Uplift behavior of bored piles in tropical unsaturated sandy soil

Comportement en traction de pieux forés en sol tropical sablonneux non saturé

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ABSTRACT: Large deposits of high-porosity, unsaturated, sandy soils are found in the southern central region of Brazil, to several meters of depth. This region regularly sees increases in uplift loads in structures such as wind energy and power transmission towers. For the current study uplift load tests were conducted on three bored piles,10m depth and with respective diameters of 0.35m, 0.4m and 0.5m. The piles were instrumented with strain gauges connected in full bridge at five depth levels. The load vs. displacement and the load transfer curves along the depth were obtained. The data obtained from the field tests (CPT, SPT) and laboratory tests (c, ϕ, γ_n) were used to predict the ultimate loads of the piles. The calculated ultimate load values were compared with those obtained in the piles load tests. The tests were carried out on the campus at the University of São Paulo, in the city of São Carlos. The average composition of the subsoil is sand (62%), clay (22%) and silt (16%), with an average porosity of 45%. The field tests produced average values of N_{SPT}=5, qc=1700kPa and fs=98kPa, respectively. The water table was encountered at a depth of 10m.

RÉSUMÉ : Dans la région centrale sud du Brésil on trouve de grands dépôts de sols sablonneux, non-saturés, jusqu'à plusieurs mètres de profondeur. L'augmentation des tensions verticales de soutien de charge est fréquente dans cette région pour des structures comme des tours d'éoliennes et des lignes de transmission d'énergie. Pour cette recherche ont été réalisés des essais d'arrachage sur trois pieux forés à 10m de profondeur et avec des diamètres de 0,35m, 0,40m et 0,50m. Les pieux ont été instrumentés avec des jauges de déformation connectées en ponts complets à cinq niveaux le long de l'encastrement. On a obtenu des courbes charge/déplacement et le transfert de charge le long de l'encastrement. Les données des essais in situ (CPT ,SPT) et des essais en laboratoires (c, ϕ, γ_n) ont été employées pour prévoir les charges ultimes des pieux. Les valeurs calculées ont été comparées avec les valeurs obtenues dans les essais de charge. Les essais ont été réalisés dans le Campus da Université de São Paulo, dans la ville de São Carlos. La composition moyenne du sous-sol est sable (62%), argile (22%) et limon (16%), avec porosité moyenne de 45%. Les essais in situ ont indiqué des valeurs moyennes de N_{SPT}=5, qc=1700kPa et fs=98kPa. Le niveau d'eau a été trouvé à 10m de profondeur.

KEYWORDS: uplift load test; unsaturated sandy soil; bored pile; instrumented pile.

1 INTRODUCTION

The southern central region of Brazil possesses vast areas of sandy soil where the water table is often deeper than 10 meters. This unsaturated soil condition, combined with the lateritic soil, enables the use of bored piles, with no need to line the boreholes in the vast majority of constructions. Where these foundations are subjected to uplift forces, there is some debate as to which method to use to calculate the ultimate load. Consequently, this study, by way of the performance of three uplift load tests on instrumented bored piles, analyzes the mechanism for transferring load to the subsoil as well as ascertaining the applicability of the methods for calculating ultimate load that are available within the technical milieu. This study was conducted on the campus of the University of São Paulo, located in the city of São Carlos, in the state of São Paulo, Brazil. Geographically, its location is defined by the coordinates 22º 01' 22" South and 47º 53' 38" West.

2 GEOTECHNICAL CHARACTERISTICS

The region sits on rocks of the São Bento Group, composed of sandstone of the Botucatu and Pirambóia formations and by ridges of basalt rocks of the Serra Geral formation. On top of these rocks appears sandstone of the Bauru Group, followed by Cenozoic sediment.

Close to the piles used in the study, five CPT tests and five SPT tests were carried out. Laboratory testing was carried out on both disturbed and undisturbed soil samples collected via an open hole down to the level of the water table, situated at a depth of 10 meters.

The tests characterized the subsoil down to a depth of 10 meters as being made up of two layers, primarily composed of fine lateritic sand separated by a line of boulders at a depth of 6.5 m. The soil in the first layer is characterized as Cenozoic sediment and in the second layer as sandstone of the Bauru Group. Table 1 shows the average values for the geotechnical parameters of the subsoil in the chosen location.

