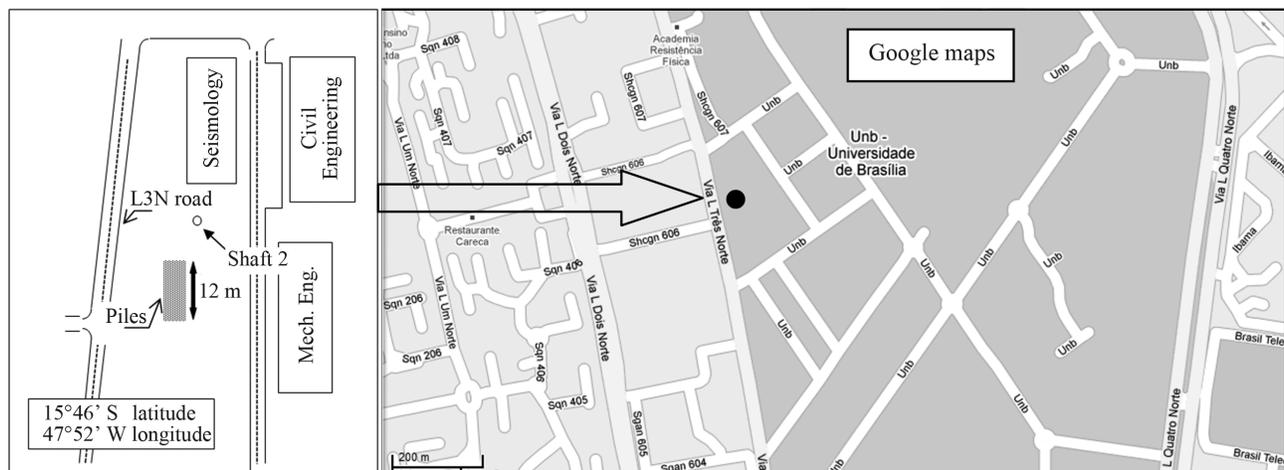


Figure 2 - Location map of the site within the University Campus.

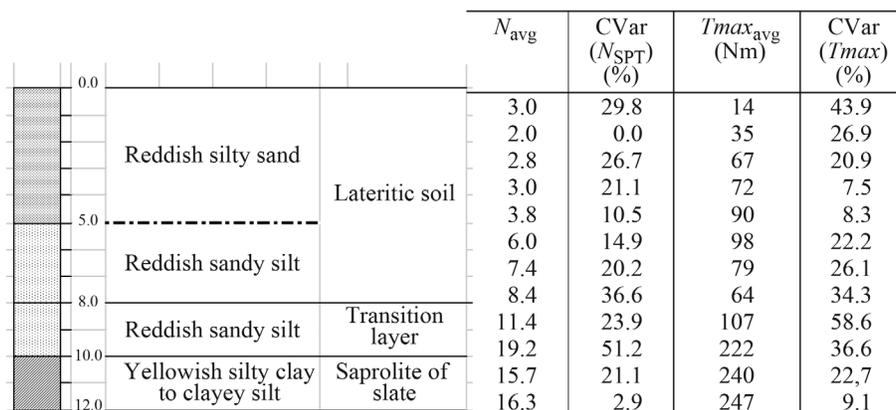


Figure 3 - Simplified geotechnical profile of the Research Site.

**Table 1** - Typical Geotechnical values of the site (after Mota 2003).

Parameter	Depth (m)									
	1	2	3	4	5	6	7	8	9	10
$\gamma_{dry}$ (kN/m <sup>3</sup> )	10.2	10.4	11.5	11.5	12.0	12.0	12.8	13.9	13.8	13.3
$\gamma_{nat}$ (kN/m <sup>3</sup> )	13.3	13.7	14.7	14.5	15.0	14.4	15.4	18.0	17.8	17.5
$\gamma_{sat}$ (kN/m <sup>3</sup> )	16.5	16.5	17.1	17.0	17.5	17.3	17.8	18.6	18.8	18.5
$G_s$	2.7	2.7	2.7	2.7	2.7	2.6	2.7	2.7	2.8	2.8
$e$	1.6	1.6	1.3	1.3	1.2	1.1	1.1	0.9	1.0	1.1
$n$ (%)	61.6	61.1	56.0	55.9	55.6	53.5	51.7	47.2	49.0	51.9
Gravel ND (%)	0.2	0.2	0.7	0.8	1.4	2.1	4.3	3.6	0.6	0.0
Sand ND (%)	56.2	56.2	53.2	53.0	49.2	34.9	30.1	42	10.2	1.4
Silt ND (%)	51.4	35.9	34.2	43.1	48.6	61.4	61.9	51.9	86.8	79.5
Clay ND <sup>1</sup> (%)	2.2	7.7	11.9	3.1	0.8	1.6	3.7	2.5	2.4	19.1
Gravel WD (%)	0.2	0.2	0.7	0.8	1.4	2.1	4.3	3.6	0.6	0.0
Sand WD (%)	41.5	41.5	41.6	33.7	31.6	25.7	22.7	33.8	10.2	3.4
Silt WD (%)	24.9	29.2	25.7	26.3	26.5	22.9	24.6	27.4	80.4	93.2
Clay WD <sup>2</sup> (%)	33.4	29.1	32.0	39.2	40.5	49.3	48.4	35.2	8.8	3.4
$w_L$ (%)	38	36	39	41	45	44	46	43	44	46
$w_p$ (%)	28	26	29	29	34	33	35	34	26	30
PI (%)	10	10	10	12	11	11	11	9	18	16

<sup>1</sup>Clay portion with no deflocculating agent; <sup>2</sup>Clay portion with deflocculating agent.

$\gamma$  = unit weight,  $G_s$  = specific gravity,  $e$  = void ratio,  $n$  = porosity,  $w$  = Atterberg limits, PI = plasticity index.

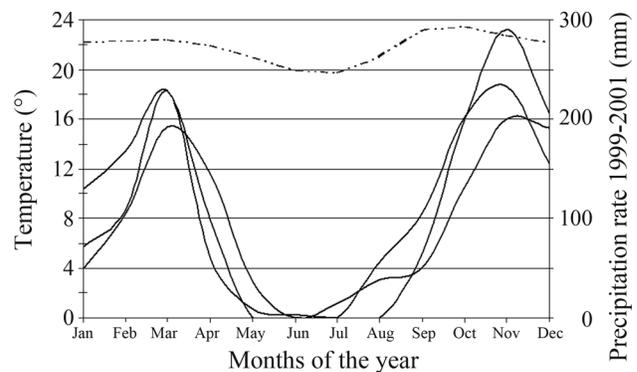
All the pile load tests were carried out in a zone not larger than 30 x 30 m, within this typical profile. The first layout of piles, of distinct construction methods, was established in mid 1995 within a grid of around 4 x 12 m (see dotted rectangle of Fig. 2). Around this original grid, several other foundations combined with site and laboratory investigations (under distinct research theses) were carried out. Manually excavated shafts, as “shaft 2” from this same figure, were bored to obtain samples for further laboratory tests. Given its tropical nature, it is obvious that somewhat distinct geotechnical values were obtained from point to point in the site, but around the range specified by the typical results of Table 1.

Figure 4 shows quantitatively the typical precipitation rates of Brasília, by records measured during years 1999 to 2001 (Mota, 2003) in the INMET station of Brasília, located around 7 km from the site. As one clearly notices, there are two distinct weather periods, being the “dry” season related to months May to September. This aspect may influence pile behavior during load, as will be shown later on, and is referred as the “weather seasonality” effect.

### 3. Experimental Study

#### 3.1. Pile load tests

Horizontally and vertically loaded piles constructed with distinct methodologies and under different soil condi-



**Figure 4** - Typical precipitation and temperature in Brasília (modified after Mota, 2003).

tions have been tested during research at the Experimental Site. Figure 5 presents the location of such load tests, related to the previous aforementioned work from Perez (1997), Jardim (1998) and Mota (2003). This figure is linked to Fig. 2, and complements it in large detail.

The tests depicted in this figure are described in Table 2, where their general characteristics are given. Pile geometric conditions, as diameter and length ( $D$  and  $L$ ), date of testing and loading type (slow or quick maintained), maximum attained load and displacement ( $P_{max}$  and  $\Delta_{max}$ ), as well as weather seasonality (wet or dry) are detailed for each test.

