Geotechnical Investigation of the Recife Soft Clays by Dilatometer Tests

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ABSTRACT: The presence of soft clay deposits requires careful evaluation of soil parameters to analyze the performance of foundations. Due to its high compressibility and low strength, soft clays usually present serious problems. Laboratory and in situ tests are usually used to obtain the soil properties. Comprehensive research has been carried out in Recife soft clay deposits in northeastern Brazil by the Geotechnical Group of the Federal University of Pernambuco, Brazil (Coutinho et al., 1997; 1999; 2002). This paper presents an evaluation of the geotechnical information from Recife soft clays (two research sites) using the dilatometer test (DMT). Classification of types of soils, stress history and in situ horizontal stress, compressibility and strength parameters are obtained and discussed with the literature results. Comparisons are also made with laboratory and in situ reference tests results. In general, the results obtained confirm the potential of the dilatometer to obtain good predictions of geotechnical parameters in these soft clay deposits. In one of the sites investigated, the research was prompted by the general failure of a concrete structure caused by buckling of steel pile foundations in 1995. A lateral load test was performed in two steel piles, and the field results were compared to those predicted using linear and nonlinear finite element analysis. In a nonlinear analysis, lateral displacements reduce drastically the vertical loading capacity of the steel pile in soft clay deposits. DMT testing turned out to be a sufficiently viable technique for obtaining data needed for generating p-y curves in very soft soils (Coutinho et al., 2005).

1. INTRODUCTION

More than 50% of the plain area of the city of Recife is underlain by soft ground deposits. Due to its high compressibility and low resistance, the presence of soft clay deposit requires careful evaluation of soil parameters to analyze the performance of the foundations. Laboratory and in situ tests are usually used to obtain the soil properties. The flat dilatometer test (DMT) was developed in Italy (Marchetti, 1980) and has become a routine site investigation tool in more than 40 countries over the world. A general overview of the dilatometer and its design applications, guidelines for the proper execution, basic interpretation methods and recent findings and practical developments are given by Marchetti et al (2001) in a report under the auspices of the ISSMGE Technical Committee TC16.

Since 1980 the Geotechnical Group of the Department of Civil Engineering of the Federal University of Pernambuco has developed a research program in the Recife soft clays deposits performing laboratory and in situ tests for many sites of the plain area (Coutinho et al 1997, 1998, 1999, 2002). The primary goals of the research program include evaluating the applicability in the Recife soil deposits of the tests developed in other countries, developing of advanced operational techniques or equipment better suited to our natural conditions, publishing the results for use by the Profession, comparing of the results with references laboratory and in situ tests and the formation and continually expanding the knowledge data base.

This paper presents an evaluation of the geotechnical information from Recife soft clays (two research sites) using the DMT. Classification of soil types, stress history and in situ horizontal stress, compressibility and strength parameters are obtained and discussed with results from the literature and from laboratory and in situ reference tests. In one of the sites investigated, the research was prompted by the general failure of a concrete structure caused by bucking of steel pile foundations. A lateral load test was performed on two steel piles, the field results being compared to those predicted by linear and nonlinear finite element analysis. The influence of lateral displacement on the vertical loading capacity of a steel pile in soft clay deposit is also investigated (Coutinho et al., 2005).

2. CHARACTERISTICS OF THE EXPERIMENTAL FIELD

Figure 1 shows the location of Recife city and the investigated soft clays sites in the lowland area (Coutinho et al., 1998). Recife has two soft clays research sites being studied by the Geotechnical Group of the Federal University of Pernambuco: RRS1 (International Club) and RRS2 (SESI-Ibura). The RRS1 is located near the center of the city and the RRS2 is located near the Recife Airport. In the later one, a geotechnical accident occurred, in 1995, causing total destruction of an one-floor structure on steel pile foundation.

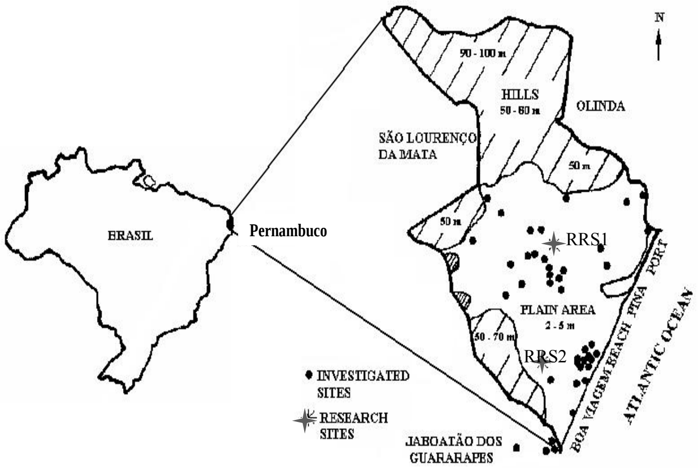
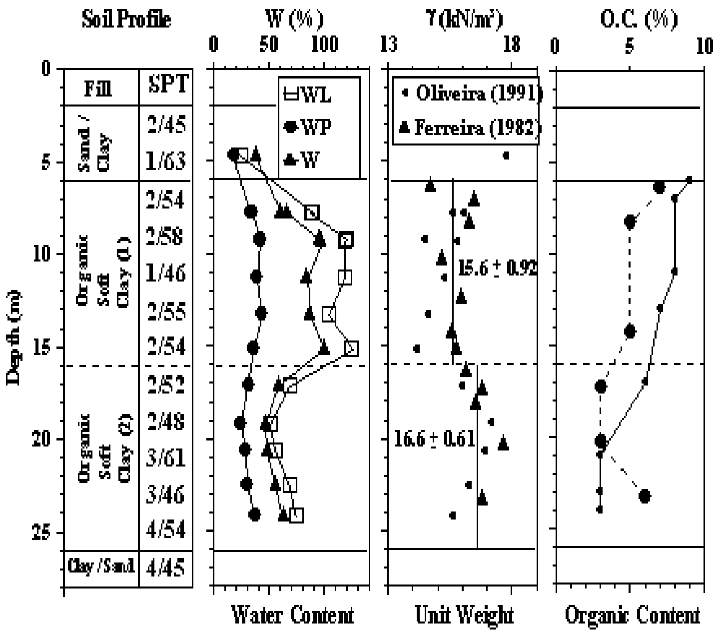


Figure 1. Location of Recife – Pernambuco / Brazil and the Research Sites (RRS1 and RRS2)

