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Steel pile under lateral loading in a very soft clay deposit

Pieux métalliques chargés lateralment dans un dépôt d'argile molle

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ABSTRACT

This work presents experimental as well as analytical studies of the behavior of laterally steel pile in a thick layer (17 meters) of soft clay. A lateral load test was performed to obtain experimental displacements in the field. This test was performed in two steel piles driven into a Recife organic clay deposit. The test was conducted in the experimental station of the UFPE Geotechnical Group (SESI-Ibura research site) in Recife, Pernambuco State, Brazil. The field results are compared to those obtained by linear and nonlinear finite element analysis. In this investigated area occurred a general rupture of a concrete structure on steel pile foundation due to horizontal displacement of the very soft organic clay deposit. The linear analysis was performed using a subroutine of beams and beam-columns on elastic foundation. The subroutine implemented in this work models the soil stiffness variation with depth as function of n_h (coefficient of horizontal subgrade reaction). The material and geometrical nonlinear analysis was performed in the pile-spring system by ANSYS commercial package. The nonlinear material soil behavior was modeled by p-y curves, obtained in this work by two methods, the fist proposed by Robertson at al. (1989) from dilatometer test and the other by Ménard et. al. (1969) from pressuremeter tests results. As a complement to Ghali's implementation (1989), it was developed a subroutine which adjusts geotechnical parameter n_h from experimental displacements results by a nonlinear least squares technique. The n_h determination was also done through Cintra and Albiero's method (1982), which is also based on experimental data. Estimates of the critical buckling loads for piles in soft organic soils for boundary condition are also presented.

RÉSUMÉ

Cet article présente les résultats d'études analytiques et experimentaux du comportement de pieux métalliques chargés lateralment dnas une couche épaisse (17 mètres) d'argile molle. Un essai de chagement latéral a été realizé pour obtenir des déplacements experimentaux in situ. Cet essai a été realizé dans deux pieux métalliques dans un dépôt d'argile organique de Recife, dans le site experimental de Groupe Geotechnique de l'UFPE, a Recife, Pernambouc, Brésil. Les résultats in situ ont été comparés a ceux obtenus pa une analyse lineaire et non lineaire en éléments finis. Dans le site expérimental il s'est produit une rupture générale de la structure en béton avec fondation en pieux métalliques, par déplacement horizontal de la couche d'argile organique molle. L'analyse lineaire a été realizée avec des poutres sur fondation élastique. Le programme implanté considère la rigidité du sol varient avec la profondeur, en fonction du n_h (coefficient de reaction horizontal). L'analyse non lineaire du système pieux-ressort a été realizé avec le programme ANSYS. Le comportement non lieaire du sol a été modelizé avec les courbes p-y, obtenus par deux méthodes: la première proposé para Robertson et al. (1989) a partir du dilatomètre et la deuxième celle de Ménard et al. (1969) a partir de l'essai présiometrique. En complémentation de l'implantation de Ghali (1989) il a été développé un programme qui adapte le paramètre n_h a partir des déplacements expérimentaux avec une technique non lineaire. La détermination de n_h a été faite aussi par la méthode de Cintra e Albiero (1989), basée dans des données experimentaux. C'est présentée aussi l'évaluation des charges critiques pour des pieux das des sols organiques mous pour trois conditions de frontière et de configuration.

1 INTRODUCTION

The presence of a soft clay deposit requires a detailed assessment of soil parameters in order to analyze the foundation performance.

In 1995, a thorough rupture in a reinforced concrete structure of a floor on steel piles in Recife, Brazil, 21 years after it had been built, without no indication of possible ruin, displayed the importance of a buckling study in steel piles caused by lateral displacement in soft soil. The subsoil at the accident site is made up of a thick layer of very soft organic clay about 17m thick. After that, the Geotechnical Group of the Federal University of Pernambuco, Brazil, has performed extensive geotechnical research program in the area (UFPE - Research Site 2).

This paper shows a study on the behavior of laterally-loaded steel piles in thick layers of soft clay, and consists of analytical and experimental stages. For further details refer to Braga (1998).

In the experimental stage, lateral loading tests in steel piles driven into the organic clay deposit were carried out where the aforementioned accident took place. In the analytical stage, predictions on the horizontal displacements of piles top were made, and also estimates for the buckling load of a steel pile in very soft clay obtained from linear and non-linear analyses of the problem through the finite element method.

