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Ménard and Cambridge selfboring pressuremeters: Correlations between mechanical parameters in Lisbon Miocene clayey soils

Pressiomètres Ménard et l'autoforeur de Cambridge: Corrélations entre des paramètres mécaniques des argiles du Miocène de Lisbonne

António G.F. de Sousa Coutinho & M.A. Ludovico Marques
Laboratório Nacional de Engenharia Civil, Lisboa, Portugal

ABSTRACT
This paper presents a mechanical characterization of Miocene overconsolidated clays of the Lisbon subsoil, carried out by pressuremeter tests. These tests were run in the scope of the enlargement of the Lisbon underground net. The tests were performed with a Ménard pressuremeter and a Cambridge selfboring pressuremeter, both with a pressure capacity of 10 MPa.
The main results are presented in this paper. Among the results achieved, the coefficient of earth pressure at rest was computed, as well as the increasing law with depth of deformability and resistance parameters. In the end, a correlation between the Ménard limit pressure and the undrained shear strength was established. It was found that the more common correlations between these two parameters published in the literature underestimate the undrained shear strengths. However, some of those correlations were established from tests where the limit pressures were at least 2 to 3 times smaller than those obtained in present study.

RÉSUMÉ
Ce document présente une caractérisation mécanique d'argiles sur-consolidées du Miocène du sous-sol de Lisbonne, qui a été effectuée moyennant des essais pressiométriques. Ces essais ont été menés dans le contexte des travaux d'ampliation des lignes du Métro de Lisbonne, avec des pressiomètres Ménard et un autoforeur de Cambridge, en ayant les deux une capacité pour appliquer des pressions de 10 MPa.
Les résultats essentiels sont présentés dans cet article. On a déterminé le coefficient d'impulsion en repos, ainsi que la loi de variation avec la profondeur des paramètres de déformabilité et de résistance. Finalement, on a établi une corrélation entre la pression limite Ménard et la résistance au cisaillement non drainé. On a vérifié que les corrélations les plus communes entre ces deux paramètres, publiées dans la littérature, sous-estiment largement la résistance au cisaillement non drainé. Cependant, quelques corrélations ont été établies ayant pour base des essais où les pressions limites atteintes étaient d'au moins 2 à 3 fois inférieures à celles obtenues dans cette étude.

1 INTRODUCTION

In the nineties of the past century, several in situ and laboratory tests were carried out on Miocene soils of the subsurface of Lisbon in the scope of the underground net extension. Among the in situ tests, pressuremeter tests (PMT), with a Ménard pressuremeter (MPM) and a Cambridge selfboring pressuremeter (CSBP), were carried out at three sites:

- Chiado hill, at Lisbon downtown;
- Alameda terminal station of the Lisbon EXPO'98 underground line; and
- Yellow line, between Campo Grande and Ameixoeira.

In the whole, 100 MPM and 70 CSBP were successful carried out. These tests were performed in the four most frequent Miocene units of the Lisbon subsoil, but in this paper only the eldest unit will be referred. This is the "Argilas e Calcários dos Prazeres", which may be translated by "Clays and Limestones of Prazeres". In this unit, 21 MPM and 13 CSBP were carried out, at depths between 15 and 45 meters, at Chiado hill and at Alameda sites.

This paper is developed on the results obtained in those tests, and its aim is to establish the coefficient of earth pressure at rest and a relationship between the MPM limit pressure and the CSBP undrained shear strength for the "Clays and Limestones of Prazeres" unit. The value of the coefficient of earth pressure at rest, as well as a relationship between the MPM limit pressure and the undrained shear strength is very useful in practice, since MPM tests are cheaper than CSBP tests, and the former are easier to carry out than the latter.

With this study, it is believed that a better knowledge about this unit has been achieved.

2 GEOLOGICAL DESCRIPTION OF THE SITES

Complete descriptions of the unit "M^1_I – *Clays and Limestones of Prazeres*" at the sites where tests were carried out is given by Ludovico Marques (1997). A brief summary is presented here.

The unit *Clays and Limestones of Prazeres* is the eldest unit of Lisbon Miocene as it belongs to Aquitanian.

At Chiado hill, this unit comprises greyish clay layers with fractures, overlaying a layer of greyish and greenish silty and marly clays, with carbonized fragments and pyrite minerals, which in turn overlays a uniform sand layer. The unit also contains boulders and benches of calcareous sandstones alternating with marly limestones, which have a fracture intercept of $F_{4\text{-}5}$ and F_5 (ISRM, 1981). These beds reach a thickness of 50 meters, and sometimes even more. N_{SPT} were in a range between 10 and 60 blows. These values include results where the 60 blows were reached, in the second stage of the test, with a penetration of only 10 cm in marly clays, in benches of calcareous sandstones and in marly limestones. Plasticity indexes obtained from Atterberg limits were about 10 to 30%. RQD (Deere, 1967) values were in the range of 10% (marly clays) to 75% (calcareous sandstones and marly limestones).