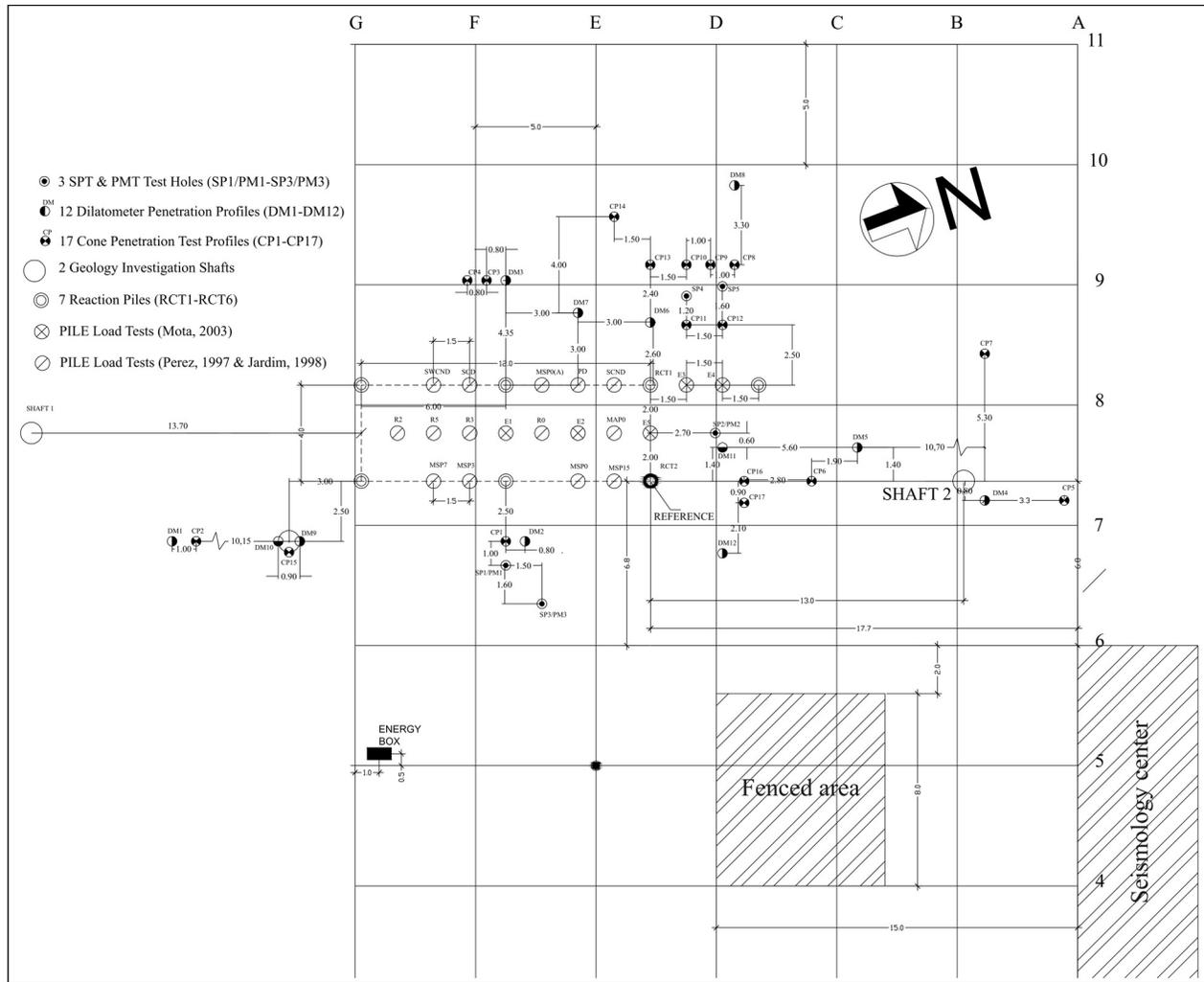


Figure 5 - Layout of some deep foundations and *in situ* tests of the Experimental Site (each square has 5 x 5 m).

All the tests were done in accordance to the recommendations put forward by the Brazilian NBR 12131 (ABNT, 2006) standard, and they consisted of (slow and quick, according to Table 2) maintained tests in two categories.

The loading tests were performed in loading intervals of 20% of the working load up to failure. The piles were subsequently unloaded in approximate 4 intervals. These load tests adopted a reaction frame and “reaction” piles few meters apart. Both the top foundation block and the reaction frame were monitored for tilting and vertical displacements, by using 0.01 mm precision dial gauges. A 1000 kN hydraulic jack was used in conjunction with a 1000 kN precision load cell.

The first testing category (Perez, 1997) consisted of vertically loaded piles with the soil in its natural moisture content, as follows:

a) Four mechanically screwed (or bored cast-in-place) piles: labeled as MSP0, MSP3, MSP7 and MSP15. They were constructed with concrete at different days after the soil excavation (0, 3, 7 and 15 days, according to above

nomenclature, where “0” means just after excavation). A fifth pile labeled MSP0(A) was also constructed and field loaded. It was cast in place just after excavation, but it was composed by a concrete mixed with a special expander additive. All the bored (MSP) piles were 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. No soil was removed during auger introduction, and, after the final depth was reached, the auger was withdrawn leaving a freshly excavated hole. Cleaning of the base of the hole was not carried out, although care was taken to try stopping auger rotation on more “competent” strata. The designed reinforcing bars were then introduced and, in the MSP0 and MSP0(A) piles, the concrete was promptly poured by using the transportable service of a local concrete company. The MSP piles had a length of ≈ 8 m and diameter of ≈ 30 cm, and were loaded by slow maintained tests;

**Table 2** - General characteristics of the pile load tests.

Nomenclature & pile type	$D$ (m)	$L$ (m)	Test date	Load type	Ref.	Observation
Vertically Loaded Piles						
MAP0: Manually bored pile molded after excavation	0.28	7.9	Jun 1997	SML		$P_{max} = 240$ kN, $\Delta_{max} = 44.7$ mm
MSP0(A): Bored pile molded after excavation with additive	0.30	8.4	Jun 1997	SML		$P_{max} = 360$ kN, $\Delta_{max} = 28.5$ mm
MSP0: Bored pile molded after excavation	0.30	7.9	Jun 1997	SML		$P_{max} = 320$ kN, $\Delta_{max} = 28.5$ mm
MSP3: Bored pile molded 3 days after excavation	0.30	8.0	May 1997	SML		$P_{max} = 320$ kN, $\Delta_{max} = 9.5$ mm
MSP7: Bored pile molded 7 days after excavation	0.30	8.0	May 1997	SML		$P_{max} = 320$ kN, $\Delta_{max} = 29.6$ mm
MSP15: Bored pile molded 15 days after excavation	0.30	8.0	May 1997	SML		$P_{max} = 280$ kN, $\Delta_{max} = 19.1$ mm
R0: Root pile with no pressure	0.22	10.2	Jun 1997	QML		$P_{max} = 330$ kN, $\Delta_{max} = 26.1$ mm
R2: Root pile with 200 kPa of injection pressure	0.22	10.1	May 1997	QML	Perez (1997)	$P_{max} = 525$ kN, $\Delta_{max} = 41.8$ mm
R3: Root pile with 300 kPa of injection pressure	0.22	10.0	May 1997	QML		$P_{max} = 360$ kN, $\Delta_{max} = 27.7$ mm
R5: Root pile with 500 kPa of injection pressure	0.22	10.0	May 1997	QML		$P_{max} = 360$ kN, $\Delta_{max} = 29.6$ mm
SCD: "Strauss" cased type pile with compacted concrete	0.30	8.9	May 1997	SML		$P_{max} = 400$ kN, $\Delta_{max} = 8.7$ mm
SCND: "Strauss" cased type pile without compaction	0.30	8.1	Jun 1997	SML		$P_{max} = 280$ kN, $\Delta_{max} = 20.7$ mm
SWCND: "Strauss" uncased type pile without compaction	0.30	8.2	May 1997	SML QML		$P_{max} = 300$ kN, $\Delta_{max} = 9.7$ mm
PD: Precast concrete driven hollow pile	0.33	8.4	Jun 1997	SML		$P_{max} = 205$ kN, $\Delta_{max} = 10.4$ mm
E1: Bored instrumented floating pile	0.30	7.6	Feb 2000	SML		Wet Season
E2: Bored instrumented floating pile	0.30	7.2	Aug 2000	SML		Dry Season
E3: Bored instrumented floating pile	0.30	7.8	Oct 2001	SML	Mota (2003)	Wet Season
E4: Bored instrumented floating pile	0.30	7.3	Mar 2001	SML		Wet Season
E5: Bored instrumented floating pile	0.30	7.8	Jun 2000	SML		Dry Season
Horizontally loaded piles						
RCT1: Bored pile used for reaction at vert. test	0.5	10.0	Sept 1997	QML		Nat. & "inundated" conditions. Max $y_{0i} = 3.7$ , $y_{0i} = 15.7$ mm
RCT2: Bored pile used for reaction at vert. test	0.5	10.0	Sept 1997	QML		Nat. & "inundated" conditions. Max $y_{0i} = 5.0$ , $y_{0i} = 10.6$ mm
R2: Root pile with 200 kPa of injection pressure	0.22	10.1	Sept 1997	QML		Nat. & "inundated" conditions. Max $y_{0i} = 3.6$ , $y_{0i} = 4.4$ mm
R3: Root pile with 300 kPa of injection pressure	0.22	10.0	Sept 1997	QML	Jardim (1998)	"Inundated" conditions only. Max $y_{0i} = 9.4$ mm
R5: Root pile with 500 kPa of injection pressure	0.22	10.0	Sept 1997	QML		Natural conditions only. Max $y_{0i} = 16.1$ mm
PD: Precast concrete driven hollow pile	0.33	8.4	Sept 1997	QML		Nat. & "inundated" conditions. Max $y_{0i} = 11.2$ , $y_{0i} = 13.1$ mm