The soil was modeled by p-y curves obtained from dilatometer (DMT) and pressuremeter (PMT) testing results performed at the site of the accident and near the damaged structure (which curves score are data entered in the ANSYS program) that bear deforming-power element. The following hypotheses were considered: the steel pile was perfectly vertical and steel pile with vertical load excentricity, that is, with initial lateral deformation.

2 GEOLOGICAL-GEOTECHNICAL CHARACTERIZATION

Recife lies on the northeast coast of Brazil. The organic clay deposit is geologically characterized for being located in the realm of two marine areas originated during the last advance (Pleistocenic) and the last recedines (Holocenic) of the sea. It was developed in fluvial-marine and mangrove setting, which age is thought to be of up to 10,000 years.

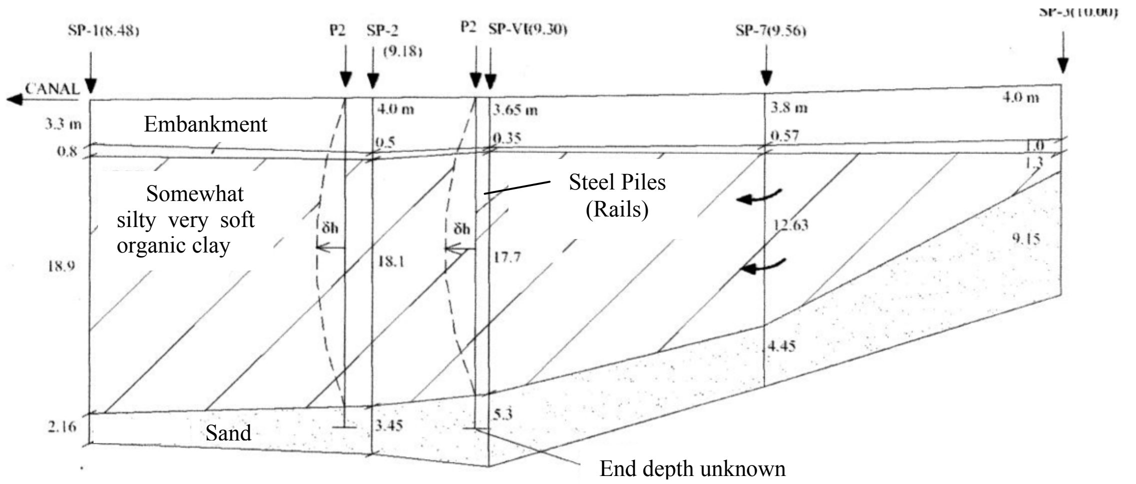

Figure 1. Geotechnical profile – horizontal pull

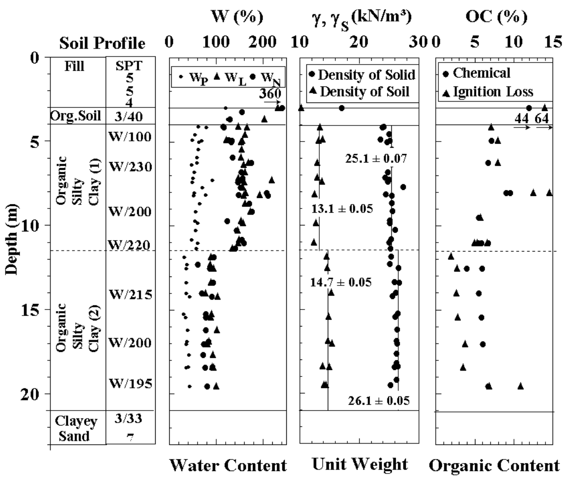

Figure 2. Results from characterization tests with depth

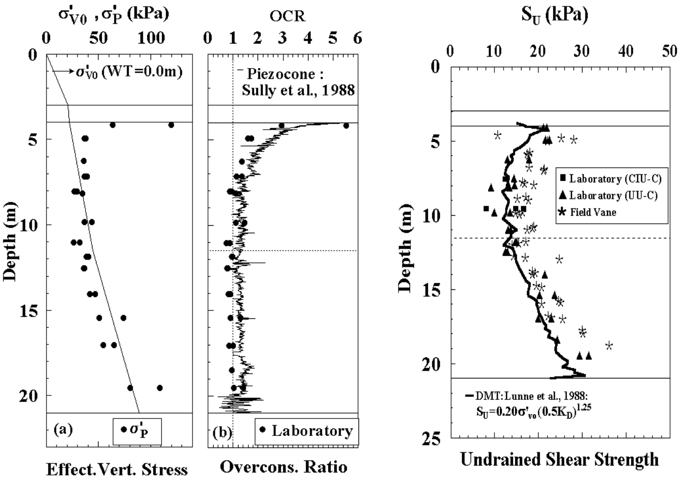

Figure 3. a) Stress history; b) Undrained shear strength vesus depth

The area studied was geotechnically characterized through a comprehensive geotechnical program (Coutinho et al, 1999). Figure 1 displays the geotechnical profile where can be seen the occurrence of an embankment layer (26 years old or more) about 3m thick, where the water level is, which has showed a range of 1.6 m throughout the year. Under the embankment a small layer of organic soil about 0.7 and 1.0m thick is observed. Next, emerges the very soft organic silty clay with 17m thick along with shell fragments and the remains of decaying vegetetion. Figure 2 displays a synthesis of the plysical characterization of the deposit, where it is possible to see a very soft organic clay with high water content and plasticity.