At Alameda site, this unit comprises layers of greyish and yellowish silty and sandy clays, and layers of clayey sands containing carbonized fragments, with intercalations of levels of boulders of calcareous sandstones. There were also benches of calcareous sandstones, with a weathering classification of $W_{4\text{-}5}$, W_3 (ISRM, 1978, 1981), and a fracture intercept of $F_{4\text{-}5}$ e F_3 (ISRM, 1981). This unit reaches more than 50 meters in thickness. N_{SPT} values were about 60 blows in marly clays, calcareous sandstones and marly limestones.

These geotechnical materials lay in the range of hard soils to soft rocks.

3 THE CONFIGURATIONS OF THE PRESSUREMETERS

A digital model of CSBP (Mark IXd), with six displacement arms and a pressure capacity of 10 MPa, was introduced in Portugal to carry out tests in Miocene soils of the Internal Road Ring of Lisbon (Sousa Coutinho et al., 1996). At this site, the probe was used in its classical configuration in the beginning, but soon it become evident that the weak rock configuration should be used. This comprises a strong rubber membrane and a *rock roller bit* of 73 mm, or even 82.5 mm for dense sands, in diameter. Later, more tests were carried out in subway tunneling for the Lisbon underground at Chiado hill and Alameda sites. However, in the former site it was found that even the weak rock configuration was not enough to ensure the success of the tests, if the probe had to drill boulders of calcareous sandstones. As boulders blocked the roller bit, it was not always possible to reach the desirable depth. Another strategy was then adopted. When there were not soundings, or any other data, that allowed the identification of the depth and the thickness of those calcareous sandstones benches in detail, two boreholes were drilled in every site using destructive techniques without sampling. The first borehole provided geological data, and it was also used to carry out MPM tests. The second one was used to settle the CSBP probe at a desirable depth, from where on it drilled by selfboring process up to the test depth, avoiding the undesirable boulders and benches.

The MPM model used was a GA/GC manufactured by APAGEO, with a pressure capacity of 10 MPa. The pressure/volume control unit had a data logger (SPAD in French). The probe of 60 mm in diameter was used in first place. Although a chinese lantern protected the membrane, there were several membrane bursts, which implied the change to the 44 mm probe with the protection of the stainless steel strips sheath. The pockets for tests were drilled with a *rock roller bit* of 62.5 mm in diameter for the 60 mm probe and with a rock roller bit of 60.3 mm for the second configuration which external diameter is 56 mm.

4 TEST RESULTS

4.1 *MPM tests*

Table 1 shows the results of the 21 MPM tests carried out. The interpretation was made according to the French norm P 94–110 (21st July 1991).

4.2 *CSBP tests*

Table 2 shows the results of the 13 CSBP tests carried out. As the CSBP configuration was prepared for hard soils (weak rock configuration), there was a little over excavation of the pressuremeter pocket. However, this is only 0.8% of the cavity extension. Sousa Coutinho (1996) discusses in detail the effects of this over excavation on the interpretation of the tests. The σ_{h0} value was computed by the Marsland and Randolph modified method (Hawkins, et al., 1990).

The G_{ur0} values were computed from an unload-reload cycle performed at a low level of cavity expansion, usually less than 0.7%. They can be taken as the initial elastic shear moduli, G_i or G_0, since they are computed at a very small cavity expansion.

The c_u values were computed form the Gibson and Anderson (1961) equation,

$$c_u = \frac{\mathrm{d}\, p}{\mathrm{d}\, \ln(\Delta V/V)} \qquad (1)$$

taking pressures and deformations typically above 6% of cavity expansion. Although the stiff clays are strain-softening materials, and Gibson and Anderson theory is strictly applicable to linear elastic–perfect plastic materials, it should be noted that at large strains the shear stress tends for a constant value and tends to be independent of the disturbance effects of the installation.