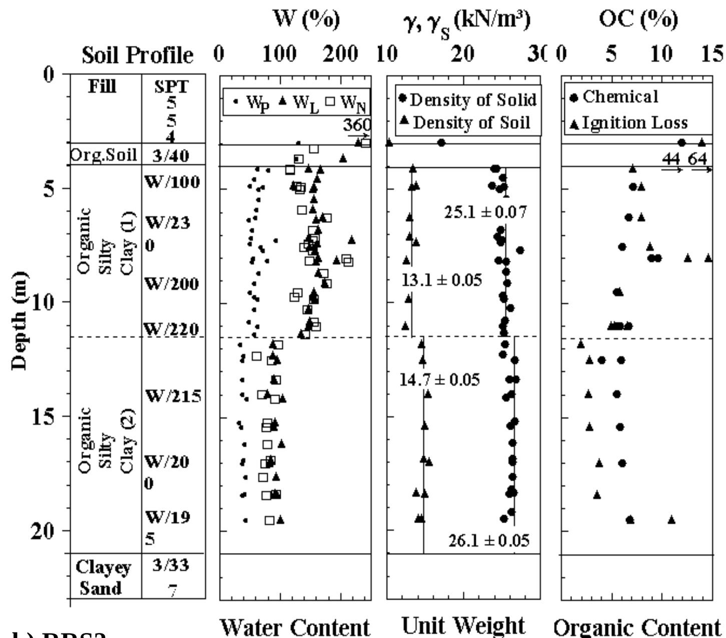
Figure 2 presents the soil profile and results of the characterization tests from the RRS1 and RRS2. The soil profile of the RRS1 consists of 6-7 meters of clayey sand and sandy clay, underlain by a soft organic clay with a thickness of about 20 meters. This organic clay can be subdivided into two layers, with the lower layer having lower plasticity. SPT (N-value) varying from 1 to 4, and are usually between 2 and 3. Underneath this, there are alternate layers of sand and clay with the SPT N-values increasing in depth. The water table level is between 1 and 2 meters deep depending on the season. The results of the characterization tests were usually quite different from each soft clay layer. The natural water content is usually presented slightly below the liquid limit in both layers, showing values in the range of 65-100% in layer 1 (6–16m) and in the range of 45-65% in layer 2 (16-26m). The plasticity index of the first soft layer is 70.4 ± 12.4%, while in the second soft layer the values are 33.0 ± 5.7%. The organic content is also higher in layer 1 (7.0 ± 1.5%) than in layer 2 (3.7 ± 1.7%). The grain size distribution for both layers can be described as 65% clay, 25% silt, and 10% sand.

The soil profile of the RRS2 consists of about 3 meters of old embankment, underlain by a clayey peat layer with thickness of about 1 meter and a very soft organic clay deposit (SPT: 0/200) with a thickness of 17 meters, subdivided into two layers. Below the organic clay, a clayey sand layer is observed. The water table level is 0 to 1 meter deep.

Artesian pressure and gas pressure also were observed showing higher pore water pressure than the hydrostatic conditions, inside of the very soft clay layers, reducing the overburden effective stress.



a) RRS1



b) RRS2

Figure 2. Results of Characterization Tests vs Depth: (a) Research Site 1; (b) Research Site 2 (Coutinho & Oliveira, 1997; Coutinho et al., 1999).

The natural water content is close to the liquid limit in both soft clay layers, being 149.7 ± 23.7% in first layer, and 84.2 ± 15.5% for the second layer. The plasticity index of the first soft layer (4-11.5m) is 97.5 ± 13.6%, while in the second soft layer (11.5-21m) the values are 53.1 ± 5.9%. The organic content is usually between 3 and 10%, with the first layer generally having slightly higher values. The grain size distribution for both layers can be described as 72% clay, 20% silt, and 8% sand.

3. DILATOMETER TESTS

Three dilatometer test soundings (D_1, D_2 and D_3) were performed at each research site. The dilatometer blade and membrane were standard as defined by Marchetti (1980). The dilatometer control unit was a 1985 model. The procedures used were in accordance with what is suggested in the literature (e.g. ASTM, 1986; Schmertmann, 1988; Campanella and Robertson, 1991). The corrected pressures and intermediate DMT parameters were obtained using Equations 1 - 3 and Equations 4 - 7, respectively.

Corrected pressures:

$$p_0 = 1.05\,(A - Z_M - \Delta A) - 0.05\,(B - Z_M - \Delta B) \qquad (1)$$

$$p_1 = (B - Z_M - \Delta B) \qquad (2)$$

$$p_2 = (C - Z_M + \Delta A) \qquad (3)$$

Intermediate DMT parameters:

$$I_D \text{ (material index)} = (p_1 - p_0) / (p_1 - u_0) \qquad (4)$$

$$E_D \text{ (dilatometer modulus)} = 34.7\,(p_1 - p_0) \qquad (5)$$

$$K_D \text{ (horizontal stress index)} = (p_0 - u_0) / \sigma'_{V0} \qquad (6)$$

$$U_D \text{ (pore-pressure index)} = (p_2 - u_0) / (p_0 - u_0) \qquad (7)$$

Figure 3 presents the results of the intermediate DMT parameters for the three DMT test soundings performed in each research site. This figure shows a repeatable and continuous profile of the measured parameters.

4. DERIVATION OF GEOTECHINICAL PARAMETERS

4.1. Stress History / State Parameters

(a) Soil type

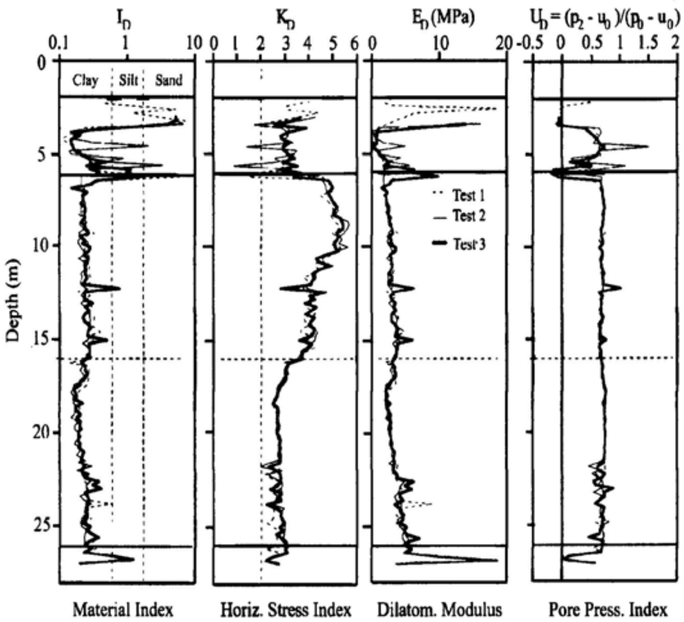
According to Marchetti (1980) the soil type can be identified as follows: clay ($0.1<I_D<0.6$), silt ($0.6<I_D<1.8$) and sand ($1.8<I_D<10$).

Figure 4 summarizes the positions of the soils tested by NGI on the dilatometer soils classification chart proposed by Marchetti & Crapps (1981) and modified by Lacasse & Lunne (1988). The newer information enables one to illustrate qualitatively the effects of overburden, overconsolidation ratio and density on the dilatometer modulus. For Norwegian soils, material indices between 0.05 and 0.1 have been obtained. The original chart was therefore extended in this direction.

