Analysis of uplift loads of precast-concrete piles in porous soils

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This paper presents the analysis of uplift load tests in three precast-concrete piles carried out in a collapsible sandy soil. The piles with 12 meters (m) length and 0.17 x 0.17 square meter (m^2) cross section were instrumented with strain gauges, in order to know the load transfer in depth. Three tests performed in a slow maintained load way were conducted in a natural condition of moisture content soil. A fourth test was carried out after the previous soaking of the soil around the pile head. The tests were performed in the experimental research site at the Universidade Estadual Paulista "Júlio de Mesquita Filho" (Unesp). The results obtained were evaluated by analytical and empirical methods.

Key words: Collapses soils. Piles. Uplift loads tests.

1 Introduction

Colluvial collapsible sand porous soils occur in west center region of Brazil, representing about 5% of the area of whole country. Their early origin is in Bauru (SP) sandstone, sedimentary rock of Mesozoic age that covers all region investigated (city of Ilha Solteira (SP), located north-west of state), in basalt of Serra Geral Formation. In many places these porous colluvial soils reach he thickness of 15 meters (m).

The soil of the place has high porosity and collapsible characteristics up to 8 m of depth. It is quite often the occurrence of sandy soils like that in the center-south of Brazil. The soil of the site was characterized through field tests (standard penetration tests [SPT] and cone penetration tests [CPT]) and laboratory tests.

Precast-concrete piles were built specially to that research. They have special elements (tie-rod and iron hose) placed in the inside, all along length. In every pile, instrumentation was installed at several depth levels to obtain the results of load transfer along the shafts. The piles were driven in sand soil with high porosity and collapsible characteristics, in the city of Ilha Solteira. Vertical slow maintained load tests were conducted in three piles subject to uplift forces.

Due to the very low bearing capacity for the driven piles obtained in the load tests, three bored piles were built and submitted to load tests in the same place, for purposes of comparison. The results presented an uplift bearing capacity four times higher for the bored piles, indicating a characteristic of skin friction loss for driven piles in this soil, probably during the driven process.

2 Penetration tests and laboratory tests

In the area of load tests, five SPTs and five CPTs were made. In these tests, the resistance provided by the soil to the vertical penetration of a Dutch cone was obtained. The external and internal diameters of the cone were 35.7 and 16 millimeters (mm), respectively. The base area was 10 square centimeters (cm^2). Table 1 shows the results of the penetration tests.

Table 1: Results of the penetration tests

Depth (m)	NSPT	Torque (Nm)	Qc (MPa)	Fs (kPa)
1	6	32	4.7	60.7
2	2	22	1.7	40.1
3	2	13	1.9	40.6
5	4	31	3.4	66.3
7	6	31	4.5	99.6
9	7	45	4.8	114
11	9	78	6.8	228.4
13	10	82	6.7	314.5
15	10	54	6.9	269.1

Obs.: Point resistance (Qc) = megapascal (MPa); skin friction (Fs) = kilopascal (kPa); number of standard penetration test (NSPT); torque = newton meter (Nm).

Source: Menezes (1997).

Table 2: General characteristics of the soil

Depth (m)	e	ρ (kN/m^3)	c (kPa)	ϕ (°)
1	0.85	16	0	32.2
2	0.93	14.8	3	31.8
3	10	14.9	2	32.5
5	0.9	14.8	2	33.3
7	0.81	15.9	3	33
9	0.74	18.4	16	30.3
11	0.7	17.7	20	28.8
13	0.7	18.8	20	28.8
15	0.77	17	17	30.1

Obs.: void ratio (e); soil specific weight (ρ) = kilonewton per cubic meter (kN/m^3), soi l cohesion (c), soil friction angle (ϕ) = degrees (°).

Source: Menezes (1997).

The laboratory tests were performed on the samples extracted meter by meter from the pit shaft. The consolidation tests, unconfined compression and consolidated undrained triaxial tests, were performed on undisturbed samples (Table 2).

3 Material and methods

According to following enumerations.

3.1 Characteristics and instrumentation of the piles

The piles had been manufactured and instrumented, being driving rig and presenting the following characteristic materials: steel with fyk = 1,500 MPa and concrete with fck ≥ 35 MPa, having section of 0.17 x 0.17 square meter (m^2) and length of 13 m (Table 3).