b) One manually augered (or bored cast-in-place) pile, defined as MAP0, and casted just after soil excavation in a similar way as previously described for the MSP0 pile. In the former case, however, the excavation was done with a shell type auger that was hand augered in the field by adopting successive 1 m steel rods. The MAP0 pile had a final approximate length of 8 m and diameter of 28 cm. The same loading as before was used;

c) Three “Strauss” (Brazilian label) type piles defined as SWCND, SCD and SCND, they were also bored cast-in-place piles. The Strauss pile is a locally used deep foundation which has the execution process close, but not exact, to the one used for “Franki” piles. They were constructed by adopting a cylindrical metallic shell with a bottom valve bailer that was handled in the field by means of a hoist mounted on a tripod. This shell was continuously advanced as the bailer removed the soil softened by a bottom punching with auxiliary water. The hole was encased for two of the piles (SCD and SCND), and not encased for the third one (SWCND). The casing was punched into the hole as soon as the shell excavation stage finished. This operation, however, was done in steps, since the shell had to be lifted up to surface several times to be internally cleaned of its “entrapped” soil. At the end of the excavation, at the desired depths, the bottom of the hole was cleaned out, the fix rebars were introduced and fresh concrete was poured. For one of the piles (SCD) the concrete was compacted afterwards by using a 2.5 kN hammer falling onto it, whereas for the other piles (SCND, SWCND) the concrete was simply poured. All the piles had a final approximate length of 8 m and diameter of 30 cm. Same field loading as before;

d) One precast driven centrifuged (displacement) concrete pile labeled as PD, was dynamically inserted into the soil by using a 32 kN (free fall) drop hammer falling from a height of 30 cm. A wood cushion was used to soften the impact on the top of the pile, and it was mounted together with the hammer on the leads of a standard crawler crane. The precast (hollow, “SCAC” company type) pile had a final length of 8.4 m with an external and internal diameter of, respectively, 33 cm and 25 cm. Same loading as before;

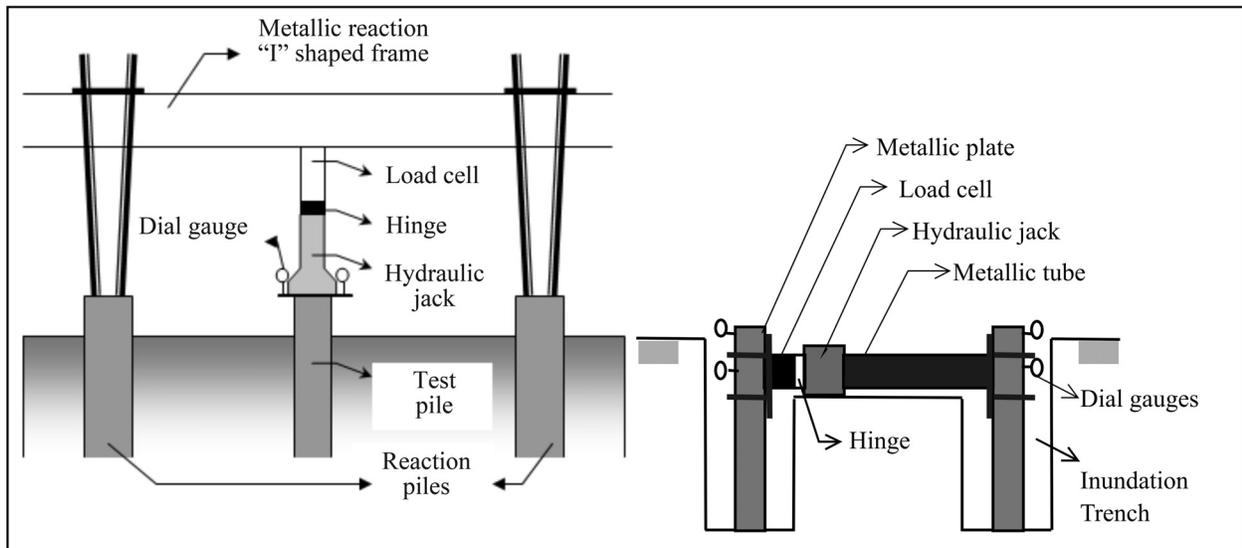
e) Four injected type piles (cast-in-place with pressure, locally know as “root” pile - with very distinct construction aspects from the known “micropile” type). They were constructed by adopting distinct injection pressures (0, 200, 300 and 500 kPa) during the formation of the mortar shaft. They are defined herein as R0 for the pile without injection pressure (mortar just poured from surface), R2 for an injection mortar pressure of 200 kPa, R3 for an equivalent pressure of 300 kPa and R5 for a pressure of 500 kPa. These piles were executed with a specially devised drill rig which operated hydraulically. The soil was excavated by a continuous and static introduction of a rotating casing with pressurized water. The water “washed out” the generated mud in front of this casing, opening a small annular gap be-

tween the casing and the excavated hole. Once drilling was finished, the interior of the casing was cleaned up and the fix rebars were introduced. Mortar was then poured inside the casing until it was filled. The top of the casing was then connected to an air pressurizing system, and air pressure was applied to the inner fluid mortar. By simultaneously applying air pressure and lifting up the casing, it was possible to form the corrugated pile’s shaft (for the piles with injection pressure). This operation was done in sequence, continuously filling up the remaining casing with fluid mortar, thus leading at the end to piles with an approximate length of 10 m and final average (nom.) dia. of  $\approx 22$  cm. They were loaded by quick maintained tests;

f) Five mechanically excavated and fully electronically instrumented piles (Mota, 2003), defined as E1 to E5. They were executed with similar conditions as those aforementioned bored piles, but loaded at specifically distinct weather seasons.

The second category of tests (Jardim, 1998) consisted of horizontal load tests on some of the piles mentioned before, which had been previously tested under vertical load (the only exceptions were for the reaction RCT1 and RCT2 piles). However, given the experience obtained during the previous stage, some modifications were introduced. The tests were done with fast loading intervals (“quick maintained load tests”) with loading intervals of 10% of the working load up to failure, and with a compulsory stabilization time of 5 min (according to the NBR12131 standard). The obtained loading rate was in the range of 3.6 kN/min (three times faster than the rate of the previous stage). Similar loading equipment and instrumentation as described before were used for these tests, with the difference that a lower capacity (500 kN) hydraulic jack was adopted, and the piles were now tested against “each other” without reaction frames, as portrayed in Fig. 6. In this figure it is also noted that the soil served as a support for the loading equipment, *i.e.*, the hydraulic jack and its extension made of a metallic tube used to transmit the load from one pile to another. In order to avoid undesirable tilting during the field test, a cylindrical hinge was adopted between one end of the jack and the load cell for both (vertical and horizontal) cases. Rigid metallic plates were fixed to both tested piles by means of metallic rings, so that a constant distance was maintained between the load application point and the base of the trench, at the pile/soil interface. A constant distance was also maintained between the upper dial gauge, at the pile’s head, and the load application point.