The limit pressures were obtained by Ménard (1957) expression, for the theoretical limit pressure, i.e., when expansion is infinite:

$$p_l = \sigma_{h0} + c_u \times [1 + \ln(G/c_u)] \qquad (2)$$

Table 1: Results of the MPM tests

z (m)	E_M (MPa)	p_l (kPa)
16.6	64	4770
18.5	67	3520
19.5	77	5920
20.5	55	6000
22.5	71	4210
22.5	91	5170
26.0	73	5220
28.0	43	4380
30.5	37	7000
31.0	63	5270
32.0	150	7620
32.5	96	8950
33.0	110	6490
34.5	150	7220
35.0	170	10210
36.5	160	8850
37.0	130	9420
38.5	120	9920
39.0	200	8720
42.5	120	12530
44.5	130	9090

Table 2: Results of CSBP tests

z (m)	σ_{h0} (kPa)	G_{ur0} (MPa)	c_u (kPa)	p_l (kPa)
16.5	345	182	944	6260
18.5	211	273	611	4550
20.1	303	(402)	1286	8980
21.4	268	246	1050	7050
25.0	296	(592)	1367	9960
25.5	390	302	1001	7110
26.0	323	235	1177	7730
29.0	345	(128)	1378	7970
29.0	496	279	1008	7170
29.5	326	229	1035	6950
35.0	597	278	1117	7880
36.0	519	393	1630	11090
40.1	512	338	2236	13970

5 INTERPRETATION

5.1 *Type of soils*

Table 3 shows some statistical values of E_M/p_l obtained from the MPM tests for Chiado hill and Alameda sites. Comparison of these values with those of Ménard and Rousseau (1962), shows that the soils are overconsolidated, as expected. The soils at Chiado hill are clearly clayish, but at Alameda site they seem to be somewhat siltier, or else, somewhat less overconsolidated, than those at Chiado hill. A study carried out by Marques et al. (1996) on samples obtained in a borehole at Alameda site showed, nevertheless, that the soil at that particular place was a CL. It should be, however, pointed out that these units of Lisbon have very different "facies" from one point to another, even at small distances between them.

Table 3: Statistics of E_M/p_l values

Site	n	X_m	s	VC (%)
Chiado	8	16	4.3	27
Alameda	13	14	4.6	33
Both	21	15	4.5	30

5.2 *Estimation of K_0*

From the total horizontal stresses at rest, obtained from CSBP tests, it is possible to compute the K_0 value, provided the unit weights of overburden soils and ground water level are known. Tests carried out on undisturbed samples showed that the average unit weight of this unit soils, and those of the overburden units, have a medium value of 20 kN/m^3. No groundwater levels were detected in the boreholes, although the study carried out by Marques et al. (1996) showed that the clays were almost saturated.

Figure 1 shows the increase with depth of σ'_{h0}. A straight line fitted to data gives σ'_{h0}=14z. As σ'_{v0}=20z, K_0=0.7.

This result is in accordance with the values obtained in K_0 tests in triaxial chambers carried out at LNEC on undisturbed samples.

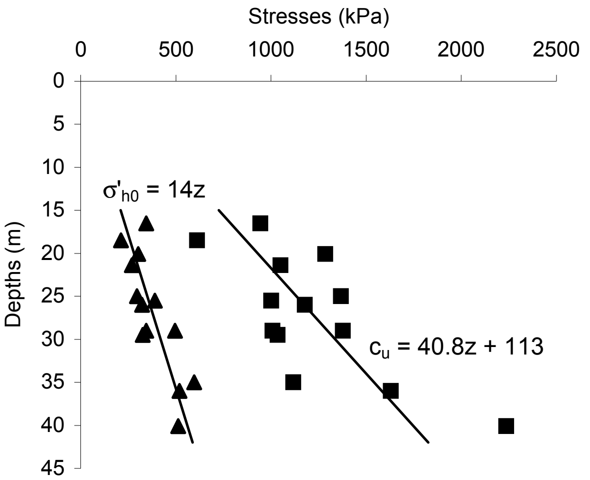

Figure 1. Horizontal stresses and undrained shear stresses

5.3 *Undrained shear strength*

From CSBP tests, the increasing in depth of c_u can be computed, as figure 1 shows. It is also possible to compute the ratio between the undrained Young modulus, E_u, and the undrained shear strength, since E_u=3×G_u. Table 4 gives the main statistical values; it is noted that the range of the values is in accordance with published experience.

Table 4: Main statistics of E_u/c_u ratio

Min	Max	X_m	s	VC (%)
280	1340	774	300	39

5.4 *Deformation parameters*

Comparison between deformation parameters obtained from both pressuremeter tests cannot be directly achieved. Firstly it is necessary to convert either G_i to E_i or E_M to G_M. The latter is taken since the cavity expansion yields a G value. Ménard converted the G obtained in the pseudo-elastic phase of the test in an E_M assuming ν=0.33 for all soils. It follows that

$$G_M = E_M / 2{,}66 \qquad (3)$$

The depth distribution of G_i and G_M is shown in figure 2. The values in brackets of G_{ur0} shown in table 2 were disregarded, since they tend to distort the distribution in depth of G.