Table 3: Characteristics of the piles

Pile	Section (m)	Structural load (kN)	weight (kg/m)	Area of the section (cm^2)
Uplift	0.17	400	73	0.029
Reaction	0.23	820	138	0.053

Source: Menezes (1997).

The static instrumentation was constituted by bars of steel where were put strain gages that were concreted inside the piles in five levels. One level of strain gauges was used, placed out of the soil influence, at the head of the piles, in order to allow the determination of the value of Young's module.

For the law of Hooke ($F = E_E.\epsilon.A$) considered the module of elasticity (E_E) and the area of the section (A) constant for all the length of the pile, stayed like variables the force (F) and the specific deformation of the soil (ϵ).

The value of EE for the first level was equal the 38 gigapascal (GPa) and below the second level of instrumentation equal 30 GPa.

3.2 Load tests

Slow maintained load tests were performed according Brazilian standard NBR-12131/97 (ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS, 1997).

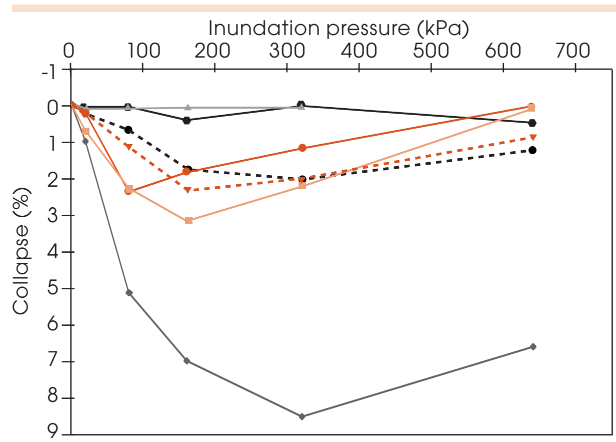
The system of reaction of the piles was constituted by a metallic beam of 7 m of length and weight of 25 kN capable to support loads of up to 1 meganewton (MN). Tie-rod had been installed in the piles of uplift and reaction for permit the test of load to the traction. It was utilized a hydraulic system (with manometer and hydraulic bomb), that it possessed a central hole for passage of the tie-rod in his interior and being able to be applied loads of up to 1 MN.

4. Results obtained

According to following enumerations.

4.1 collapse index

The laboratory tests conducted with inundation in some pressures, have shown that the soil is collapsible up to 7 m of depth (Graphic 1).



Graphic 1: Curves of collapse of the soil for some depths

Source: Menezes (1997).

4.2 Bearing capacity: methods analytical

For the estimate bearing capacity of the pile using the analytical methods, the parameters of compactness and resistance of the soil (cohesion and angle of friction) gotten in the tests in laboratory had been considered (MENEZES, 1997). In order to obtain and compare results several methods were applied, as briefly described in the following itens.

4.2.1 Method of Das (1983)

Ultimate load lateral

$$P_{LU} = [(\pi/2)d\rho(z_{CRITIC})^2K_U tg\delta] + [\pi d\rho z_{CRITIC}K_U tg\delta(z - z_{CRITIC}/2)]$$

$$(z/d) = (11.4/0.24) = 47.5$$

where: $D_R \geq 70\% \rightarrow$
$(z/d)_{CRITIC} = 0.156 \times D_R + 3.58$; D_R = 35% (sand low compact)

$$(z/d)_{CRITIC} = 0.156 \times 35 + 3.58 \rightarrow$$
$$(z/d)_{CRITIC} = 9 \rightarrow$$
$$z_{CRITIC} = 9 \times 0.24 \rightarrow$$
$$z_{CRITIC} = 2.2 \text{ m}$$

$$(z/d) = 47.5 > (z/d)_{CRITIC} = 9;$$
$K_U = 1.7$ (CARVALHO, 1991);
$$\delta = 28° \ (\phi = 32°)$$

where: $D_R = 35\% \rightarrow$
$(\delta/\phi) = 0.7$ (CARVALHO, 1991) $\rightarrow$
$\delta = 0.7 \times 32° \rightarrow \delta = 22.4°$

$$P_{LU} = [(\pi/2)0.24 \times 17.5(2.2)^2 \times 1.7tg22.4°] + [\pi 0.24 \times 17.5 \times 2.2 \times 1.7tg22.4°(11.4 - 2/2)]$$

$$P_{LU} = 22 + 211 \rightarrow P_{LU} = 233 \text{ kN}$$

Ultimate load total

$$P_{UT} = P_{LU} + \text{weight of the pile}$$
$$P_{UT} = 233 + 12$$
$$P_{UT} = 245 \text{ kN}$$