This deposit is subdivided into two clear-cut layers, the first overdensed (OCR ≤ 3) from 4 to 11.5m, and the second roughly normally densed (OCR < 1.2) from 11.5 to 21m. Figure 3 shows the history stress and undrained shear along the depth.

Among the geotechnical investigations performed (Coutinho et al, 1999), it can be cited the dilatometer and pressuremeter testings (Figures 4 and 5). All tests were performed near the damaged structure.

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3 PROBABLE CAUSE FOR THE ACCIDENT

The accident turned out to be a general rupture in reinforced concrete structure of a floor on steel piles (three rails of the TR-25 type), 21 years after it had been built-without no indication of possible ruin. A sudden settlement measuring about 1m struck only one the edges of the building leaving it with a deformed form slightly linear.

With the geotechnical profile of the area subsoil and according to the characteristics of the accident, a hypothesis was proposed as for the accident to have been triggered by slow lateral movement of the organic clay layer, which over time would have provoked a lateral displacement in the pile (Figura 1), in addition to the vertical load (self weight + negative friction) causing the pile to collapse.

This slow lateral displacement of the soil would have been ensued by the difference in thickness of the embankment layer on the organic clay with the latter layer, tending to move from its smaller thickness point (1.3m SP-3) towards the higher thickness point (18m-SP-2).

4 LATERAL LOADING TEST

A lateral loading test on a steel tubular pile was carried out at the site of the accident with a hydraulic ram performing between two piles according to ASTM-D 3966 norm (1990).

The scheme for the lateral loading test and instrumentation is shown in Figure 6.

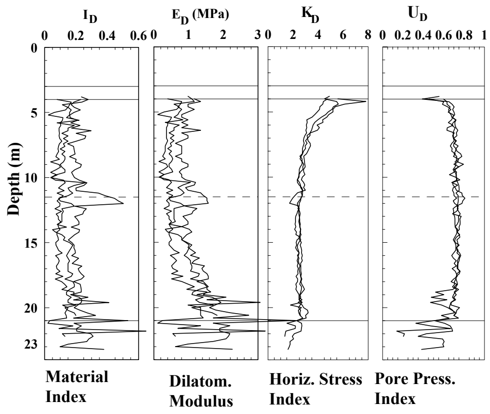

Figure 4. Dilatometer index

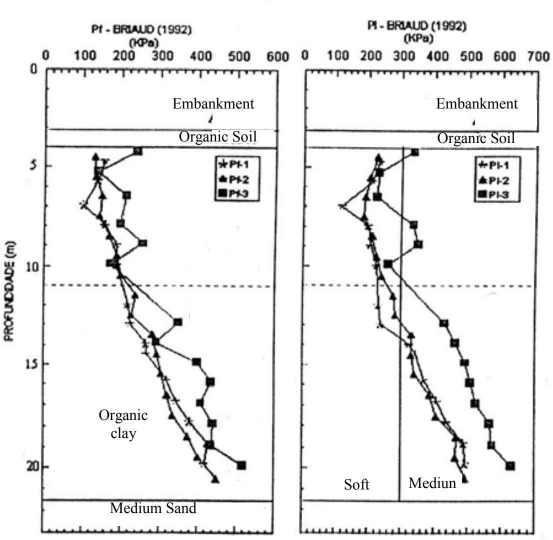

Figure 5. Fluence and limit pressure (P_L and P_F)