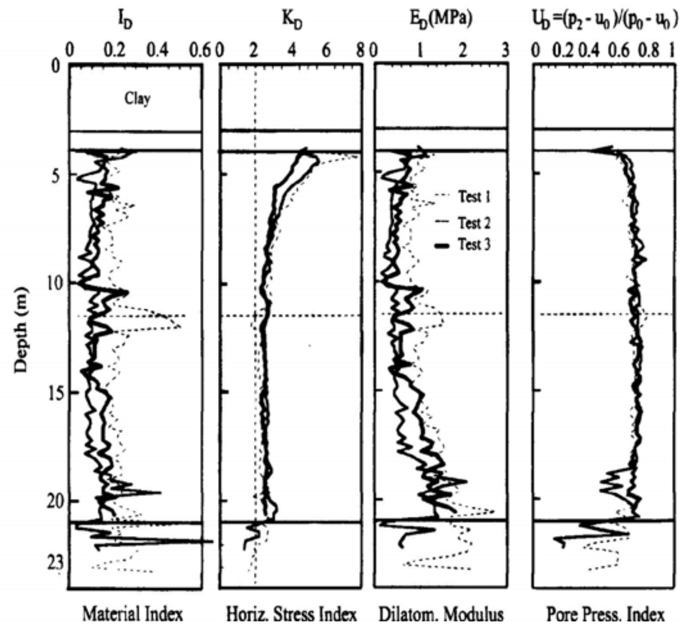
The positions of the Recife soft clays deposits are superimposed on that classification chart in Figure 4 and they agree with the soil sample descriptions shown on Figure 2.

(b) Unit Weight

Figure 5 presents comparisons of the unit weight predicted by the Marchetti and Crapps (1981) dilatometer soil classification chart (Figure 4) and reference unit weights measured in the laboratory for the both Recife Research Sites (RRS1 and RRS2).



a) RRS1



b) RRS2

Figure 3. Dilatometer test results – I_D, K_D, E_D, U_D vs Depth: (a) Research Site 1; (b) Research Site 2 (Coutinho & Oliveira, 1997; Coutinho et al., 1999).

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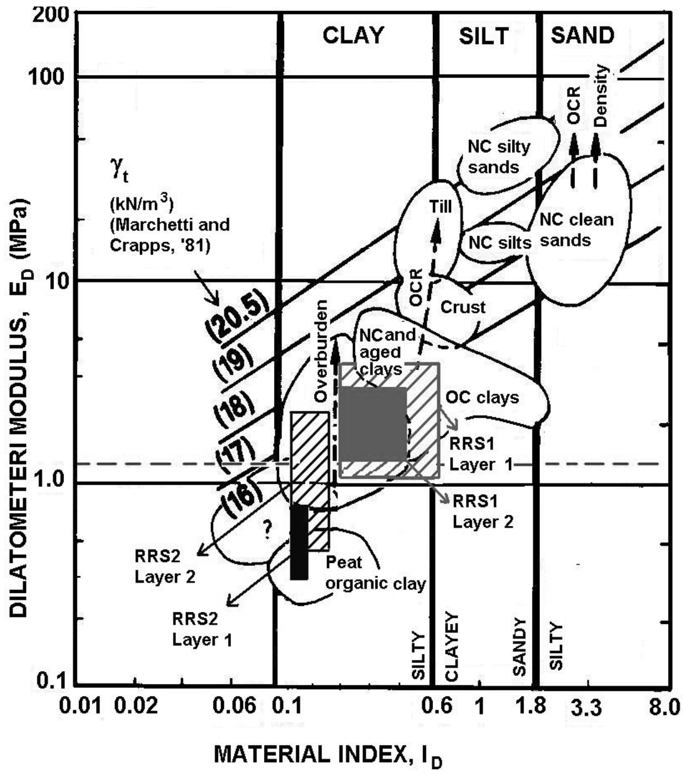
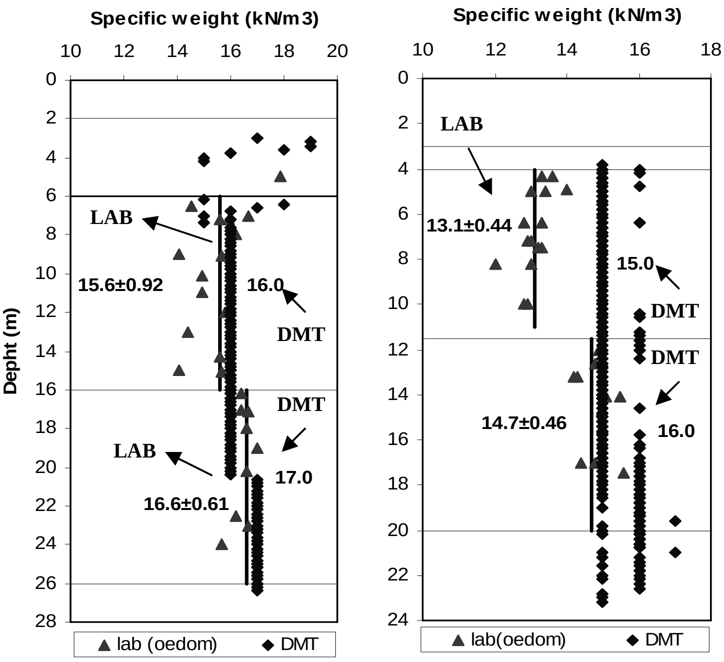


NC = Normally consolidated
OC = Overconsolidated

Figure 4. Classification chart for soils test. Effects of overburden, overconsolidation ratio and density (Lacasse & Lunne, 1988) with results of Recife Soft clay deposits.



(a) RRS1 **(b) RRS2**

Figure 5. Comparison between γ_{DMT} vs. $\gamma_{lab.}$: (a) Research Site 1; (b) Research Site 2.

Lacasse & Lunne (1988) observed that the chart tends to underpredict the unit weight in soft clays. Marchetti et al. (2001) comment that the main scope of the chart is not the accurate estimation of unit weight, but the possibility of constructing an approximate profile of σ'_{vo}, needed in correlations.

In Figure 5a (RRS1) can be seen that in general the estimated results agree with the laboratory results, in both layers of the deposit. For the RRS2 (Figure 5b) it can be seen that, in the layer 2 the estimated results are close to the laboratory; however, in the layer 1, where the clay is in a very soft consistency (E_D<1000kPa), with presence of organic content and high percent of natural water content (149.7 ± 23.7%), the values of unit weights obtained from the chart are higher than the laboratory. These results are different from that observed by Lacasse & Lunne (1988).

(c) Coefficient of earth pressure at rest K_0

The effective in situ horizontal stress, σ'_{h0} (or coefficient of earth pressure at rest K_0) is an important geotechnical parameter but very difficult to obtain accurately with any device. In general, there is an uncertain reability, because of the scarcity of reference values (Lunne et al, 1990).