4.2.2 Method of Grenoble (*)

*This method does not present a year of formulation by to have been based in diverse authors (RIBIER, 1962; MARTIN, 1963; 1966).

Ultimate load lateral

$$P_{LU} = P_{F\phi} + P_\gamma + P_{FC} + P_{FQ}$$

where: $P_{F\phi}$ = friction parcel;
P_γ = gravity parcel (only for $\lambda > 0$);
P_{FC} = cohesion parcel = 0 (ORLANDO, 1985);
P_{FQ} = overload parcel = 0.

Determination of $P_{F\phi}$

$$P_{F\phi} = A_L\rho zM\phi;$$
$$M\phi = M\phi_o[(1 - 0.333)tg\lambda(z/r)];$$
$$M\phi_o = [sen2(\phi+\lambda)]/4cos^2\lambda$$

where: $\lambda = 0 \rightarrow$
$M\phi_o = 0.2247 \rightarrow$
$P_{F\phi} = 4 \times 0.17 \times 11.4 \times 17.5 \times 12 \times 0.2247 \rightarrow$
$P_{F\phi} = 366$ kN

where: $\lambda = f/8 \rightarrow$
$M\phi_o = 0.2389 \rightarrow$
$P_{F\phi} = 4 \times 0.17 \times 11.4 \times 17.5 \times 12 \times 0.2389 \rightarrow$
$P_{F\phi} = 389$ kN

where: λ = inclination of the surface of rupture (soil) with the vertical plan (pile)

Determination of P_γ

$$P_\gamma = A_L\rho zM_\gamma;$$
$$M_\gamma = M_{\gamma o}[(1 - 0.333)tg\lambda(z/r)];$$

$$M_{\gamma o} = -0.5tg\lambda \rightarrow M_{\gamma o} = 0.035$$
$$P_{\gamma} = 4 \text{ x } 0.17 \text{ x } 11.4 \text{ x } 17.5 \text{ x } 11.4 \text{ x } 0.035 \rightarrow$$
$$P_{\gamma} = 54 \text{ kN}$$

Determination of P_{LU}

where: $\lambda = 0 \rightarrow P_{LU} = 366$ kN

where: $\lambda = \phi/8 \rightarrow$
$P_{LU} = 389 + 54 \rightarrow$
$P_{LU} = 443$ kN

Ultimate load total

$$P_{UT} = P_{LU} + \text{weight of the pile}$$

where: $\lambda = 0 \rightarrow$
$P_{UT} = 366 + 12 \rightarrow$
$P_{UT} = 378$ kN

where: $\lambda = \phi/8 \rightarrow P_{UT} = 443 + 12 \rightarrow$
$P_{UT} = 455$ kN

4.2.3 Method of Meyerhoff and Adams (1973)

Ultimate load lateral

$$P_{LU} = (CA + \sigma_{VM} K_U tg\delta)A$$
$$P_{LU} = (105 \text{ x } 1.7tg28°)(4 \text{ x } 0.17 \text{ x } 11.4)$$
$$P_{LU} = 775 \text{ kN}$$

Ultimate load total

$$P_{UT} = P_{LU} + \text{weight of the pile}$$
$$P_{UT} = 775 + 12 \quad P_{UT} = 787 \text{ kN}$$

4.3 Bearing capacity: methods empirical

For the estimate bearing capacity of the pile the uplift, Décourt and Quaresma (1978) – on the basis of tests SPT – and Velloso (1981) – on the basis of tests CPT – adopt a factor of security equal 0.7, in relation the formulas of bearing capacity the compression (DANZIGER, 1983).