These loading tests were carried out in two phases, with the soil in its natural field water (moisture) content in the first phase and in its “inundated” conditions in the second phase. In order to “inundate” the soil a 60 cm deep trench was dug all around the piles, with a diameter of approximately 2 times each pile’s diameter. The inundation took place after the horizontal loading tests with the soil in the natural moisture content conditions. Soil inundation



**Figure 6** - Sketch of vertical and horizontal loading tests (modified after Jardim, 1998).

was achieved by filling the trenches with water and keeping the water level constant for about 48 h before the loading tests. This procedure tried to simulate the effects of a heavy rain on the soil deposit, given the known collapsibility characteristics of the Brasília porous clay. The following piles and loading sequence was adopted for the horizontal tests:

a) Reaction piles RCT1 and RCT2: They were tested against each other, with the soil initially at the natural moisture content condition, and later inundated;

b) Precast driven centrifuged concrete pile PD: Tested against the reaction piles, also with the soil in its natural and later in inundated conditions;

c) Root piles R2, R3 and R5: They were tested against each other, but in accordance to the following sequence: R2-R5 with the soil in its natural moisture content conditions, leading to the structural rupture of R5, followed by R2-R3 with the soil under “inundated” conditions, which also led to the structural rupture of R3.

All the tests were carried out at the final stage of the 1997 dry season, between August to October. Figures 6 and 7 respectively present the sketch on how the load tests were done and some of the load-deflection resulting curves. Notice that  $\Delta$  is the vertical deflection on top of the pile and  $y_0$  is the horizontal deflection at ground level. Maximum values of  $y_0$  at either natural (“n”) or inundated (“i”) soil conditions attained during the tests are also given in Table 2.

### 3.2. Pile echo tests

Pile echo tests, or simply PET, have been carried out in some of the piles analyzed in this paper. This test uses the Pulse-Echo method for quick quality control of a large number of piles, in order to verify their integrity and length.

The pile top is struck with a lightweight handheld hammer. The reflected wave is captured and analyzed by the PET’s digital accelerometer to provide information regarding the length and shape of the pile.

These tests were carried out in 2010 in order to check the geometrical information provided in Table 2, by using a recently acquired PET tester under an ongoing research project from the GPFees Group.

Although many of the piles could not be found anymore in 2010, given the alterations that the Experimental site suffered throughout the last years<sup>2</sup>, those which could be tested confirmed the geometry expressed in aforementioned table, assembled with data from the original theses at this site.

## 4. Results and Discussion

The analyses are subdivided into major topics, as loading in the vertical direction, and its derived parameters, and loading in the horizontal direction. They are presented and discussed next.

### 4.1. Vertical direction

#### 4.1.1. Construction methodology effect

This particular discussion has already been presented elsewhere (Cunha *et al.*, 2001) and is addressed here in its essence to enhance the final conclusions.

The vertical failure load was defined as the average value between the predictions of Brinch-Hansen (1963) and Mazurkiewicz (1972), since, according to Perez (1997), these methods yielded failure loads which were closer to the “physical failure” values (asymptote of the load-de-

<sup>2</sup> The current Site is scheduled to be closed very soon, given the lack of space and university policy. A new area has been provided for this purpose within UnB campus.

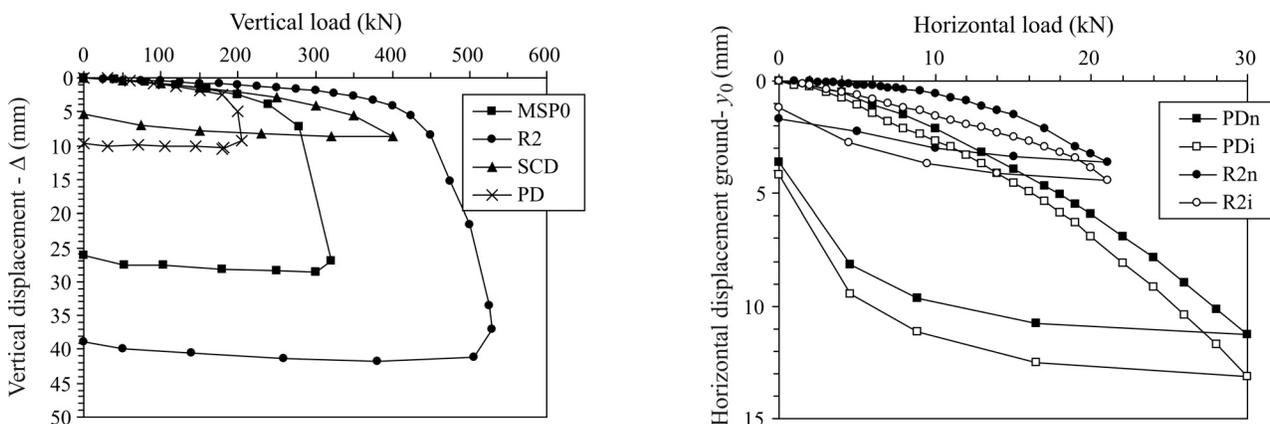


Figure 7 - Some of the obtained load-displacement curves (modified after Jardim, 1998).

flexion curve) from each of the foundations. The only exception is for the Strauss piles, in which the NBR6122 (ABNT, 2010) standard method was adopted because it presents a procedure to define the failure load for continuously increasing testing curves, in which the maximum load is not clearly depicted. This feature was noticed for the Strauss piles.

Figure 8 presents a plot of the vertical failure load of all piles tested by Perez (1997). In regard to this figure some observations can be given:

- The mechanically bored pile with the expander additive (MSP0(A)) had a failure load 8.0% higher than the equivalent load of the pile without the additive (MSP0);
- The failure load of the mechanically bored piles (MSP) decreased with the time span between excavation and casting (from MSP3 onwards). The failure load has unexpectedly increased 8.5% from MSP0 to 3, perhaps due to unnoticed differences in nominal (“as built”) length/diameter of these piles;
- The failure load of the root piles has marginally increased with the increase of injection pressure (comparing R0 to R3 and R5). This load has considerably increased (as far as 55%) from R0 to R2, but has dropped sharply af-

terwards, which may be indicative of an “optimal” injection pressure of 200 kPa (in terms of bearing capacity) for this type of pile and soil characteristics. A possible explanation is derived from the simultaneous (and distinct) effects of collapse and increase of lateral stress on the failure load of the soil. It is postulated that such combined factors (stress/collapse) unequally affect the capacity and the rigidity of the soil when increasing the injection pressure inside the borehole. Thus, it seems that a “threshold” pressure of around 200 kPa exists, beyond which a major structural soil breakage starts to take place. These combined effects led to the marginal increase in the failure load of the piles from R0 to R5, predominantly due to the gradual increase of the level of lateral stress with the increase of injection pressure. Besides, with the increase of injection pressure there was a marginal increase in the final (“as built”) pile diameter. It appears, however, that beyond the “threshold” 200 kPa pressure, the influence of the collapse of the soil on the failure load surpasses the influence of any of the other factors (as the increase in lateral stress and pile diameter). This happened because, as hypothesized herein, the structure of the soil surrounding the hole was somehow destroyed. With this destruction the soil/pile interface partially lost its lateral friction. It has already been experimentally shown that the generalized collapse of the soil is extremely non beneficial, since it reduces the interface friction. The pile capacity was very close from R3 to R5, probably given the generalized collapse of the soil surrounding the borehole at such high pressure stages;

- The pile with the compacted concrete SCD had the highest failure load for the Strauss type piles. It seems then that it was the “concrete driven” effect, not the use of casing during excavation, that caused a beneficial response on the bearing capacity of this type of pile. Besides, by constructing with casing (comparison SCND vs. SWCND), the capacity decreases, as most probably there is a reduction in lateral friction by the more regularly shaped shaft of the SCND pile;

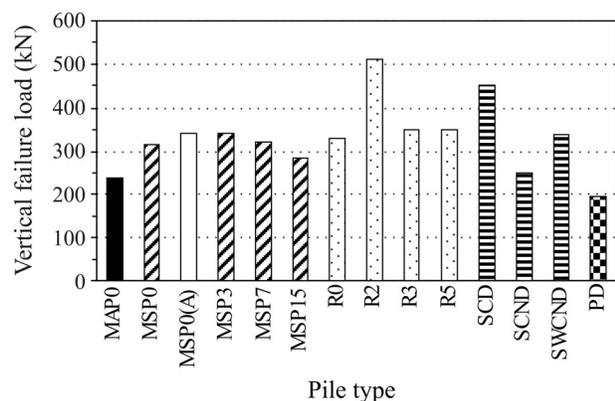


Figure 8 - Influence of construction method (after Cunha et al., 2001).