In this research the Equations 8 to 10 were used for obtaining the K_0 values from correlation proposal in the literature.

$$K_0 = (K_D/1.5)^{0.47} - 0.6\text{; (Marchetti, 1980)} \qquad (8)$$

$$K_0 = 0.34\, K_D^{0.54}\text{; (Lunne et al., 1990)} \qquad (9)$$

$$K_0 = (1 - \sin\phi')\, OCR^{\sin\phi'}; \qquad (10)$$

(Mayne & Kulhawy, 1982).

Figure 6 presents the average values of K_0 that were obtained using Equation (9) and (10) considered, showing that the DMT results (Lunne et al., 1990) were close to the "laboratory" correlation (Mayne & Kulhawy, 1982). Lunne et al. (1990) estimated that for the "young" clays the uncertainty associated with K_0 from DMT is about 20%.

Figure 7 confirms this result and shows that the Marchetti (1980) K_0 correlation presents significant higher values than the reference values considered in this research.

Numerical studies (Yu, 2004) which assume that the insertion of the dilatometer is a flat cavity expansion process enabled a theoretical relationship between K_D and K_0 (also K_D and OCR) to be obtained. The numerical estimative of K_0 for three different clays compared to predictions obtained directly from Equation 8 showed that the Marchetti (1980) proposal can be used with reasonable confidence for the soils investigated.

(d) Overconsolidation ratio OCR

The overconsolidation ratio OCR has been usually defined as the ratio of the "maximum" past

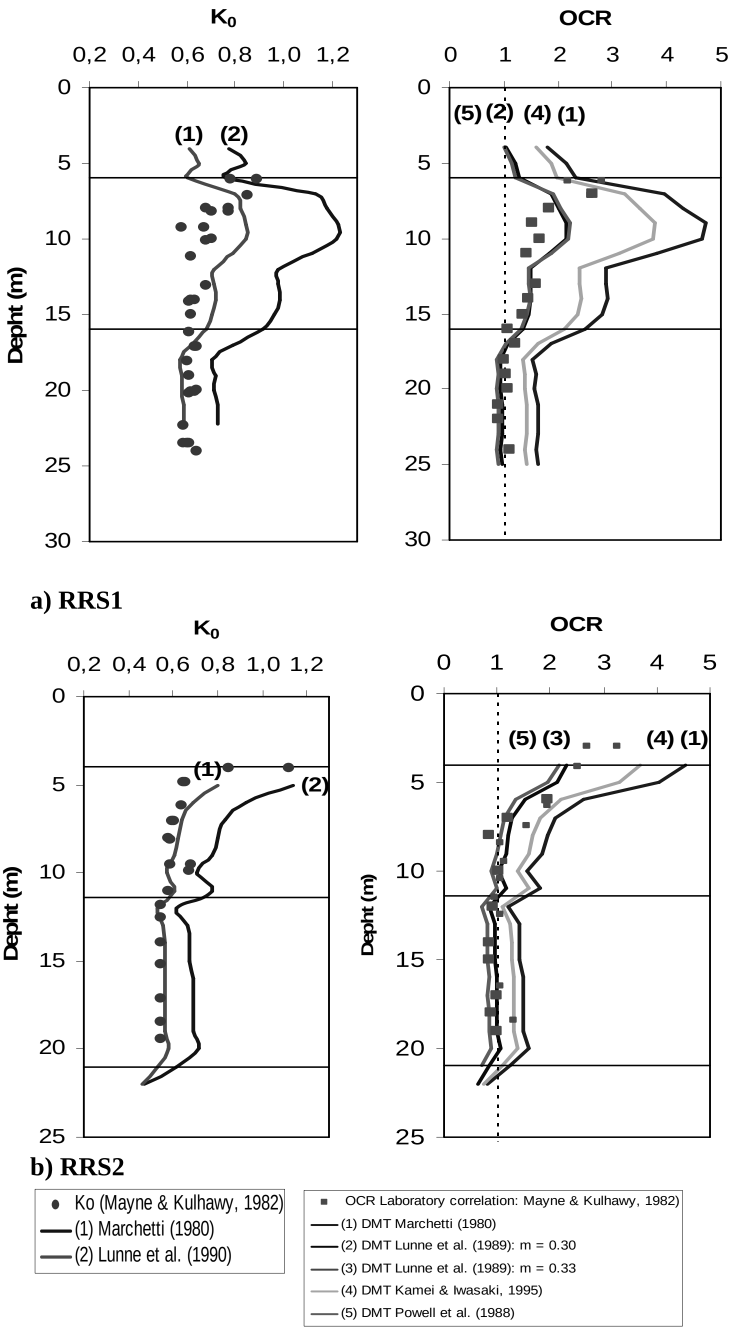


Figure 6. Stress history and in situ horizontal stress parameters: (a) Research Site 1; (b) Research Site 2.

effective stress and the currently vertically applied stress.

Marchetti (1980) pointed out the similarity between the K_D and OCR profiles and later confirmed by several authors (e.g. Jamiolkowski et al, 1988). In the present research this similarity is also very well observed with the "exception" of the upper part of the first soft clay layer in the RRS1.

For uncemented clays OCR can be simply predicted as:

$OCR = (0.5K_D)^{1.56}$ (Marchetti, 1980) (11)

Equation 11 has built-in the assumption that K_D=2 for OCR=1. This assumption has been confirmed in many genuinely NC (no cementation,

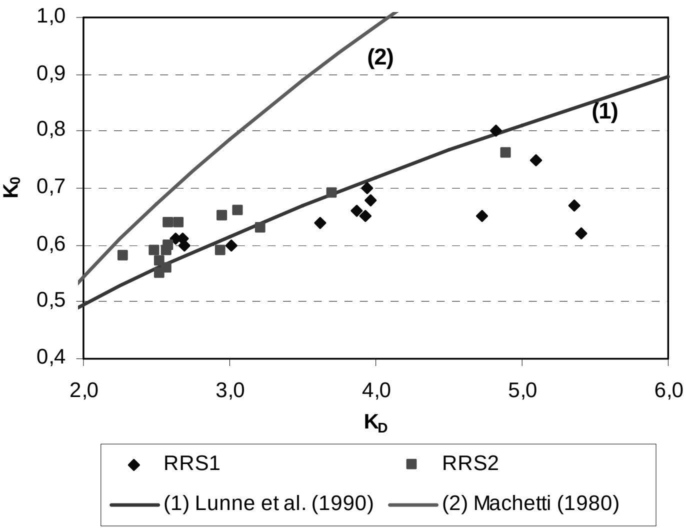


Figure 7. Coefficient of earth pressure at rest K_0 stress parameters: (a) Research Site 1; (b) Research Site 2.

aging, structure) clay deposits (Marchetti et al.,2001). In the present research OCR values were also predicted from other correlations proposed in the literature.

$OCR = m\ K_D^{1.17}$; m=0.30 – 0.33 (Lunne et al 1989) (for young clays: < 60,000 years) (12)

$OCR = (0.34\ K_D)^{1.43}$ (Kamei & Iwasaki, 1995) (13)

$OCR = 0.24K_D^{1.32}$ (Powell & Uglow, 1988) (14)

Figure 6 presents results of OCR profiles from the Recife research sites obtained using oedometer tests. Predictions of OCR from DMT correlations are shown in Figures 6 and 8.