4.3.1 Method of Décourt and Quaresma (1978)

Ultimate load lateral

$$P_{LU} = (f_U pL)0.7$$
$$f_U = (NSPT_{\text{SIDE AVERAGE}}/3) + 1 = (6/3) + 1 = 3 \text{ t/m}^2 \text{ ou } 30 \text{ kPa}$$
$$P_{LU} = 30 \text{ x } 4 \text{ x } 0.17 \text{ x } 11.4 \text{ x } 0.7 = 163 \text{ kN}$$

Ultimate load total

$$P_{UT} = P_{LU} + \text{weight of the pile}$$
$$P_{UT} = 163 + 12$$
$$P_{UT} = 175 \text{ kN}$$

4.3.2 Method of Velloso (1981)

Ultimate load lateral

$$P_{LU} = \lambda \alpha p L f_U$$
$$\lambda = 0.7 \text{ (uplift piles)}$$
$$a = 1$$
$$f_U = ALL = 113 \text{ kPa}$$
$$P_{LU} = 0.7 \text{ x } 1 \text{ x } 4 \text{ x } 0.17 \text{ x } 11.4 \text{ x } 113 = 613 \text{ kN}$$

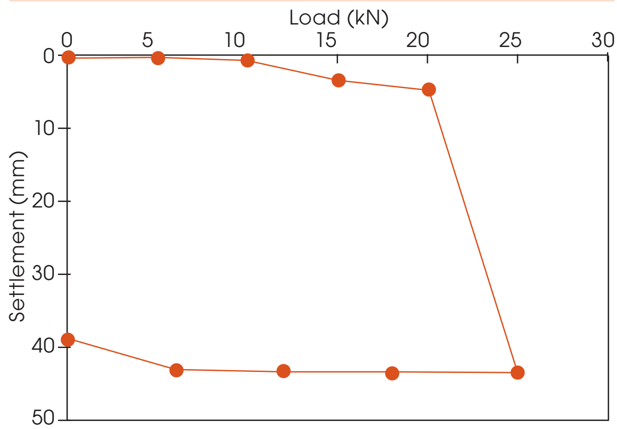
Ultimate load total

$$P_{UT} = P_{LU} + \text{weight of the pile}$$
$$P_{UT} = 613 + 12$$
$$P_{UT} = 625 \text{ kN}$$

4.4 Load tests

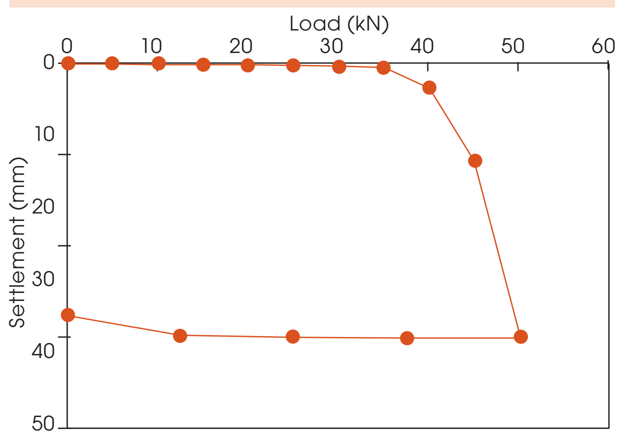
The loads were applied in increments of 5 kN. Three tests were performed in a natural condition of moisture content soil. Graphics 2, 3 and 4 show the load-settlement curves obtained.

A fourth test (pile not tested before) was carried out after the previous soaking of the soil around the pile head (Graphic 5). In this pile it was opened a dig in the soil of 0.60 m of depth and area of 1 x 1 m^2. After that, inundation in the soil was made, with controlled discharge. The time of inundation of the soil was of 72 hours with average discharge of 0.5 m^3/h.



Graphic 2: Load-settlement curve (Pile 1)

Source: Menezes (1997).