Table 1. Average geotechnical parameters

Depth	**1 m – 6 m**	**6 m – 12m**
Formation	**Sediment**	**Residual**
γ_n (kN/m^3)	16.3	18.9
w (%)	16	16
Gs	2.73	2.76
e	0.94	0.71
n (%)	48	42
Sr (%)	47	62
LL (%)	28	32
IP (%)	11	17
Sand (%)	62	61
Silt (%)	13	10
Clay (%)	25	19
c' (kPa)	6	20
ϕ' (º)	30	23
SPT - N_{72}	4	7
CPT – qc (MPa)	1.07	2.36
CPT – fs (kPa)	45	150

N.B. γ_n (bulk specific weight), w (water content), Gs (grain density), e (avoid ratio), n (porosity), Sr (degree of saturation), c' (cohesion – CD triaxial); ϕ' (friction angle – CD triaxial)

4 PILES AND INSTRUMENTATION

Three 10m long auger bored piles were inserted with respective diameters of 0.35m, 0.4m and 0.5m. The boring was carried out without using water. The concreting was carried out immediately following the opening of the holes. The simple compression resistance of the concrete used was 26MPa at 28 days.

The reinforcement of the piles consisted of two 20mm and two 32mm corrugated steel bars, both 10m in length.

The instruments consisted of strain gauges at five levels along the depth (0.6m – reference section; 3.1m; 5.3m; 7.5m; 9.7m). The strain gauges were connected in full bridge to 20mm steel bars, 0.6m long, which were screwed to the reinforcement. At each level along the depth, two diametrically opposed, instrumented bars were installed (Figure 1).

Tell tales were also installed on the piles but these did not give precise readings for analysis due to the low displacement values measured.

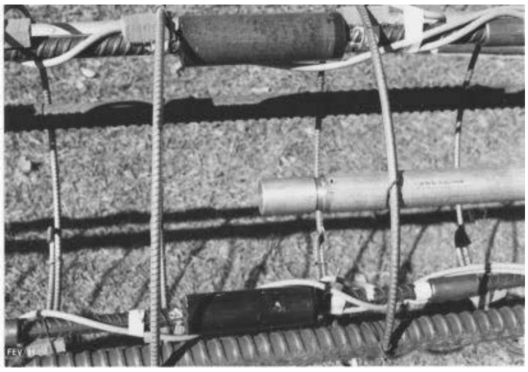

Figure 1. Instrumentation using strain gauges and tell tales.

3 LOAD TESTS

The load tests were of the slow maintained load type, with successive 40kN loads, observing the principles of Brazilian standard NBR 12131. The reaction system comprised root piles 16m long with a diameter of 0.25m, for a working load of 500kN, and four I-shaped steel beams with a load capacity of 2MN. Loads were measured using load cell with a capacity of 1MN and displacement was measured using four dial gauges to within an accuracy of 0.01mm. The ultimate loads (Q_{ult}) obtained were 387kN, 440kN and 478kN respectively. The load vs. displacement curves obtained are shown in Figure 2.

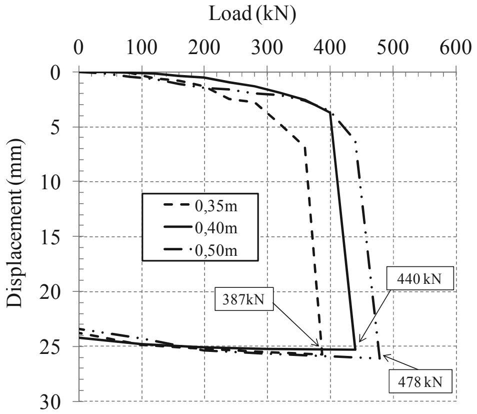

Figure 2. Load vs. displacement curves.

Figure 3 shows, for the ultimate loads and working loads, (Qult/2) the lateral friction for each pile segment, recognizing a rupture in the pile-soil interaction.