Figure 6a, for RRS1, shows a small overconsolidated upper crust (OCR values decreasing from a value of about 3.0 to 1.3), and remaining approximately 1.3 until reaching layer 2 which is normally consolidated (OCR≈1.0). The OCR data distinguishes layer 1, which is generally overconsolidated, from layer 2, which is generally normally consolidated.

Figure 6b, for RRRS2, shows a similar pattern to that of Figure 6a for RRS1 with layer 1 having an overconsolidated crust. However, the OCR values decrease more rapidly at RRS2 (from an OCR of about 3 to a value of 1) than at RRS1. Layer 2 at both sites is normally consolidated (OCR≈1.0).

From the K_D profile (Figure 3) in both research sites the NC layer 2 (Figure 6) has $K_D \approx 2.0$ to 3.0 indicating some level of cementation/structure/aging (Marchetti et al., 2001). The values of K_D are lower at RRS2 than RRS1 indicating that the level of cementation/structure/aging at RRS2 is likely less than at RRS1.

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Figures 6 and 8 show that the correlations for OCR proposed by Lunne et al. (1989) using m = 0.30–0.33 and Powell et al. (1988) can be used with reasonable confidence in Recife soft clays. The Marchetti (1980) and Kamei & Iwasaki (1995) OCR correlations present significant higher values than the reference values considered in this research.

Numerical estimates of OCR from the theoretical relationship between K_D and OCR developed by Yu (2004) (see also Schnaid, 2005) for three different clays showed that the Marchetti correlation can be used with reasonable confidence for the clays investigated with OCR<8.

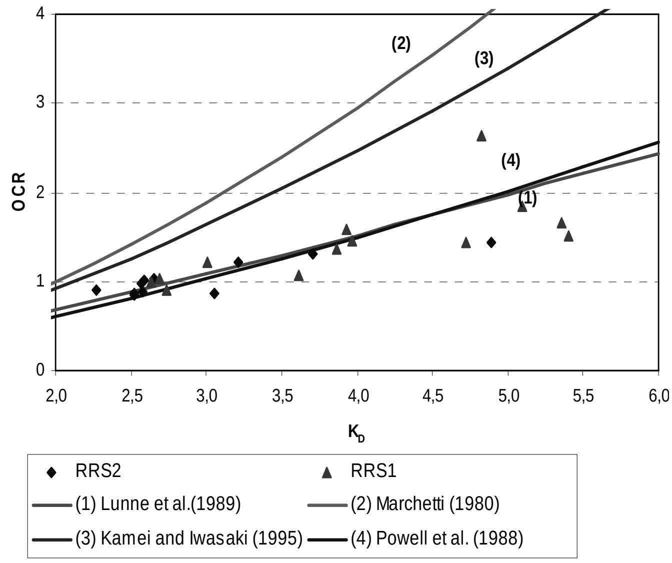

Figure 8. Stress history and in situ horizontal stress parameters: (a) Research Site 1; (b) Research Site 2.

4.2. Characteristics of Deformation

Figure 9 shows the results obtained in the research sites for the compressibility parameters from oedometer tests: void ratio (e_0), compression index (C_{C1}), swell index (C_S). They are basically constant in each soft layer with higher values in layer 1.

Constrained tangent modulus values (M) from laboratory tests and DMT tests are compared in Figure 9 at the same in situ overburden stress. The Marchetti (1980) correlation for clays (I_D < 0.6) was used:

$M_{DMT} = R_M.E_D;$ (15)

Where $R_M = 0.14+2.36 \log K_D$ (16)

The results show a very reasonable agreement in the soft layer 2 – RRS1. In the other layers, in general, M_{DMT} were slightly higher (0 - 20%) than oedometer results (RRS2 – layer 1 and 2; RRS1 – layer 1).

Lunne et al. (1989) stated that, for clays, it was recommended to use the Marchetti (1980) correlation.

Experience has shown that M_{DMT} is highly reproducible and in most cases varies between from about 0.4 MPa to 400 MPa. Comparisons both in terms of $M_{DMT} - M_{reference}$ and in terms of predicted vs. measured settlements have shown that, in general, M_{DMT} is reasonably accurate and dependable for everyday design practice (Marchetti et al., 2001).

4.3 Characteristics of flow

(a) Coefficient of horizontal consolidation

The method used in the present research for deriving C_h from DMT dissipations was the DMT-C (Schertmann, 1988; Robertson, 1989) considering a time factor (T_{30}) corresponding to t_{30} determined from the C-decay dissipation curve (Pereira, 1997).

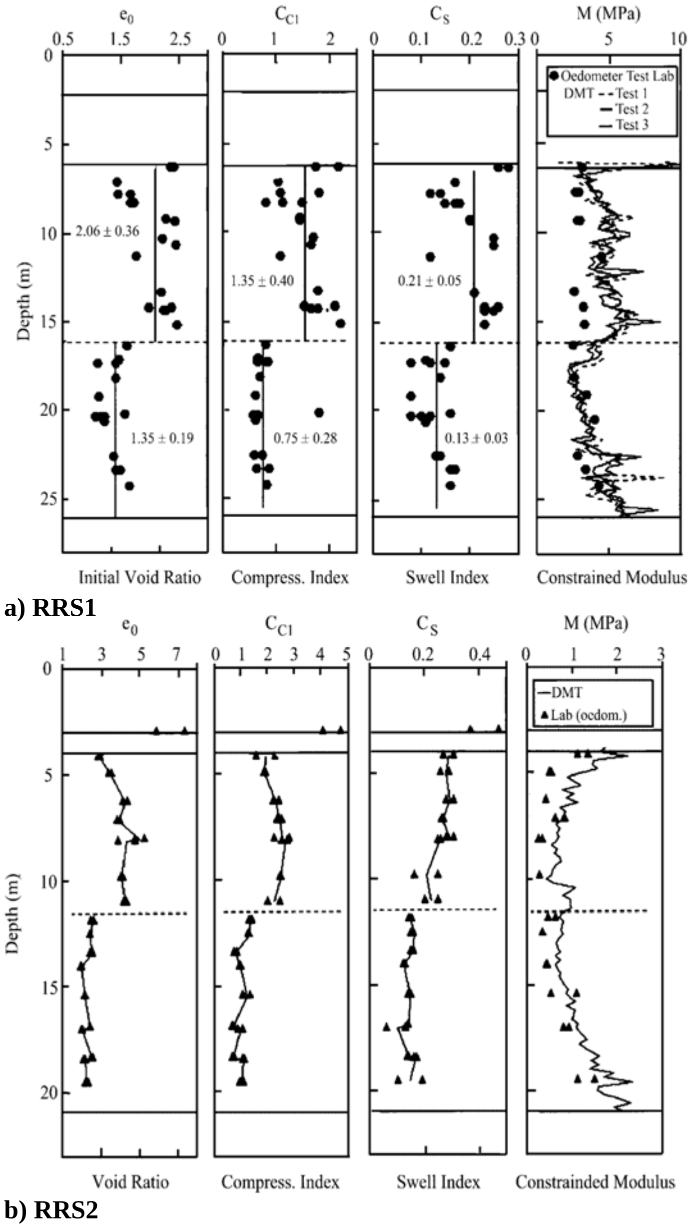

Figure 9. Compressibility parameters – oedometer tests and DMT: (a) Research Site 1; (b) Research Site 2 (Coutinho & Oliveira, 1997; Coutinho et al., 1999).