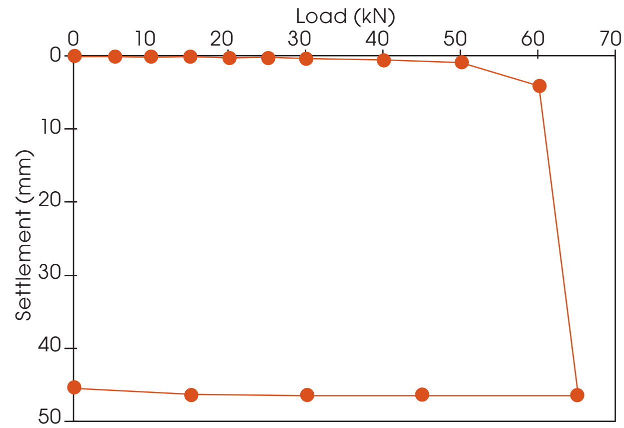


Graphic 3: Load-settlement curve (Pile 2)

Source: Menezes (1997).

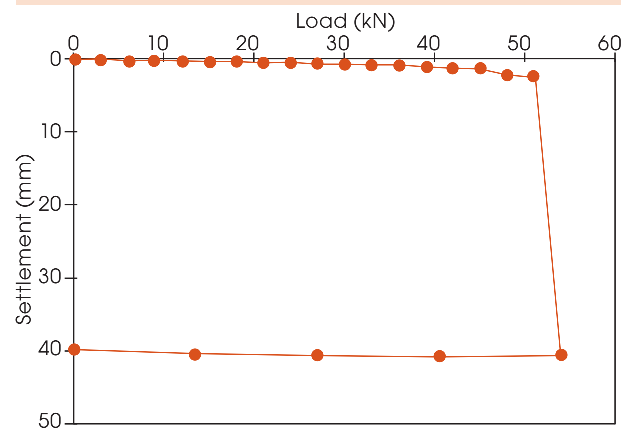
4.5 Load transfer

Due to the low skin friction up to 8 m deep, the analysis for the load transfer data, took into account only two parts of the piles shaft. The first being related to the head of the pile and the other related to the intermediate portion of the pile. The load distribution along piles is shown in Graphics 6, 7, 8 and 9.



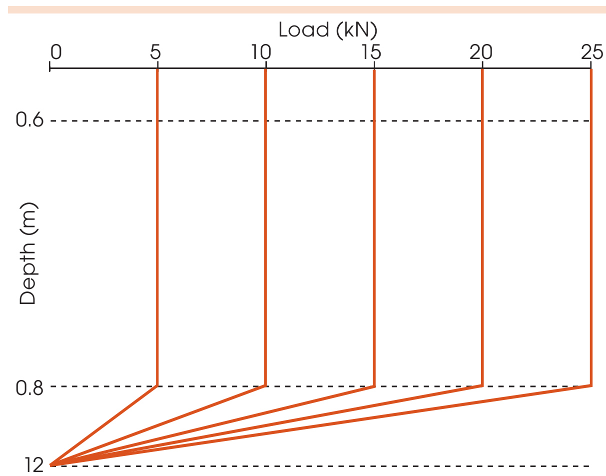
Graphic 4: Load-settlement curve (Pile 3)

Source: Menezes (1997).



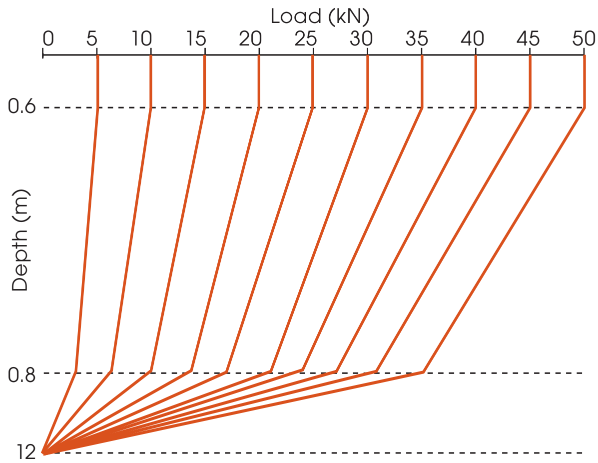
Graphic 5: Load-settlement curve (Pile 4)

Source: Menezes (1997).