From the two equations shown in figure 2, a relationship between G_i and G_M can be established assuming several values for z in both equations. Taking into consideration equation (3), G_i can be estimated from E_M as follows:

$$G_i = 1{,}45\, E_M + 140 \qquad (15 < z < 45\ m) \qquad (4)$$

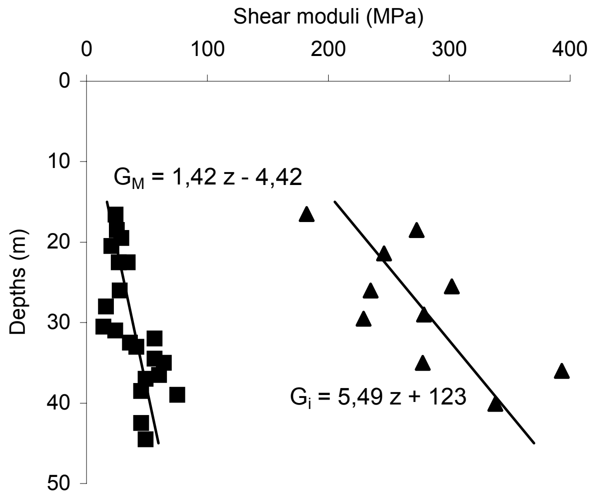

Figure 2. Comparison between G_M and G_i (CSBP)

5.5 *Comparison between the limit pressures obtained by both pressuremeters*

The limit pressures obtained by both pressuremeters are shown in figure 3. It is noted that the values computed from CSBP tests are greater than those computed by MPM tests. Fitting a straight line to both p_l distributions, it is verified that increasing rate with depth of both p_l is almost the same, and differences are only in the origin constant. This difference might be explained by different definitions of p_l. Whilst the MPM p_l is the pressure necessary to duplicate the initial volume of the cavity, the CSBP p_l is the pressure at which the cavity expands to infinite.

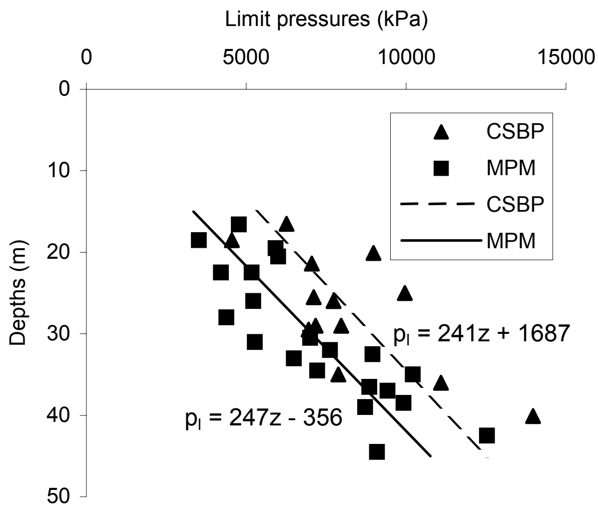

Figure 3. Comparison between MPM and CSBP limit pressures

5.6 *Relationship between p_l (MPM) and c_u (CSBP)*

The relationship between MPM p_l and CSBP strength parameter c_u, can be readily achieved, using the expressions already computed for p_l, c_u and σ_{h0}. Taking values at some depths for those expressions, one has:

$$c_u = \frac{p_l - \sigma_{h0}}{5.7} + 175 \qquad (15 < z < 45\ m) \qquad (5)$$

In the literature there are some proposals for relations between p_l and c_u. Clarke (1995) lists several, and comparing the results between them and the expression achieved in this work it is noted that all, except one, yield lower values for c_u. Figure 4 shows these relations. It should be noted that those expressions clearly underestimate the values of c_u obtained directly by CSBP tests (table 2). Although the authors are unaware of the data used for setting up all those expressions, some of them (e.g. Amar and Jézéquel, 1972) were established from MPM p_l clearly smaller than those obtained in this study. This might explain why those relations cannot predict c_u for such clays.