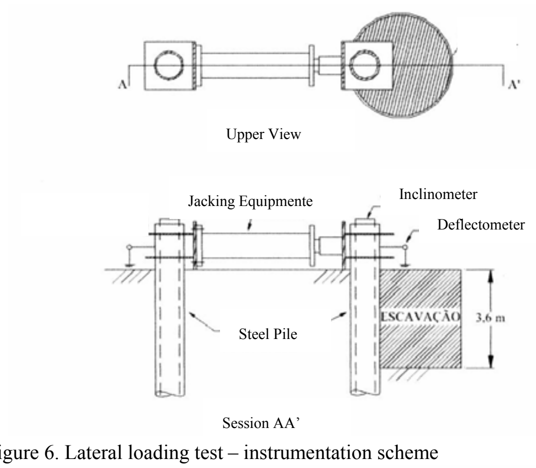

Figure 6. Lateral loading test – instrumentation scheme

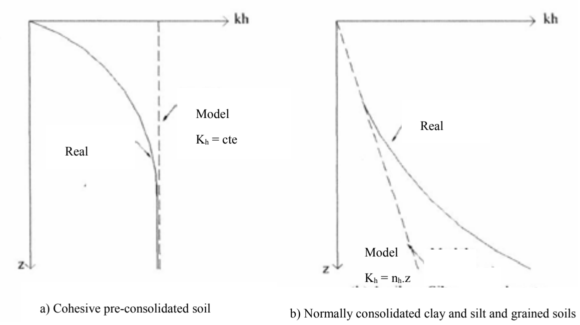

Figure 7. Soil rigidity variation with deph

5 UTILIZED MODELS

Because the pile diameter is usually small when compared to its length, analysis of the pile could be performed through the diferential equation of a beam on an elastic base. The soil approximation reaction method is frequently applied in calculating piles, which takes after the model by Winkler (1967) where soil reaction is replaced with a series of independent and elastolinear springs.

There are two theories to show soil approximation reaction which are as follows:

a) Soil horizontal reaction module (linear theory)

The soil horizontal reaction module k_h (soil rigidity) varies along the length of pile as shown by Davisson (1963) and reported by Prakash and Sharma (1990) (Figure 7).

In the expression for the soil horizontal reaction module k_h displayed in Figure 7 (b), n_h is the constant of soil horizontal reaction expressed in power per unit of volume.

Lateral displacements of steel tubular piles were obtained from the readings of an inclinometer inside the steel pile and extensometers on the top. In order to obtain more accurate results from organic clay layer, an excavation of the embankment around the steel piles was performed.

The results from this loading test were utilized in a finite element program implemented by Ghali et al. (1989) to calculate the beam or columns.

On this routine the soil is represented by the reaction module of soil k_h, varying according to Davisson (1963), (Figure 7), the coefficient of soil horizontal reaction n_h being a data entered in the program.

The n_h values according to the routine implemented and utilized in this work were as follows: organic clay layers n_h = 849.1 kN / m²; sand layer n_h = 15,000 kN / m².

b) Curves p-y (nonlinear theory)

The actual relationship between soil reaction *p* and its deformation *y* is nonlinear and varies with depth. To incorporate this nonlinearity in soil behavior it is required modifications in the model by Winkler through the use of the p-y curves.

The p-y curves were found through the semi-empirical method proposed by Robertson et al (1989), which utilizes data from dilatometer tests, and the semi-empirical method proposed by Ménard (1969), which utilizes data from pressumeter tests.

6 ANALYSES PERFORMED

The analyzed steel pile is a tubular steel pile with an outside diameter of 12.0 cm, inside diameter of 10.8 cm, and 0.6 cm thick and elasticity module E = 21x10^7 kN/m². This steel pile was used for the lateral loading test mentioned earlier.

Comparisons between horizontal displacements measured on the top of steel piles with predictions using linear and nonlinar analyses were made.

Concerning the critical load of buckling, analyses to estimate it were performed through the finite element method using ANSYS program taking into consideration the following case : steel pile buried to its entire length in three different soil layers (Figure 8)

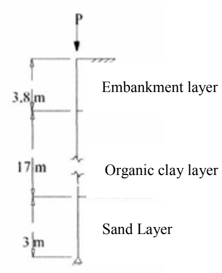

Figure 8. Case studied to estimate buckling load

The following boundary conditions were considered: on the top (free or marked) and fixed on the end. For the analysis through the finite element method the steel pile modeled with beam elements. The plane beam element was utilized, which is available at ANSYS.