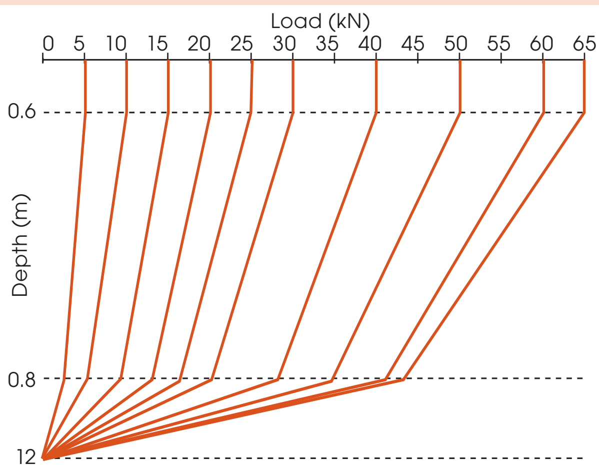
Graphic 6: Load transfer results (Pile 1)

Source: Menezes (1997).



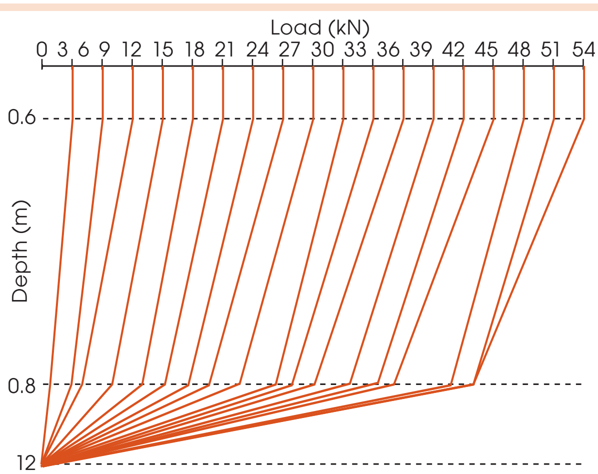
Graphic 7: Load transfer results (Pile 2)

Source: Menezes (1997).



Graphic 8: Load transfer results (Pile 3)

Source: Menezes (1997).

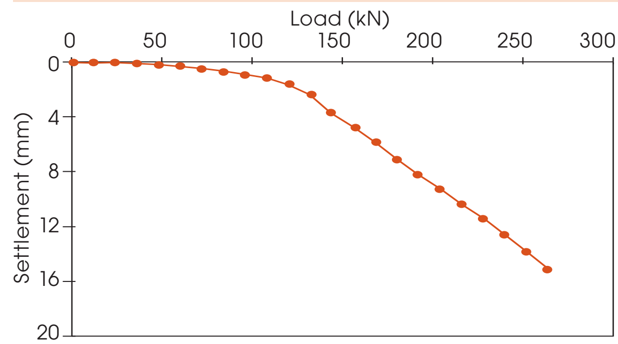


Graphic 9: Load transfer results (Pile 4)

Source: Menezes (1997).

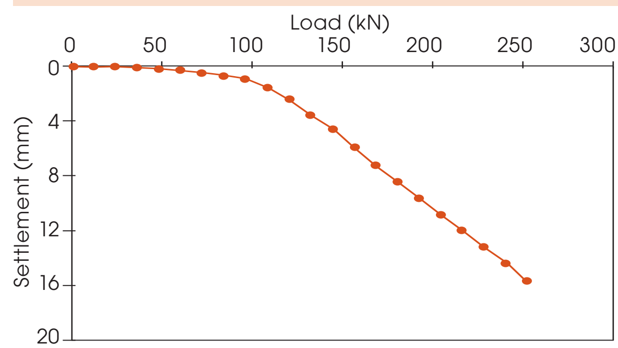
4.6 Bored piles

Vertical static load tests were applied to three bored piles (12 m long and 0.20 m in diameter) so that uplift load values could be found in another type of foundation (bored piles). Graphics 10, 11 and 12 show the load-settlement curves obtained. The aim of these tests was to confront the values obtained with those values reached by the precast-concrete piles.