Table 1 presents the C_h values that were obtained in the RSS2. Figure 10 shows the comparison with C_v values from laboratory oedometer tests, for the depth of 7.40 m.

The DMT C_h values obtained in soundings D-1 and D-2 showed some differences at 12.4 and 17.4 meters but were similar at the depth of 7.40 m (Table 1). In general, the DMT C_h values were higher than the C_v (C_h/C_v = 1 to 3) laboratory results as was expected (Figure 10).

The method recommended by Marchetti el al (2001) for deriving C_h from DMT dissipations is the DMT-A method. Another accepted method is DMT-A_2 method that is considered basically an evolution of the DMT-C method.

Case histories indicated that the C_h from DMT-A are in good agreement (or "lower" by a factor 1 to 3) with C_h backfigured from field observed behavior (Marchetti et al., 2001).

The DMT-A_2 method (and the DMT-C method) rely on the assumption that the contact pressure A_2 (or C), after the correction, is approximately equal to the pore pressure in the soil facing the membrane. Such assumption is generally valid for soft clays, but dubious in more consistent clays. The DMT-A method does not rely on that assumption (Marchetti et al., 2001).

(b) Coefficient of horizontal permeability

Schmertmann (1988) proposes the following procedure for deriving k_h from C_h:

- Estimate M_h using $M_h = K_0 M_{DMT}$, i.e. assuming M proportional to the effective stress in the desired direction.

- Obtained $k_h = C_h\ \gamma_w/M_h$. (17)

4.4. Undrained shear strength (S_u)

In the present research the DMT Su values were predicted from the following correlations:

$S_u = 0.22\ \sigma'_{V0}\ (0.5\ K_D)^{1.25}$; (18)
(Marchetti, 1980)

$S_u = 0.20\ \sigma'_{V0}\ (0.5\ K_D)^{1.25}$; (19)
(Lacasse & Lunne, 1988)

$S_u = 0.350\ \sigma'_{V0}\ (0.47\ K_D)^{1.14}$; (20)
(Kamei & Iwasaki, 1995)

Figure 11 presents the S_u values from both research sites (RRS1 and RRS2) obtained through the dilatometer and the references tests – Vane and triaxial compression tests (UU-C and CIU-C, with $\sigma'_C \cong \sigma'_{OCT\ in\ situ}$).

Table 1. Coefficient of horizontal consolidation values from DMT – RRS2 (Pereira, 1997).

	Depth (m)	C_h ($\times 10^{-4}$ cm^2/s)
Test D-1	7.40	3.737
	12.40	12.279
	17.40	6.121
Test D-2	7.40	3.336
	12.40	4.198
	17.40	1.954

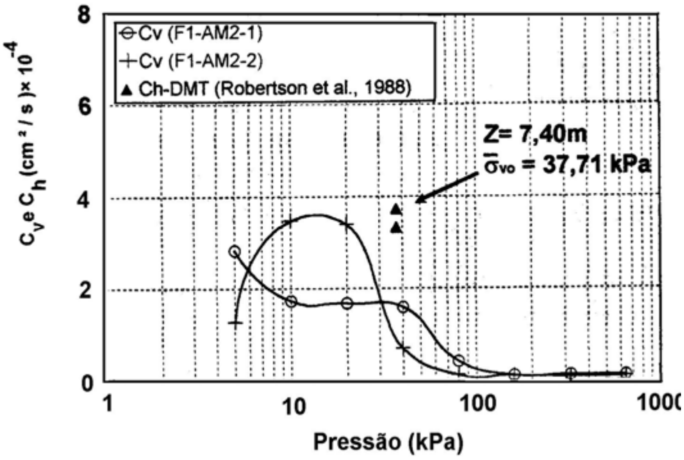

Figure 10. Coefficient of horizontal consolidation - DMT and Oedometer results - RRS2 (Pereira, 1997).

In Recife Research Site 1 (Figure 11a and 12) the Marchetti's correlation S_u values in general are close or slightly higher than the vane tests and the laboratory triaxial results. The Lacasse & Lunne (1988) correlation S_u values were in general close to the laboratory triaxial tests and lower or close to the vane tests results. The Kamei & Iwasaki (1995) correlation gave higher S_u values than both tests (Figure 12). In the Recife Research Site 2 (Figure 11b and 12) the Marchetti's correlation Su values in general are close or slightly lower than the vane tests and close or slightly higher than the laboratory tests results. The Lacasse & Lunne (1988) correlation S_u values were close or slightly lower than the triaxial compression tests and lower than the vane tests results. The Kamei & Iwasaki (1995) correlation in general presented S_u values close to the vane tests and higher than the laboratory triaxial tests results.

Marchetti et al. (2001) comments that the correlation $S_u = 0.22\ \sigma'_{V0}\ (0.5\ K_D)^{1.25}$ has generally been found to be in an intermediate position between subsequent datapoints presented by various researchers (e.g. Lacasse & Lunne, 1988; Powell & Uglow, 1988). Experience has shown that, in general, Su_{DMT} is quite accurate and dependable for design, at least for everyday practice.

Numerical analysis of the installation of flat dilatometers reported by some authors have provided useful insights of the dilatometer test and

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generally support the Marchetti (1980) empirical correlation for S_u (Schnaid, 2005).

Considering both research sites, an estimation of S_u for the Recife soft clays deposits can be obtained with reasonable confidence for practical purposes. S_u compares favorably with the vane test using the original correlation (Marchetti, 1980) and with triaxial compression test results using the correlation proposed by Lacasse & Lunne (1988).

5. Comparative study – laboratory x DMT

Table 2 shows a summary of the correlations used in the present research to obtain from DMT results for some important geotechnical parameters, OCR, K_0, S_u and M. Values of the geotechnical parameters from DMT were compared with that obtained by reference tests.

Column 5 of the Table 2 (Recife Experience) shows the results from the quantitative comparative study between the geotechnical parameters values predicted from the DMT and the results from the reference tests. In can be observed that for the Recife soft clay the estimation of geotechnical parameters is quite accurate for practical purpose from results of DMT using correlations from the literature. Column 6 presents the DMT correlations recommended to be used in the Recife soft clays deposits and the uncertainty associated with the prediction of the geotechnical parameters.