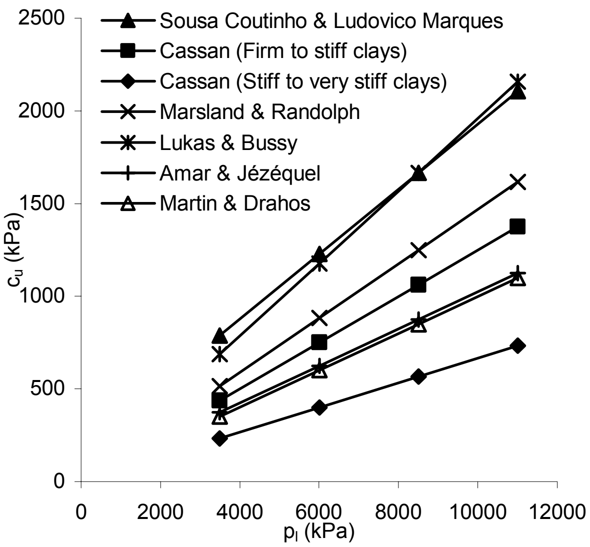

Figure 4. Comparison of several relations between p_l e c_u

A final remark on this subject is that, according to Clarke (1995), the strengths computed form CSBP tests are equal to those on triaxial tests on stiff clays. It follows that c_u values computed from equation (5) can be used as a first approach in a problem where MPM tests have been carried out in this geological unit. However, care should be taken, and additional tests have to be performed, since there is a considerable scatter in data of this study.

6 SUMMARY AND CONCLUSIONS

This study has presented the results of pressuremeter tests carried out in the eldest Miocene unit of Lisbon, *Clays and Limestones of Prazeres*.

The results enabled to establish a coefficient of earth pressure at rest, a correlation between the Ménard modulus E_M and the initial (tangent) shear modulus G_i, derived from CSBP tests, and a correlation between the Ménard limit pressure p_l and the undrained shear strength c_u, computed from CSBP tests. These correlations are very useful in practice, since they allow estimating as a first step the undrained elastic shear modulus and the undrained shear strength from MPM tests, of the unit *Clays and Limestones of Prazeres*. However, it should be pointed out that it is always necessary to carry out further tests to do a better mechanical characterization, since the study presented herein is based only on 21 MPM tests and 13 CSBP tests, and there is some scatter in data.

With this study it is believed that an improvement in the knowledge of the mechanical properties of the unit *Clays and Limestones of Prazeres* has been achieved.

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REFERENCES

Amar, S. and Jézéquel, J.F. 1972. Essais en place et en laboratoire sur sols cohérents, comparaison des résultats. *Bull Liaison des LPC*, Paris, No. 58, pp 97–108.

Clarke, B.G. 1995. *Pressuremeters in Geotechnical Design*. Blackie Academic & Professional.

Deere, D.U. 1967. Geological considerations. *Rock Mechanics in Engineering Practice*, 1st chapter, Stagg & Zienkiewicz, London.

Gibson, R.E. and Anderson, W.F. 1961. In situ measurement of soil properties with the pressuremeter. *Civil Engineering and Public Works Review*, Vol. 56, nº 658, May, pp. 615–618.

Hawkins, P.G., Mair, R.J., Mathieson, W.G. and Muir-Wood, D. 1990. Pressuremeter measurement of total horizontal stress in stiff clay. *Proc. 3rd Int. Sym. On Pressuremeters*, Oxford, Thomas Telford, pp. 321–330.

ISRM 1978. Suggested methods for the quantitative description of discontinuities in rock masses. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr.* Pergamon Press. Vol. 15, pp. 319–368.

ISRM 1981. Basic geotechnical description of rock masses. *Int. J. Rock Mech. Min. Sci. & Geomech. Abstr*. Pergamon Press. Vol. 18, pp. 85–110.

Ludovico Marques, M.A. 1997. *Teoria e prática do pressiómetro autoperfurador: Novos desenvolvimentos em solos rijos e rochas brandas*. Master Thesis, UNL, Lisboa.

Marques, F.E.R., Langton, D.D., Furtado, R.J.A. e Almeida e Sousa, J.N. 1996. Determinação das características de compressibilidade e de consolidação das "Argilas dos Prazeres". *6th Geothecnical Portuguese Congress*, Vol I, pp. 179–188.

Ménard, L. 1957. Mesures in situ des propriétés physiques des sols. *Annales des Ponts et Chaussées*, Paris, No.14, pp 357–377.

Ménard, L. and Rousseau, J. 1962. L'évaluation des tassements – Tendences nouvelles. *Sols–Soils* No.1 pp 13–28.

Sousa Coutinho, A.G.F. 1996. A utilização do pressiómetro autoperfurador. *Seminário sobre aplicação de ensaios in situ no dimensionamento de fundações*. IST, Lisboa.

Sousa Coutinho, A.G.F., Ludovico Marques, M.A., Costa, A. and Veiga, J. 1996. *Ensaios com pressiómetros autoperfurador no local de construção do túnel do Grilo da CRIL.* Relatório, LNEC.