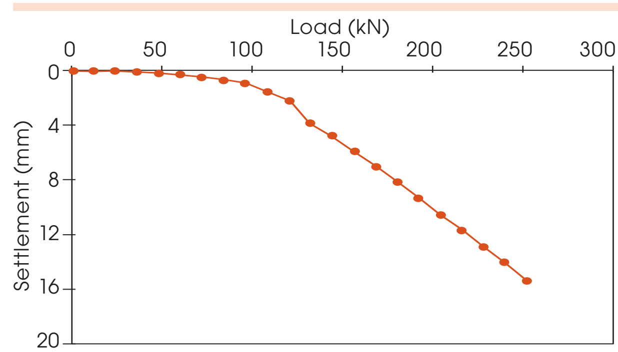
Graphic 10: Load-settlement (bored pile 5)

Source: Menezes (1997).



Graphic 11: Load-settlement (bored pile 6)

Source: Menezes (1997).



Graphic 12: Load-settlement (bored pile 7)

Source: Menezes (1997).

5 Analysis of results

Considering it loss of friction in the initial stretch of the piles (what was verified), the formulas of bearing capacity stayed committed, what would not to occur the piles of compression. The values calculated through the analytical and empirical methods are in Tables 4 and 5, respectively.

Table 4: Results of analytical methods

Methods	Lateral (kN)	Total (kN)
Das (1983)	224	245
Grenoble (*)	366	378
Meyerhoff and Adams (1973)	775	787

Source: Menezes (1997).

Table 5: Results of empirical methods

Methods	Lateral (kN)	Total (kN)
Décourt and Quaresma (1978)	163	175
Velloso (1981)	613	625

Source: Menezes (1997).

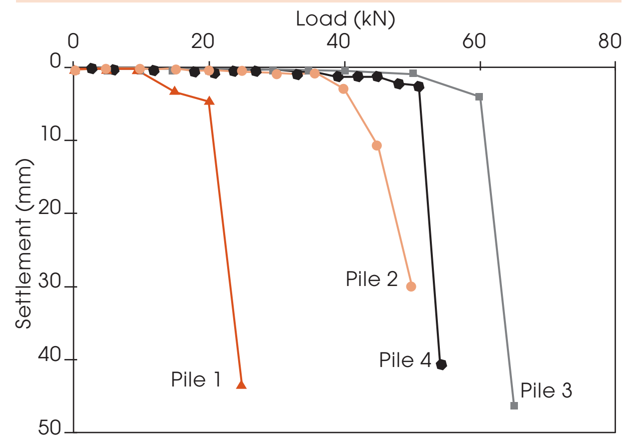
The ultimate load values were extremely low in all piles (Table 6). The pile skin friction values were much lower than the expected when compared with those reached by the bored piles.

Table 6: Results of static tests

Pile test	Soil type	Capacity (kN)
1	Non-soaked	25
2	Non-soaked	50
3	Non-soaked	65
4	Soaked	54

Source: Menezes (1997).

As the analysis results showed skin friction loss, it becomes clear that these formulas do not show a good bearing capacity forecast. Graphic 13 presents the load-displacement curves obtained.



Graphic 13: Load-settlement (piles 1, 2, 3 and 4)

Source: Menezes (1997).

- The pile 1 presented a bearing capacity well lower the piles 2 and 3. That is possible due to existence of residual load in the lower stretch of that pile that would act in the same direction of the force of traction, diminishing the effort for its uplift. The existence of this residual load means that the tip of the pile already is reacting when the lateral friction begins to develop.
- For the piles the compression, this lateral friction, that soon after driving acts from top to bottom, "holding" the pile in the soil and balancing the residual load in the tip, needs to be reverted. In case of uplift piles would not have need of this reversion, therefore the load applied would act in the same direction of the residual load.
- Admitting itself that for the uplift piles of this work, the residual loads in the tip were of the same magnitude, one concludes that they could of some form have influenced to provoke the decrease resistance to the traction obtained in the pile 1.
- With regard to pile 4 (soaked) not to have presented loss of bearing capacity in relation the other piles (non-soaked), this can be explained by this pile not to have suffered some

anomaly from the soil when of its driving. The same it did not occur with the other piles (1, 2 and 3) that had short values of bearing capacity.