- The dynamic insertion of the precast driven pile in this type of soil considerably affected its original structure, given its fragile nature. It was noted, for all compared piles, that the driven PD pile was the one with the lowest failure load. It shall be mentioned, however, that such observation is based solely on the single test of this type carried out in the site.

#### 4.1.2. Weather seasonality effect

Mota (2003) carried out pile load tests in distinct seasons of the year, at both “wet” and “dry” periods in accordance to Fig. 4. These results are plotted in Fig. 9, where the seasonality aspect is indirectly evaluated by the comparison of bearing capacity against pile name (or weather season), using distinct estimation methodologies for pile failure values.

The tested piles have lengths of around 7.5 m as presented in Table 2. Their bases were located in the transition layer, where the  $N_{SPT}$  blow count is not high enough to turn them into end bearing type foundations. Moreover, as noticed by Mota (2003), the “active” zone of (considerable) suction variation is approximately comprised within the 3 initial meters of the soil profile, which represents almost 40% of the pile’s average length.

So, it is reasonable to suspect that the piles were indeed subjected to the influence of the weather seasonality, *i.e.*, the suction variation of the active zone. This variation in the soil’s suction must have caused a distinction in the lateral friction of the piles along the seasons of the year, thus allowing for the differences. Other causes may not be discarded, but the similar characteristics and close distance apart of the piles, as seen in Fig. 5, do point to this logical explanation.

#### 4.1.3. Bearing capacity

The estimation of the bearing capacity of bored piles is usually undertaken in Brazil by empirical ( $N_{SPT}$  based) techniques, as the recognized original methods of Aoki & Velloso (1975) and Décourt & Quaresma (1978) (*i.e.*, criteria A). These methods were then chosen to evaluate the

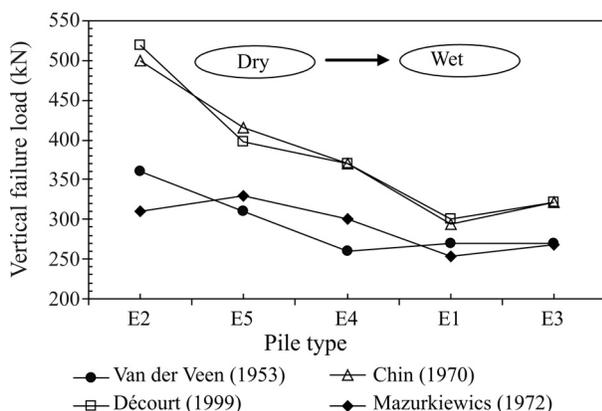


Figure 9 - Influence of weather seasonality (after Mota, 2003).

bearing capacity of the floating bored piles of Mota (2003), using the closest SPT values to each respective pile (according to Fig. 5). The methodologies were also tested with local correction factors proposed by Rodrigues *et al.* (1998) to be used in the Brasília porous clay (*i.e.*, criteria B).

Mota (2003) has compared in Fig. 10 the failure loads by each of the techniques against experimental pile load test results. The latter results come from Van der Veen (1953) estimation criteria, since, according to this author, this criteria leads to results in the conservative safe side, also close to the average values from all tested failure methods (expressed in Fig. 9).

Figure 10 allows the perception that, although the experimental data is markedly influenced by weather season effects, such trend is not found by the empirical estimations. This is so given the lack of sensitivity of the SPT test to suction variations at the Brasília porous clay, as already stated elsewhere (Cunha *et al.* 2007).

It is also noticed that the tested methods do tend, in general, to give results in the safe side. It also seems that Décourt and Quaresma (1978) method produces values that are closer to the load tests at each pile. Without local correction factors, this method estimates an average value (for all piles) lower than the average experimental data. By using correction factors, the average estimated value approaches the experimental one, but on the unsafe side.

#### 4.1.4. Average lateral friction values

All five bored piles from Mota (2003) were instrumented, but only one of them yielded results which could be interpreted, due to problems in the glue used (as usual, found after experiments). Therefore, Fig. 11 presents the lateral measured friction values of pile E1, during its loading stage till failure (according to aforementioned criteria) at 270 kN and  $\Delta_{max}$  of 16.1 mm.

As noticed, the lateral friction seems to be fully mobilized in depths 0 to 3.4 and 5.4 to 7.4 m, at the ultimate load

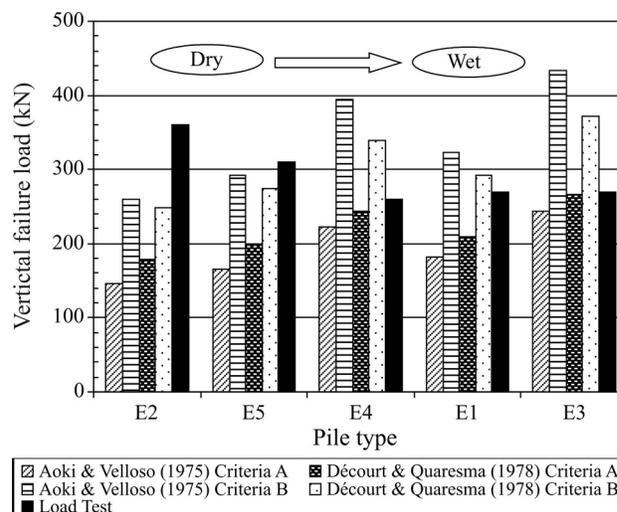
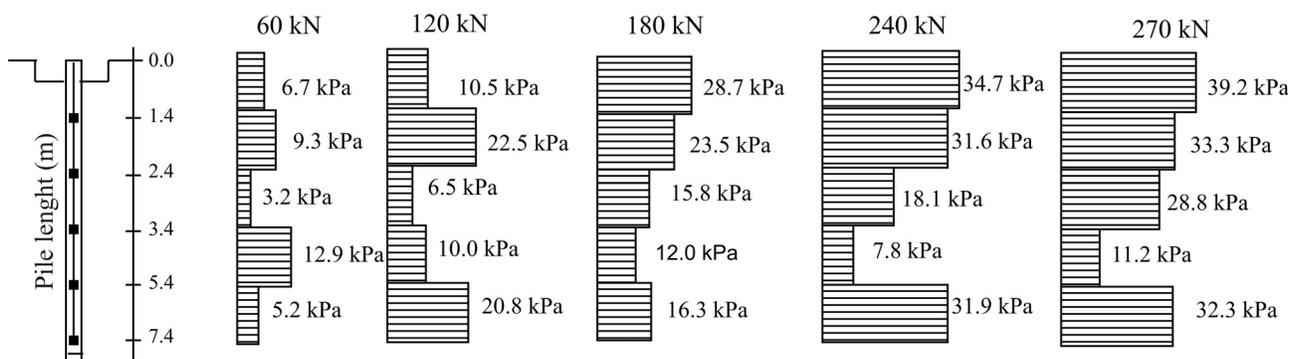


Figure 10 - SPT estimation methods (after Mota, 2003).



**Figure 11** - Results of vertical lateral friction stress during E1 load test (after Mota, 2003).

or displacement level ( $\sim 5\%$  dia.). This aspect is better depicted by comparing the results at the last two loading stages. Nevertheless, for depths 3.4 to 5.4 m the applied level of displacement wasn't enough to generate maximum friction values. Besides, for depths beyond 5.4 m, there is a clear perception that the lateral friction is of higher magnitude than values at shallower depths. By comparing that with data from Fig. 3, one also notices that, below 5 m the maximum SPT torque increases (in general) as well and the profile changes from silty sand to sandy silt, perhaps explaining the differences.