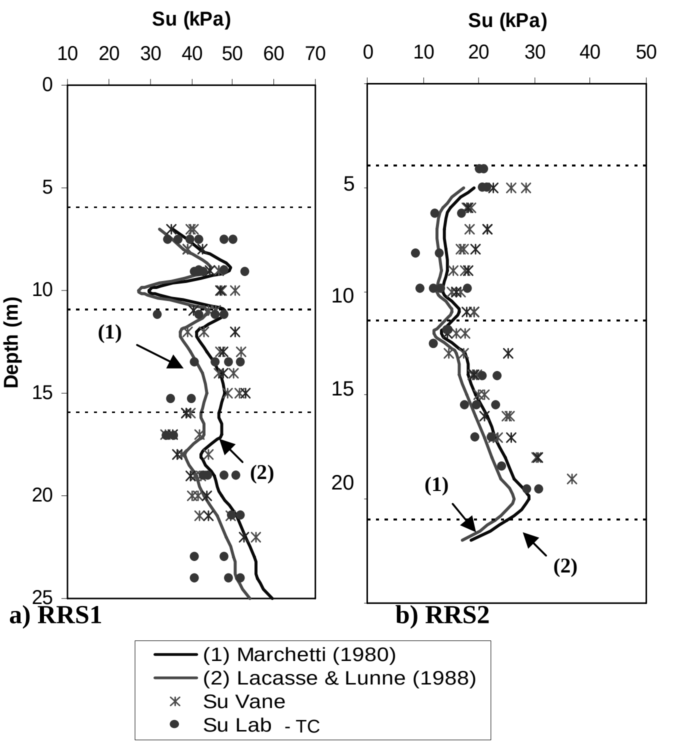

Figure 11. Su vs. depth: DMT, triaxial compression tests, and uncorrected field vane tests; (a) Research Site 1 (b) Research Site 2 (Coutinho et al., 1999).

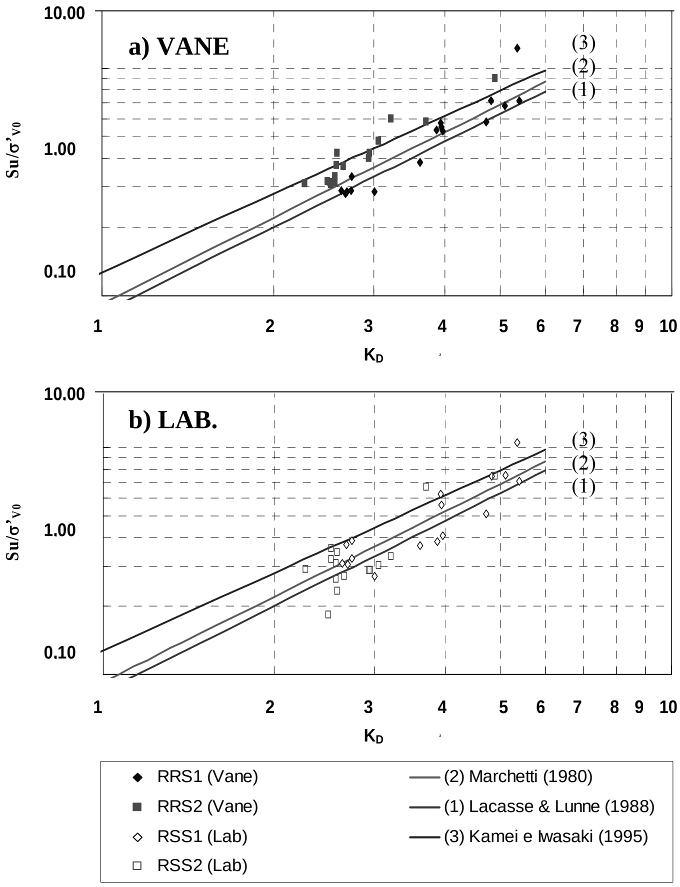

Figure 12. S_u vs. K_D parameters: (a) Vane Test; (b) Laboratory test – TC.

6. Practical application – Steel Pile Under Lateral Loading in a Very Soft Clay Deposit

In 1995, a thorough rupture in a reinforced concrete structure of a floor supported on steel piles embedded in a 17 meters thick soft clay layer in Recife, Brazil, occurred 21 years after it had been built, with no warning of potential failure.

Figure 13 presents the geotechnical profile of a cross section of the area and the hypothesis proposed for the accident. A slow lateral movement of the organic clay layer provoked lateral displacement of the piles which were supporting the total vertical load (structure self weight + negative friction) causing a buckling failure. This case demonstrates the importance of a buckling study in steel piles caused by lateral displacement in soft soil.

Afterwards, the Geotechnical Group of the Federal University of Pernambuco, Brazil, has performed extensive geotechnical research program in the area (UFPE - RRS2).

A study was developed on the behavior of laterally loaded steel piles in thick layers of soft clay, consisting of analytical and experimental stages (Coutinho et al, 2005). 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 and also for the buckling load of a steel pile in very soft clay were made from linear and non-linear analyses through the finite element method.

The soil was modeled with p-y curves obtained from dilatometer (DMT) and Ménard pressuremeter (PMT) testing results performed at the site of the accident and near the damaged structure that bear deforming-power element. The following assumptions were considered: the steel pile was perfectly vertical and steel pile had vertical load eccentricity, that is, with initial lateral deformation. The p-y curves were found through the semi-empirical method proposed by Robertson et al. (1989), which uses data from dilatometer tests, and for the semi-empirical method proposed by Ménard (1969), which uses data from pressumeter tests.

Table 2. Comparative study – DMT correlations versus reference tests

PARA-METER	CORRELATIONS - DMT	EQUATIONS	REFERENCE TEST	RECIFE EXPERIENCE	CORRELATION RECOMENDED
OCR	**Lunne et al.(1989)**	$OCR = m K_D^{1.17}$; $m = 0.3\text{-}0.33$ (young clays: < 60.000 years)	**oedometer**	± 10%	**Lunne et al (1989)** $OCR = 0.3 K_D^{1.17}$ $m = 0.30\text{-}0.33$ ±10%
	Marchetti (1980)	$OCR = (0.5 K_D)^{1.56}$ (uncemented clays) ($I_D \leq 1.2$)		40 – 160% (average) 80% (higher)	
	Kamei & Iwasaki (1995)	$OCR = (0.34 K_D)^{1.43}$		10 – 120% - average 55% (higher)	
	Powell et al. (1988)	$OCR = 0.24 K_D^{1.32}$		± 15%	
K_0	**Lunne et al.(1990)**	$K_0 = 0.34 K_D^{0.54}$ (young clays: < 60.000 years)	$K_0 = (1 - \text{sen}\,\phi') OCR^{\text{sen}\,\phi'}$ **(Mayne & Kulhavy, 1982)**	± 10%	**Lunne et al (1989)** $K_0 = 0.34 K_D^{0.54}$ ± 10%
	Marchetti (1980)	$K_0 = (K_D / 1{,}5)^{0.47} - 0.6$		40% (higher)	
Su	**Marchetti (1980)**	$Su = 0.22\ \sigma'_{V0}\ (0.5\ K_D)^{1.25}$	**(Triaxial) UU-C / CIU-C**	± 20%	**VANE TESTS Marchetti (1980)** $Su = m\ \sigma'_{V0}\ (0.5\ K_D)^{1.25}$ $m = 0.22 \pm 0.03$ ± 15%
			(Vane)	± 15%	
	Lacasse & Lunne (1988)	$Su = 0.20\ \sigma'_{V0}\ (0.5\ K_D)^{1.25}$	**(Triaxial) UU-C / CIU-C**	± 15%	**TRIAXIAL TESTS Lacasse & Lunne (1988)** $Su=0.20\sigma'_{V0}(0.5\ K_D)^{1.25}$ ± 15%
			(Vane)	± 18%	
	Kamei & Iwasaki (1995)	$Su = 0.350\ \sigma'_{V0}\ (0.47\ K_D)^{1.14}$	**(Triaxial)**	± 30%	
M	**Marchetti (1980)**	$M= R_M \cdot E_D$; with $R_M = 0.14 + 2.36 \log K_D$; ($I_D < 0.6$)	**oedometer**	0 - 20% (higher)	**Marchetti (1980)** 20% (higher)