7 RESULTS

7.1 *Horizontal displacement*

The horizontal displacements measured and predicted through linear and nonlinear analysis, at land level after excavation versus loading applied, are displaed in Figure 9 and summarized in Table 1. It can be noted that the nonlinear analyses (DMT and PMT) results are very close to the values measured showing, in general, differences ranging from 1 % for 20 %.

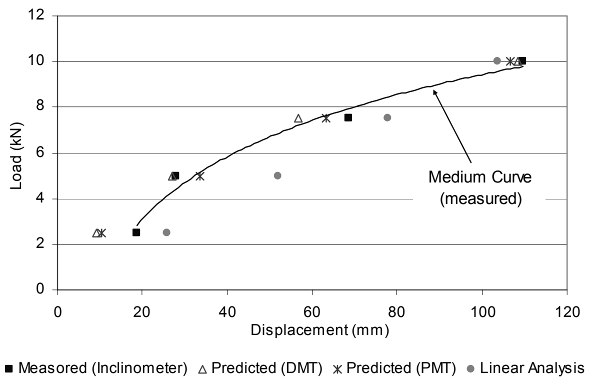

Figure 9. Predicted and measured displacements

Table 1. Predicted and measured displacements

	H = 2,5 kN	H = 5,0 kN	H = 7,5 kN	H =10,0 kN
	Desloc. (mm)	Desloc. (mm)	Desloc. (mm)	Desloc. (mm)
Anál. Linear	25,95	51,9	77,85	103,81
DMT	9,22	27,09	56,85	108,56
PMT	10,32	33,52	63,27	106,74
Medido	18,59	27,86	68,71	109,65

7.2 *Buckling load and Load of Capacity*

In the analysis of the collapse on steel piles, two important facts must be taken into consideration: a) whether the steel pile is throughly vertical and; b) whether there is no excentricity in the vertical load. The results of critical loading of buckling found through the analyses in ANSYS are in Table 2.

Table 2. Estimates of critical loading of buckling

Soil Representation	Critical Loading of Buckling (kN)	
	Free Top	Labeled Top
Curve P-Y (DMT)	2.988,64	2.988,64
Curve P-Y (PMT)	1.925,11	1.925,11

Table 3. Critical loading due to accidental displacements

	Critical Loading (kN)			
	Curves P-Y (DMT)		Curves P-Y (PMT)	
Deformation (cm)	Free Top	Labeled Top	Free Top	Labeled Top
0	2.988,64	2.988,64	1.925,11	1.925,11
1	1.738,18	1.738,18	1.877,44	1.877,44
2	1.183,99	1.183,99	510,21	510,21
3	360,29	360,29	98,86	98,86
5	58,19	58,19	70,89	70,89
10	46,25	46,25	56,90	56,90

It was assumed that the steel pile suffered horizontal displacements showing a second degree parabola form. These displacements were triggered by lateral nodal loads at the scores 1, 2, 3, 4 , 5, and 10 cm in L / 2. The analysis results of critical loading due to accidental displacements performed according to ANSYS are summarized in Table 3.

It was observed that the critical load is considerably sensitive to the effect of accidental displacements which decreases rapidily its value.

The analyzed case depicts the actual situation at the accident site, and the loading capacity, which according the analysis of the p-y curves (DMT) is decreased 70% for an accidental displacement of 1 cm, reaching a decrease of 98% for an accidental displacement of 5cm.

The loading capacity of the steel pile under analysis was calculated through the Aoki-Velloso method (1975) using data from SPT performed at the accident site.

According to this method, PR rupture load of an isolated steel pile is equal to the sum of the portion of PL friction with the portion of PP end. We attained the following values:

PR = 373 kN PL = 228 kN PP = 145 kN

The working load for the steel pile would be 186,5 kN and be within the interval which determines the occurrence of ruin corresponding to an accidental displacement between 3 and 5cm, as shown in Table 5.

8 CONCLUSIONS

Lateral displacements reduce drastically the vertical loading capacity of the steel pile in soft clay deposits as can be observed through the nonlinear analysis.

The lateral displacements obtained through the nonlinear analysis by using p-y curves ascertained from DMT and PMT tests are sufficiently in accordance with results measured in the lateral loading test already mentioned.

The pressuremeter testing (PMT) performed, albeit a pre-drilling one, a detail which may provoke disturbance in the soil, turned out to be a sufficiently viable test for obtainining data needed for generating p-y curves for soils with very soft consistency.

Even in the event of very soft clay, steel piles with perfectly rectilinear conformations and loads well centered show increased loads of buckling. However, the small horizontal displacements taking place decrease greatly its bearing capacity.

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