Albuquerque *et al.* (2001) observed in several load tests with bored piles with dia 0.45 m and length 12 m, at the Campinas Univ. Research Site (residual soil from diabase), that the lateral friction was fully mobilized for average pile head displacements of around 5 mm ( $\sim 1\%$  of dia.). Their ultimate friction values were in the range of 40 kPa, close to herein values.

The results of Fig. 11 do also tend to agree with numerical simulations (on distinct piles) carried out by Cunha & Kuklík (2003) in the Brasília porous clay. The values seem to be in the same magnitude of the expected (predictions) by these authors.

It is necessary to emphasize, however, that such observations are quite limited by the reduced amount of data and lack of (well instrumented in this site) experimental piles which could corroborate with the given trend.

Load cell results<sup>3</sup> at the base of this particular pile have also shown that from the load applied at its top only 0.5% effectively reached its tip at ultimate stage (in average less than 1% throughout load test). This aspect is aligned with the normal design assumption of bored piles in Brasília as behaving as fully "floating" piles. Off course, one can argued that mobilization at pile base do take place but at a rather higher level of displacement. Anyway, this wouldn't be feasible from a design point of view.

<sup>3</sup> Mota (2003) states that the load cell was probably not well aligned, or "fully" supported, at the base of the excavation. This may have some impact on the results.

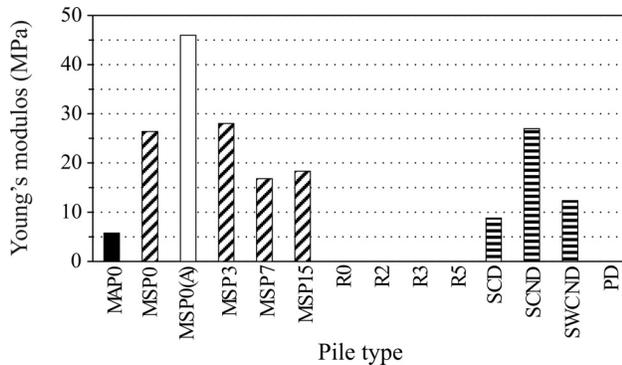
#### 4.1.5. Young's Modulus of the soil

This item has also been presented elsewhere (Cunha *et al.*, 2001) and it is included here to enhance the understanding of the key (behavioral) aspects of deep foundations founded in this particular tropical soil.

The vertical Young's Modulus of the soil surrounding each of the piles was determined with a unique point of the load-settlement field curve, *i.e.*, the point in which the load was half the value of the bearing capacity value. By using this (working) load and its associated settlement it was possible to numerically backanalyze a unique, average, Young modulus by adopting a program denominated DEFPIG (Deformation Analysis of Pile Groups, Poulos 1990). This software determines the deformations and load distribution within a group of piles and isolated piles subjected to general loading. It was specifically written for piles designed under the "conventional approach", by considering a group of identical elastic piles having axial and lateral stiffness that are constant with depth. It also allows for the eventual slippage between the piles and the surrounding soil. The stress distributions are computed from the theory of elasticity, more specifically from Mindlin's solutions for an isotropic, homogeneous, linear elastic medium.

Hence, Fig. 12 presents a plot of the Young modulus of the soil around each of the piles. In regard to this figure the some observations can be given:

- The mechanically bored pile with the expander additive (MSP0(A)) had a Young modulus of the soil around its shaft much higher than the equivalent modulus of the pile without additive (MSP). This means that this former pile has settled much less than the latter one, at similar loading conditions. This fact may be physically interpreted by a possible higher lateral pressure (than the normal MSP case) exerted by the soil around the MSP0(A) pile's shaft, rather than an eventual rearrangement of the soil structure (with



**Figure 12** - Backanalyzed Young Modulus (after Cunha *et al.*, 2001).

stiffness increase). The phenomena, however, is still subjected to other arguments;

- The manually augered pile (MAP0) had a Young modulus of the soil around its shaft much lower than the equivalent modulus of the mechanically bored pile (MSP0);

- Similar trend as before in Fig. 8 (for vertical failure) are observed for the mechanically bored piles, *i.e.*, the Young modulus of the soil around the piles decreased with the increase of the time span between excavation and casting (from MSP3 onwards). Hence the piles settled more with the increase of time span between excavation and casting;

- Some of the piles (as the root “R” and the precast driven PD piles) could not be backanalyzed by the program, since the obtained moduli were unrealistically high. This was related to the nature of these piles, rather than to the program itself. These piles had very low settlements (around  $\pm 1$  mm, at working loads), which were of the same magnitude of their (estimated) structural elastic compression. This particular feature has hampered the backanalysis,

since it was done on the basis of an assumed structural Young modulus for each of the piles. Hence, small differences in the assessment of the elastic compression of the piles (by the program) yielded large estimations on the value of the Young modulus of the soil;

- The Strauss pile with the compacted concrete SCD had the lowest Young modulus for the soil around its shaft (hence the highest settlement at working load) in comparison to the others Strauss type piles. This feature is exactly the opposite of what has been found in terms of capacity, and may be indicative of the fact that, for this type of tropical collapsible soil, the concrete “compaction effect” is beneficial solely in terms of failure load. Besides, by constructing without compacting the concrete (comparison SCND vs. SWCND), the use of casing seems to be preferable, as it considerably reduces the settlement at working loads (as one notices in this figure with the higher Young modulus). The causes for this and the former observed aspect are difficult to explain, but perhaps are intrinsically related to some features of the soil as the stress increase and relaxation, or dynamic effect during pile construction. More research is necessary to better understand this point.

## 4.2. Horizontal direction

### 4.2.1. Displacements

The main purpose of the horizontal load tests was the definition of the failure loads. Nevertheless, it is interesting to compare the results of the relative displacements (horizontal  $y_0$  value at soil/pile interface divided by the diameter) from the tests.

Table 3 presents the main characteristics observed and computed for each of the tests, for the soil at both inundated and natural water content conditions. From this table some features are found:

**Table 3** - Specific characteristics from horizontal load tests.

Pile type	Maximum load (kN)	$y_{0max}/D$ (%)	Failure load (kN)	Work load (kN)	$y_{0work}/D$ (%)
RCT1 n	75	0.7	90*	45	0.2
RCT1 i	82.5	3.1	90*	45	0.4
RCT2 n	75	1.0	90*	45	0.2
RCT2 i	82.5	2.1	90*	45	0.6
R2 n	21	1.6	30*	15	0.7
R2 i	21	2.0	30*	15	1.1
R3 i	21	4.3*	30*	15	1.8
R5 n	21	7.3*	30*	15	2.3
PD n	30	3.4	50**	25	2.6
PD i	30	4.0	50**	25	2.9

(\*) Van der Veen (1953) criteria; (\*\*) NBR 6122 (2010) criteria; (\*) structural failure (Jardim. 1998).

n = natural. i = inundated conditions. Working load for Safety Factor of 2.0.

- The maximum relative displacements attained throughout the tests were under 10%. Besides, by considering a working load of half the estimated failure value, one concludes that equivalent relative displacements at working conditions do not surpass an approximate value of 3%. Indeed, the few tests in which a structural failure of the shaft took place were pushed to displacements as high as ~ 7% of the pile diameter, which corroborates to the low magnitudes at the working conditions;

- The relative displacement of the piles at working conditions with inundated soil was, in general, much higher (double and above) of equivalent values for the soil at natural conditions. The only exception is the precast PD pile, perhaps because during its dynamic insertion it has considerably affected the original structure of the soil around its shaft (as mentioned before), hence mobilizing an annulus of soil already disturbed in any of the testing cases. This observation seems to agree with the fact that the relative displacements of the PD pile were the highest ones of this table;

- Failure loads estimated for similar piles with soil at inundated or natural water content conditions were indeed very close numerically. In part, this is related to the simplifications and adjustment problems of the extrapolation method employed by Jardim (1998) to derive the failure loads. By observing Fig. 7 one notices that physical failure was not reached by most of the tested piles (examples for R2 and PD), and indeed some sort of extrapolation criteria, or idealization, had to be used to define the ultimate value. Another reason relates the volume of soil mobilized during failure. Notice that, in this case (distinctively from the vertical direction) a large volume of soil is encompassed during horizontal compression, rather than a thin annulus at the pile/soil interface (more prone to be influenced by soil inundation). Given this aspect, the next comparison will focus solely on the results at natural water content conditions.