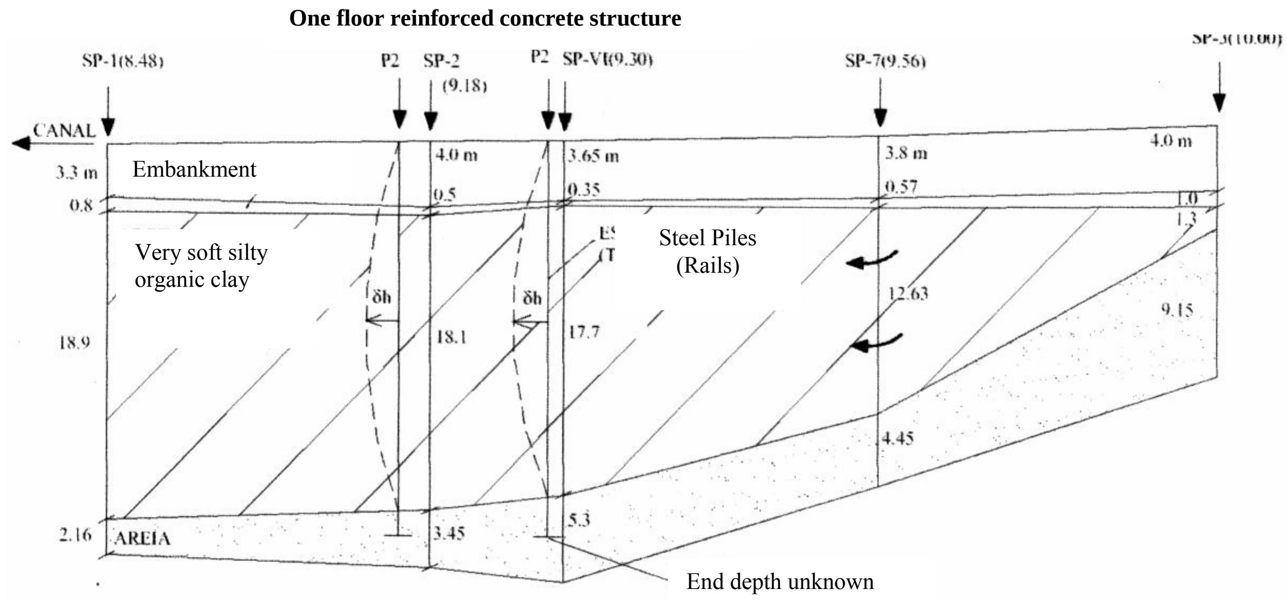

Figure 13. Geotecnical profile – horizontal pull

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The horizontal displacements were measured (inclinometer) and predicted with linear and nonlinear FEM analyses for level land grades and after fill excavation. Figure 14 and Table 3 presents the results obtained versus the applied loads. 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 % to 20 %.

In the analysis for the collapse of the steel piles, two important facts must be taken into consideration: a) whether the steel pile was completely vertical and; b) whether there was any eccentricity in the vertical load.

It was assumed that the steel pile suffered horizontal displacements and showed 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 (1989) are summarized in Table 4.

Table 3. Predicted and measured displacements (Coutinho et al., 2005)

H = (kN)	2.5	5.0	7.5	10.05
	Horizontal Displacements (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
Measured	18.59	27.86	68.71	109.65

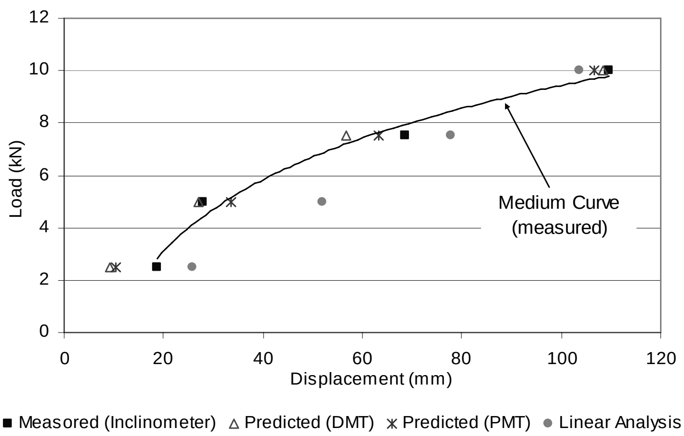

Figure 14. Predicted and measured displacements (Coutinho et al., 2005)

It can be observed that the critical load is considerably sensitive to the effect of accidental displacements which rapidly decreases its value.

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. As shown in Table 4 the working load for the steel pile would be 186.5kN and was within the interval which determines the occurrence of failure corresponding to an accidental displacement between 30 and 50cm.

Table 4. Critical loading due to accidental displacements (Coutinho et al., 2005)

Defor-rmattion (mm)	Critical Loading (kN)	
	Curves P-Y (DMT)	Curves P-Y (PMT)
	Free / Labeled Top	Free / Labeled Top
0	2,988.64	1,925.11
10	1,738.18	1,877.44
20	1,183.99	510.21
30	360.29	98.86
50	58.19	70.89
100	46.25	56.90

7. CONCLUSIONS

The flat dilatometer test has been extensively used and calibrated in soil deposits all over the world. An extensive and carefully planned investigation performed in Recife soft clays confirms the important potential of the DMT in the determination of soil type, geotechnical parameters and application for laterally loaded steel pile analyses.

The DMT correlations are recommended to be used in Recife soft clays deposits for geotechnical design parameters (uncertainty associated ≤ 20%).

The predicted lateral displacements obtained from the nonlinear analysis by using p-y curves obtained from DMT tests closely match the results measured with the lateral load test.

Lateral displacements drastically reduce the vertical loading capacity of a steel pile in soft clay deposits, as can be observed through the nonlinear analysis, making possible the occurrence of a buckling failure.

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