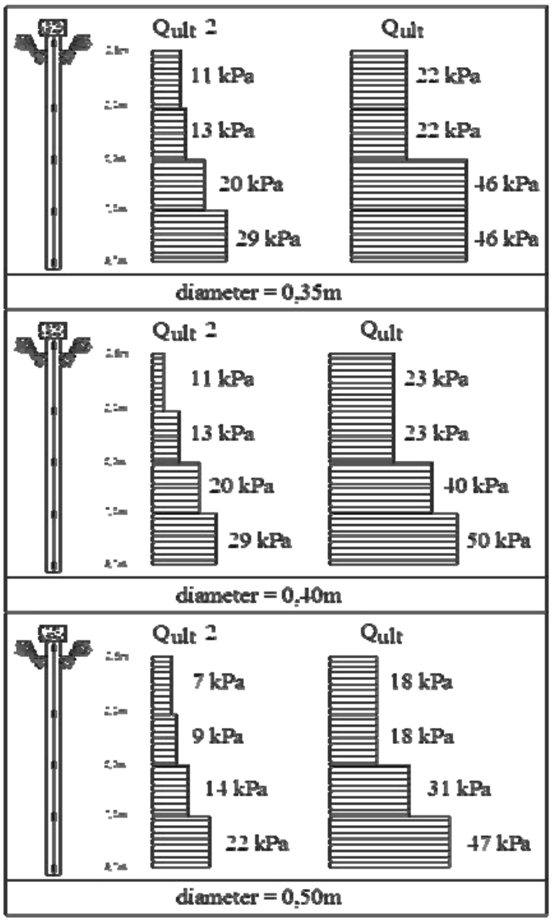

Figure 3. Lateral Friction Distribution Graph.

Figure 4 shows the average lateral friction curves resulting from shaft displacement. Tables 2, 3 and 4 show the load values at depth for all piles.

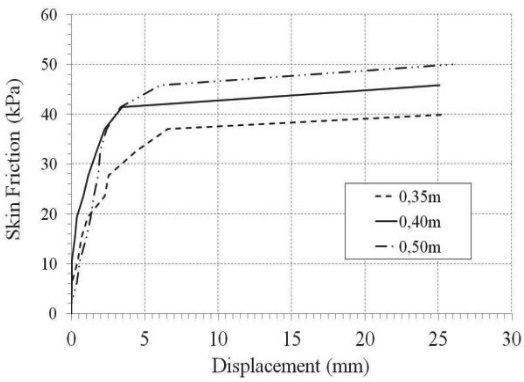

Figure 4. Average skin friction x shaft displacement curves.

Table 2. Load at depth for the 0.35m pile.

Load at top (kN)	Load at respective levels (kN)			
	3.1m	5.3m	7.5m	9.7m
40	35	29	27	21
80	76	47	41	23
120	107	90	70	23
160	146	119	80	21
200	171	140	91	23

240	210	167	103	25
280	253	196	115	27
320	280	222	126	23
360	315	253	154	23
387	-	272	161	23

Table 3. Load at depth for the 0.4m pile.

Load at top (kN)	Load at respective levels (kN)			
	3.1m	5.3m	7.5m	9.7m
40	37	35	32	1
80	64	56	46	3
120	118	104	75	11
160	150	126	83	11
200	187	158	110	11
240	217	185	123	11
280	249	211	136	11
320	284	233	144	11
360	318	265	163	11
400	340	278	166	8
440	-	302	190	8

Table 4. Load at depth for the 0.5m pile

Load at top (kN)	Load at respective levels (kN)			
	3.1m	5.3m	7.5m	9.7m
40	36	32	32	4
80	79	68	65	7
120	119	111	93	22
160	158	133	101	18
200	183	158	120	22
240	212	180	133	40
280	248	208	151	29
320	287	241	162	29
360	320	269	176	29
400	348	295	194	40
440	377	320	216	18
478	-	345	233	18

5 ANALYSIS

The load tests show, for the ultimate load, an average skin friction (sf) for the three piles of 21kPa (sf_1) for the Cenozoic sediment layer and 45kPa (sf_2) for the residual soil layer.

From Figure 4 it can be seen that the almost total mobilization of skin friction is for 5mm displacement.. Fellenius (2012) showed that, to mobilize the ultimate pile shaft resistance requires very small relative movement between the pile and the soil, usually only a few millimeters in inorganic soils and that the direction of the movement has no effect on the load-movement for the shaft resistance. That is, push or pull, positive or negative, the maximum shear stress is the same. Moreover, the movement necessary for full mobilization of the shaft resistance is independent of the diameter of the pile.

In analyses using semi-empirical formulae, the rupture in the pile-soil contact area was assumed. As the soil being studied was soft sand and the piles were relatively short, the tensile skin friction was assumed to be equal to the compression skin friction. Poulos (2011) states that for piles in medium dense to dense sands, this ratio typically ranges between 0.7 and 0.9, but tends towards unity for relatively short piles, and that a significant advance in the understanding of this problem was made by Nicola and Randolph (1993).

Table 5 shows the results obtained in the load tests ($P_{Ult.PC}$) compared to those obtained from the methods employed to predict the ultimate load of the piles ($P_{Ult.Cal.}$).