#### 4.2.2. Bearing capacity

The horizontal failure load was estimated by the classical theory of Broms (1964 a,b) for long, or slender, piles in which the failure takes place with a plastic hinge in the pile shaft, *i.e.*, it primarily depends on the structural yield moment of the pile itself.

The distinct graphic solutions for unrestrained piles at both cohesive and cohesionless soils were adopted, since this particular soil has cohesive-frictional characteristics. So, in order to furnish the methodology with soil resistance values, the CKOD triaxial results presented in Cunha *et al.* (1999) for soil samples at natural water content conditions were adopted. Given the fact that horizontal behavior is more dependent on superficial soil layers, only the test results for the undisturbed sample of 3 m of depth was used. This refers to a drained cohesion of 11 kPa and a drained friction angle of 27.9°.

The yield moments of the piles are those presented by Lima (2001), calculated respectively with the structural resistances of both concrete and steel reinforcement used during construction of the piles.

In order to use this methodology an assumption had to be made by Lima (2001) to employ the cohesion resistance factor within the graphical solution. As it is well known, Broms' methods are valid for cohesive (undrained) and cohesionless (drained behavior) materials. Hence, the use of the friction angle was straightforward with the graphics, but this was not so with the cohesion value. Since the solution was developed for the undrained cohesion, rather than the drained one, it was assumed by this author that one could furnish this latter value within the graphical solution to obtain the failure load caused by the cohesive part of the effective shearing resistance of the soil. This is so given the fact that the soil at the experimental site does not behave in an undrained mode, as there is no water level there.

With aforementioned simplification, open to criticism, the method was tested against the experimental failure load results expressed in Table 3. Figure 13 shows the comparison using each adopted parcel of the soil's resistance.

As clearly noticed, both ways of calculating do lead to reasonably close results, being therefore sufficiently acceptable for practical use. The larger differences between experimental to estimated values relate to the precast driven pile, perhaps, again, because the soil is more disturbed around this pile (compared to other foundations) as commented before. If this is the case, it certainly relates to a soil characteristic more distinct to the undisturbed material tested at the triaxial tests. Nevertheless other aspects can also be raised to explain the differences, as aforementioned questions related to the extrapolation of the failure load.

#### 4.2.3. Coefficient of subgrade reaction

The use of the "beam on elastic foundation" theory for horizontally loaded pile problems requires the specification of a soil modulus which represents the linear, or proportional, relationship between the horizontal pile

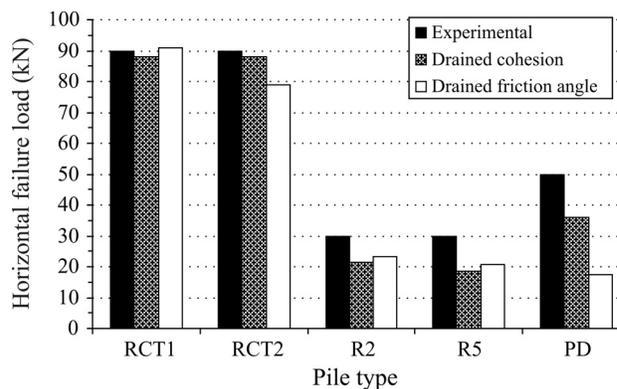


Figure 13 - Horizontal failure load – soil at natural conditions (modified after Lima, 2001).

displacement and the respective soil reaction. This modulus is defined for each distinct section of the pile along its depth, and is termed the “modulus of subgrade reaction” of the soil ( $K$ ). It can be then used to simulate “Winkler” springs during the analysis of laterally loaded piles, as presented by Reese and Matlock (1956) in their classical paper.

The  $K$  coefficient is related to the total width of the pile’s shaft, and has a dimension  $FL^{-2}$  ( $kN/m^2$ ). If we introduce the lateral subgrade reaction modulus  $K_h$  for a pile of unit width, we obtain:

$$K = K_h \times D \quad (1)$$

where  $D$  is the diameter of the pile and  $K_h$  has a  $FL^{-3}$  ( $kN/m^3$ ) dimension.

The subgrade reaction moduli ( $K$  and  $K_h$ ) have distinct values, or variation, for distinct soil types, and hence, two different cases can be considered. The first case assumes  $K$  constant with depth, and the second case assumes a linear variation of  $K$  with depth. The latter according to the following equation:

$$K = \eta_h \times \text{depth} \quad (2)$$

where  $\eta_h$  represents the rate of increase of the subgrade reaction modulus, or the “coefficient of horizontal subgrade reaction” of the soil, in units of  $FL^{-3}$  ( $kN/m^3$ ).

In general, for sandy soils and for soft clays the subgrade reaction modulus increases linearly with depth. This idealized hypothesis is in accordance with the (drained) characteristics of the tropical unsaturated soil deposit of the experimental site. Therefore, only the coefficient of subgrade reaction modulus was backcalculated here.

The backanalysis was, however, simplified by assuming a constant structural Young’s modulus of the pile during the loading process (25 GPa for root piles and 20 GPa for all others). Thus, it does not follow the more advanced analytical technique put forward by Reese *et al.* (1998), by not taking into account the (unknown) variable stiffness of the piles.

In order to obtain  $\eta_h$  it was necessary to use the relationship between the horizontal applied load and the pile/soil displacement at the soil surface ( $y_0$ ) as given by Matlock & Reese (1961):

$$y_0 = 2.435 \times H \times \frac{T^3}{EI} \quad (3)$$

$$T = \sqrt[5]{\frac{EI}{\eta_h}} \quad (4)$$

where  $E$  is the structural Young’s modulus of the pile;  $I$  = structural moment of inertia and  $H$  = horizontal load.

However, in most of the cases the horizontal load is not applied at the pile/soil interface, but at some other point on

the pile. It will then generate a displacement  $y_i$  at the pile head that can be calculated by Kocsis (1971) equations. These equations relate the displacement at any level of the pile head above the ground (as measured during load tests) to the displacement at the pile/soil interface  $y_0$ , taking on account the pile head rotation (function of  $T$ ,  $EI$  and  $H$ ), the horizontal load ( $H$ ), the pile characteristics ( $I$  and  $E$ ), and  $\eta_h$ .

Therefore, in order to obtain the backanalyzed coefficients at each (calculated and plotted as in Fig. 7)  $y_0$  displacement level a spreadsheet was developed to interactively solve the general formula for each experimental pair of known values of top head displacement and horizontal load (details in Jardim, 1998).

The results for the (reaction) bored, the root and the (precast) driven concrete piles, with the soil at both natural water content and inundated conditions, are shown in Fig. 14. The moduli were backanalyzed up to the maximum displacement values of the load tests, as expressed in Table 3.

From this figure it is noticed that:

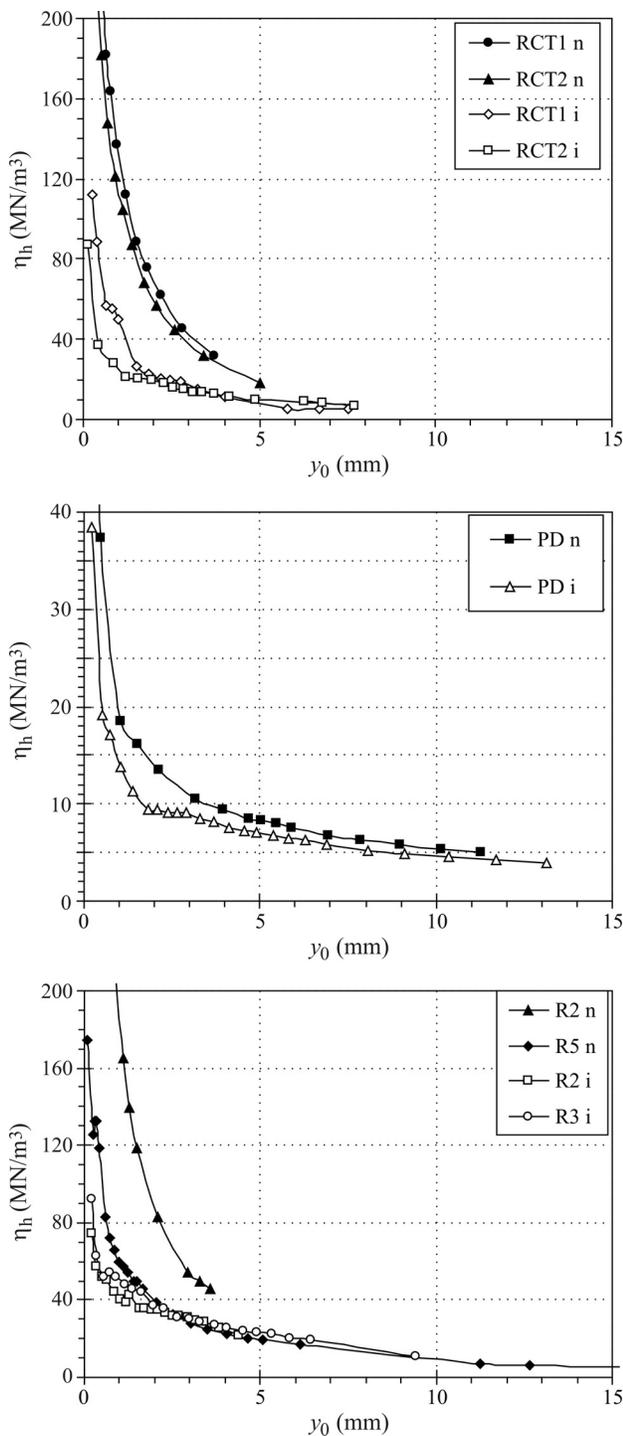
- As expected, the moduli have considerably decreased at an asymptotic rate with the increase of the displacement level. Besides, when comparing R2 and R5 at natural soil conditions, one also notices that the moduli decreased with the increase of the injection pressure beyond 200 kPa, perhaps related to aforementioned structural aspects of this soil;