- The great vibrations during the driving of the piles, provoke for the trepidation to not adhesion of the pile to the soil, due its characteristics of high porosity, not having a subsequent recuperation of the interaction pile-soil.
- The hypothesis of the effect of the form of installation of the pile to have provoked the loss of lateral friction is to more probable. During the driving which was observed that it was an "empty" between the pile and the soil reached 2 m of depth. Accounts of engineers and operators of driving machines had indicated that this fact generally occurs when are nailed piles in this type of soil of São Paulo State. Procedures of if playing sand in the head of the piles during the driving, to go itself filling the emptinesses between the pile and the soil, had been commented some engineers.

6 Final considerations

- In this work was verified that for that type of soil (porous), the collapse index is higher in the first meters
- The lower values of bearing capacity for the three precast piles, in comparison with bored piles, indicate that the precast piles used are not adequate for soil friction requirement in that soil.
- The field analysis and observations indicate that the vibrations due to the driving process cause irrecoverable lateral pile displacement from the soil at a depth of some meters
- The soaking of the surface soil around the pile head during 48 hours causes a 50% skin friction loss up to 8 m deep.
- As demonstrated, when submitted to uplift load the precast piles showed low bearing capacity. With the ultimate load values for both types of piles, one can better interpret the results.
- The bored piles, submitted the same conditions of *in situ* that the precast-concrete piles, indicated to its good capacity of load to the uplift.

References

ASSOCIAÇÃO BRASILEIRA DE NORMAS TÉCNICAS. *NBR-12131/97: provas de carga estáticas em estacas*. Rio de Janeiro: ABNT, 1997.

CARVALHO, D. *Análise de cargas últimas a tração de estacas escavadas, instrumentadas, em Campo Experimental de São Carlos (SP)*. 1991. Tese (Doutorado em Engenharia Civil)-Escola de Engenharia de São Carlos, Universidade de São Paulo, São Carlos, 1991.

DANZIGER, F. A. B. *Capacidade de carga de fundações submetidas a esforços verticais de tração*. 1983. Tese (Doutorado em Engenharia Civil)-Coordenação dos Programas de Pós-Graduação em Engenharia, Universidade Federal do Rio de Janeiro, Rio de Janeiro, 1983.

DAS, B. M. J. et al. A procedure for estimation of uplift capacity of rough piles. *Soils and Foundations*, Tokyo, vol. 23, no. 3, pp. 122-126, 1983.

DÉCOURT, L.; QUARESMA, A. R. Capacidade de carga de estacas a partir de valores de SPT. In: CONGRESSO BRASILEIRO DE MECÂNICA DOS SOLOS E ENGENHARIA DE FUNDAÇÕES, 6., 1978, Rio de Janeiro. *Anais*... Rio de Janeiro: 1978. vol. 1, pp. 45-53.

MARTIN, D. *Étude à la rupture de différents ancrages sollicitées verticalement*. 1966. Thése (Doctorat de Ingénieur)-Faculté des Sciences de Grenoble, Grenoble, 1966.

______. *Foundations profondes sollicitées à l´arrachement en milieu cohérent tridimensional*. 1963. Thése (Doctorat de Spécialité)-Faculté des Sciences de Grenoble, Grenoble, 1963.

MENEZES, S. M. *Análise do comportamento de estacas pré-moldadas em solo de alta porosidade do interior do Estado de São Paulo*. Tese (Doutorado em Engenharia Civil)-Escola Politécnica, Universidade de São Paulo, São Paulo, 1997.

MEYERHOFF, G. G.; ADAMS, J. I. The ultimate uplift capacity of fundations. *Canadian Geotechnical Journal*, Ottawa, vol. 5, no. 4, pp. 224-244, 1973.

ORLANDO, C. *Fundações submetidas a esforços verticais axiais de tração. Análise de provas de carga de tubulões em areias porosas*. 1985. Tese (Doutorado em Engenharia Civil)-Escola Politécnica, Universidade de São Paulo, São Paulo, 1985.

RIBIER, B. *Étude des argiles fortement plastiques et essais de soulèvement des pieus*. 1962. Thése (Doctorat de Spécialité)-Faculté des Sciences de Grenoble, Grenoble, 1962.

VELLOSO, P. P. C. *Considerações sobre a estimativa da capacidade de suporte e dos deslocamentos vertical e horizontal de estacas em solo*. 1. ed. Brasília: UnB, 1981.

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