Table 5. Ratio of the ultimate load value obtained in the load test to the calculated value

Ratio $P_{Ult.PC}$ / $P_{Ult.Cal}$	0.35m	0.4m	0.5m
Meyerhof and Adams (1968)	2.03	1.79	1.26
Meyerhof (1973)	0.71	0.66	0.51
Das (1983)	1.21	1.00	0.85
Martin (1966) - Univ. Grenoble	0.82	0.81	0.68
LCPC (Fellenius, 2012)	1.21	1.19	1.00
Aoki and Velloso (1975) SPT	2.16	2.13	1.79
Aoki and Velloso (1975) CPT	2.30	2.29	1.99
Decourt (1996)	1.08	1.06	0.89
Philiponnat (1978)	1.44	1.14	0.96

For the two soil layers, the average skin friction ratios for the piles based on the results of the CPT tests are sf_1=0.5.fs_1, sf_2=0.3.fs_2, sf_1=0.02.qc_1, sf_2=0.02.qc_2, respectively. By expressing the ratios sf = k.fs and sf = C.qc, the values for k demonstrated by Slami and Fellenius (1977) range from 0.8 to 2 while those for C range from 0.008 to 0.018 for sandy soils. Bustamante and Gianeselli (1982) present the C coefficient ranging from 0.005 to 0.03, as governed by the magnitude of the cone resistance, type of soil and type of pile.

The LCPC Method (in Fellenius 2012), based on the experimental work of Bustamante and Gianeselli (1982), establishes that sf=C.qc, for bored piles in sand and with a qc of less than 5MPa, the value of C is equal to 1/60. Given these values, sf_1=19kPa and sf_2=39kPa can be computed, values which are close to those obtained in the load tests, namely 21kPa and 45kPa, respectively.

Using the method espoused by Décourt (1996), which uses SPT test data, tensile ultimate load values were calculated for the three piles. According to the current suggestion of the author, it is also necessary to use a correction coefficient (β_L) due to the soil being lateritic. In this case, β_L=1.2 was used, giving rise to the results presented in Table 3.

The method proposed by Martin (1966) and developed at the University of Grenoble, includes various important aspects such as cohesion, angle of friction, overload, specific soil mass and the weight of the foundations themselves. Moreover, it is recognized that the rupture surface forms an angle λ at the base of the pile. In the calculations performed, the hypothesis of angle λ equal to zero was the one which most closely approximated the load test results.

The method proposed by Meyerhof (1973) considers adhesion, pile-soil angle of friction, effective vertical stress and a pull-out coefficient that depends on the angle of friction of the soil and the type of pile. The method employed by Das (1983) was developed for sandy soils and includes the pile-soil angle of friction and a pull-out coefficient which depend on the relative density of the sand, the pile-soil angle of friction and the soil's angle of friction. The problem with these two methods lies in the correct definition of the abovementioned parameters.

In order to predict the ultimate loads from the load vs. displacement curves of the load tests, the method employed was that proposed by Décourt (1999) based on the stiffness concept (Fellenius 2012), which divides each load with its corresponding movement and plots the resulting value against the applied load, Figure 5. Ultimate load prediction simulations were performed, without using all of the load vs. displacement curve data and it was found that, starting from 70% of the maximum load in the test, the method presents good results in terms of determining the ultimate load.

The average tensile skin-friction values found are close to the values found in compression load tests with the same type of pile in the same soil.

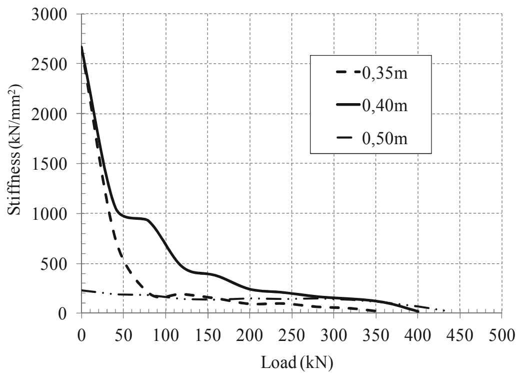

Figure 5. Stiffness curves

6 CONCLUSIONS

- The determination of the ultimate load using CPT demonstrates good results when the LCPC Method is used.
- The determination of the ultimate load using SPT demonstrates good results when using the Décourt (1996) method, assuming sandy soil and the adoption of an correction parameter (β_L = 1.2) due to the lateritic soil in the location;
- The Das (1983) method provides good results in determining ultimate load, however its use depends on the pile-soil angle of friction and a pull-out coefficient, parameters which are not always available for the particular soil being analyzed;
- The best prediction of ultimate load using the Martin (1966, 1973) method is obtained using a rupture surface inclination angle equal to zero, indicating rupture in the pile-soil interaction;
- The load transfer function presents good definition for unit skin-friction, finding that displacements in the order of 5mm were required for their almost total mobilization;
- The Décourt (1999) stiffness method demonstrated good results in the prediction of ultimate load based on the analysis of the load vs. displacement curve.

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