- Based on the previous statement, Cintra (1981) has respectively suggested  $y_0$  design intervals of 4-8 mm for the soil at natural conditions, and 12-18 mm for the inundated case. Nevertheless, based on the working load levels of herein cases, Jardim (1998) suggested the use of the design intervals 4-10 and 6-12 mm to respectively represent the soil at the natural water content and at inundated conditions. These intervals do already encompass the working displacement levels of the piles depicted in Table 3;

- Based on the aforementioned intervals it was possible to obtain average backanalyzed moduli for practical use, as presented in Table 4. It shall be noticed that the injection (R2n) pile case was discarded in the averaging, given the very distinct result of this load test in comparison to others of this same pile type. This table do serve, therefore, as a start point for designing similar foundations on this same soil, when using the described theoretical methodology of this item;

**Table 4** - Suggested avg. reaction moduli at working loads (after Jardim 1998).

Pile type	$\eta_h$ (MN/m <sup>3</sup> )	
	Natural conditions	Inundated conditions
Bored	16	7
Root	19.5	14
Precast	7	5.5



**Figure 14** - Backanalyzed coeff. of subgrade reaction (after Jardim 1998).

- The moduli for the inundated condition are lower than those from the soil at natural conditions, at same pile and displacement levels. This reflects the higher displacements attained at such former test conditions, as noticed before. An average decrease of around 50, 30 and 20% was noticed in relation to the moduli at natural soil conditions, respectively for the bored, the root and the precast pile type;

- The backanalyzed moduli from the driven precast pile were the lowest from all load tests, again reflecting the fact that the dynamic insertion considerably affected the soil's original structure, given its fragile nature.

### 5. Conclusions

This paper emphasized the main results obtained from load tests on several large scale deep foundations located at the Experimental Research Site of the Geotechnical Graduation Program of the University of Brasília. Typical foundations adopted in this city, and the Federal District as well, were vertically and horizontally loaded, yielding loading displacement curves that were interpreted according to recognized empirical and theoretical methods from the soil mechanics.

Experimental data acquired from the load tests, as the displacement levels at failure and working conditions, or the lateral friction mobilized at the pile shaft, were also presented and discussed. Empirical or theoretical methods currently employed to respectively derive the ultimate vertical and horizontal capacity of loaded piles have been explored together with backanalyzed elastic moduli, also for both considered directions.

The analyses have also allowed a reasonable insight into some of the most relevant variables that affect the behavior of the deep foundations once vertically or horizontally loaded on tropical soils. The influences of several external aspects, as the construction methodology or the weather seasonality, have been addressed in the paper, but in a limited manner. Although some of the results come from research theses which have been finished more than a decade ago, they have been assembled for the first time in a comprehensive manner, allowing a perspective of some of the key aspects when designing pile foundations on the collapsible tropical soil of Brasília.

Although the results are restricted to the conditions of the analyses, based on a limited set of data, they allow preliminary generalizations of the overall behavior. Moreover, they do highlight the fact that the phenomena involved with such processes are rather complex. In this regard, this paper has provided a better understanding of some of the features which are involved by the loading mechanisms of isolated piles on tropical soils. It shall be noticed, however, that some comments have been hypothesized in order to explain the results, and do need further research for a more grounded appreciation in the future.

Therefore, from the trends observed with the data and analyses, some general conclusions can be drawn:

- Manually augered piles are not recommended as foundation solution to replace mechanically excavated ones, with exception to perhaps low level constructions or lightly loaded structures;
- The use of an expander additive mixed in the fresh concrete increases the vertical failure load and decreases

the vertical settlement of bored piles, and shall be adopted whenever possible;

- Bored piles should be preferably cast-in-place between 0 to a maximum of 3 days after the soil excavation, since there is a tendency of decrease of vertical bearing capacity, and of increase on the vertical settlement, with increasing time spans between hole excavation and concreting after the 3<sup>rd</sup>. day;

- The vertical bearing capacity of root piles does increase with the level of pressure, but up to a value of around 200 kPa. This pressure appears to be a “threshold” value beyond which a major structural breakage, or collapse, takes place in the soil surrounding the borehole. Hence, such pile types should be limited to low injection pressures, specially on the collapsible layers of this deposit, close to surface;

- The dynamic insertion of some pile types, as the precast driven, should be avoided in this type of material, given its fragile nature at the collapsible layers. This effect influences the results of both the vertical and the horizontal capacity values;

- Suction effects, or weather seasonality, do influence the vertical capacity of floating bored piles founded in this soil, and should be taken into account specially for short length piles;

- The traditional Décourt and Quaresma (1978) method can in principle be safely used in design for vertically loaded bored piles, with  $N_{spt}$  results at any time of the year;

- Bored piles do seem to behave as floating ones in this particular deposit, with very low vertical bearing capacity values at the base. Perhaps the base pressure could be more effectively mobilized at higher displacement levels, *i.e.*, in a range above the usual admissible values. Ultimate lateral shaft values below 40 kPa mobilized at vertical head displacements of around 5% of the pile diameter can be used as a design starting value - to be further verified *in situ* given the (already cited) constraints of herein data;

- Horizontal displacements attained on loaded bored piles increase at both working and failure conditions when the soil is inundated. As noticed for the root piles, the injection pressure beyond a certain level also increases this displacement;

- The bearing capacity estimated for bored, root or precast driven piles horizontally loaded in this soil, via the traditional extrapolation or the norm criteria, do not change considerably with subsoil conditions (*i.e.* inundation or not);

- The traditional Broms (1964a,b) method can in principle be safely used in design for horizontally loaded bored, root and precast driven piles in this soil. Nevertheless, in the case of the precast driven, the methodology seems to be very conservative;

- The coefficient of subgrade reaction decreases with the level of horizontal displacement of the pile. Hence, to be suitable for practical purposes, this modulus should be

chosen at the corresponding design range that the pile is expected to displace in its life;

- Similar to the vertical case, the dynamic insertion of some pile types, as the precast driven, should be avoided in this soil. This technique has caused the driven pile to have the lowest values of horizontal subgrade reaction at the working loads.

This paper is a collection of the contribution of many theses from the Geotechnical Graduation Program of the University of Brasília. Given the small number of foundations, the limited spatial size of the studied area within the geographical context of the Federal District, and the multitude of external factors that could affect the results, it is evident that more studies are still necessary (and are underway).

Therefore, it shall be emphasized that the conclusions drawn herein have to be considered of limited range and applicability. Nevertheless, these results, together with the experience acquired during the exercise, can be of high interest for researchers and foundation designers of this region and abroad. In many aspects, the presented data of this paper can be readily used in practical design of equivalent foundations on similar soil deposits as the one studied herein, or, at least, be used as a start point for the project in “non classical” tropical soil types.

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