In situ tests in Brasília porous clay

J.A.R. Ortigao, R.P. Cunha, and L.S. Alves

Abstract: An in situ testing programme was carried out in 1992 aimed at obtaining design parameters for the construction of the Brasília Underground line, Brazil. The top layer of soil consisted of an unsaturated and collapsible soft porous clay layer 5–30 m thick followed by residual soils from slate and interlayered metasiltstones and quartzites. A series of Marchetti dilatometer (DMT) logging tests results were compared with Ménard pressuremeter (PMT) and horizontal plate loading (PLH) tests, as well as laboratory tests on block samples. In situ stresses, strength, and deformation parameters were obtained for the porous clay. The DMT yielded very good results: excellent repeatability, low cost, and results that agree with other in situ tests and laboratory data.

Key words: porous clay, in situ testing, dilatometer, pressuremeter.

Résumé : Un programme d'essai in situ a été réalisé en 1992 dans le but d'obtenir les paramètres de calcul pour la construction du métro souterrain de Brasília, Brésil. La couche supérieure de sol consistait en une couche d'argile poreuse molle non saturée et effondrable de 5–30 m d'épaisseur suivie de sols résiduels allant d'ardoises à des intercouches de métasiltstones et quartzites. Une série de résultats d'essais de profilage au dilatomètre Marchetti (DMT) sont comparés aux essais de pressiomètre Ménard (PMT) et de chargement de plaque horizontale (PLH), de même qu'aux essais de laboratoire sur des blocs d'échantillons. Les contraintes in situ et les paramètres de résistance et de déformation ont été obtenus pour l'argile poreuse. Le DMT a donné de bons résultats : excellente répétitivité, faible coût, et résultats qui sont en accord avec les autres essais in situ et données de laboratoire.

Mots clés : argile poreuse, essais in situ, dilatomètre, pressiomètre.

[Traduit par la rédaction]

Introduction

"The data must be wrong" wrote Terzaghi to Vargas back in the 1940s when they both analysed data of newly discovered soil from São Paulo (Vargas 1994). This deposit was to be called porous clay. Then Terzaghi came to São Paulo, as he did many times during hif life, and could see for himself the *impossible* porous clay.

The work presented herein is also about an oddly behaved soil, the porous clay from Brasília, located during an investigation connected with the Brasília Underground. A field investigation programme was carried out to obtain design parameters for tunnel design using boreholes and logging tests such as the standard penetration tests (SPT), Marchetti dilatometer tests (DMT), and piezocone tests (CPTU). These empirical tests were calibrated against laboratory and in situ tests such as the Ménard pressuremeter (PMT) and horizontal plate loading (PLH) tests. The PMT tests were analysed through a curve-fitting technique comparing the field expansion curve with a theoretical one generated by the Carter et al. (1986) theory. A series of triaxial and oedometer tests were carried out on block samples obtained from test pits excavated above the water level.

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This paper summarizes the results of the in situ stress ratio K_0, deformation moduli, and strength parameters, and compares in situ and laboratory test data.

Geology and site conditions

Regional geology and geomorphology have been described in detail by Macedo et al. (1994). The region is flat, as is characteristic of the central plateau highlands. It is covered by a layer of Latosols and lateritic soils named porous clay, overlying residual soils from slate or a sequence of interlayered metasiltstones and quartzites, which geologists call metarhythmites. They are named Paranoah formation of the upper Precambrian.

The climate alternates from a 6 month rainy season to a very dry winter, leading to laterization processes of leaching soluble salts at the top of the porous clay and depositing them below. This process is responsible for the large number of pores at the top of the clay layer, resulting in high void ratios, low unit weights, and high permeability.

This study was located in the south of Brasília where a 10 m diameter tunnel was excavated (Ortigao and Macedo 1993). The soil profile was initially investigated by boreholes at 30 m intervals along the tunnel line in which SPT's were carried out at every metre depth. The porous red clay is 8–30 m thick; the SPT index is low, varying from 2 to 3. The water level is generally very deep, except at the tip of the south wing (see Fig. 1) where it is found at 8–10 m depth only. High seasonal variation of the water level, due to the high permeability of the porous clay, is another characteristic of the soil conditions.

Fig. 1. Soil profile at the south wing of Brasília. Numbers preceded by the letter S represent stations.

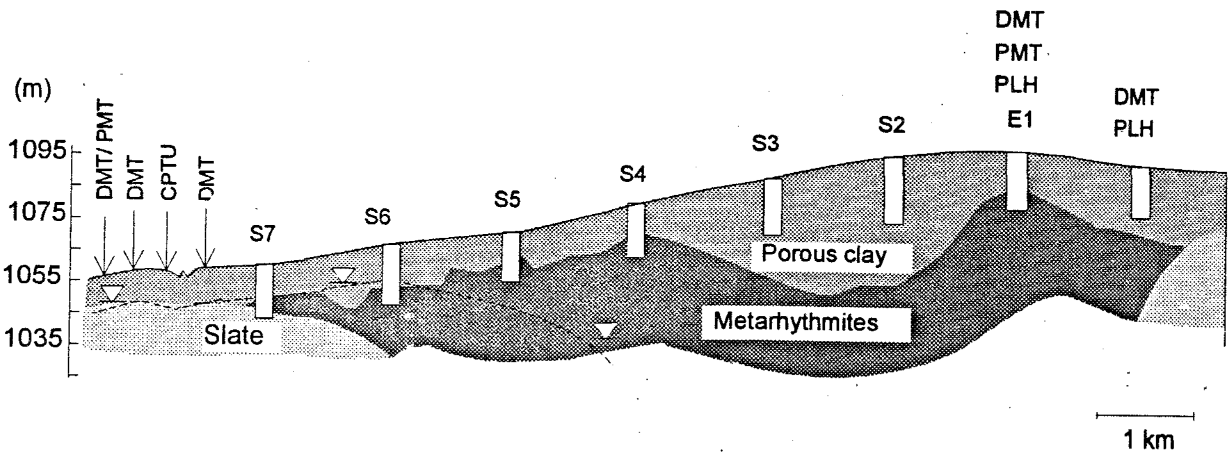
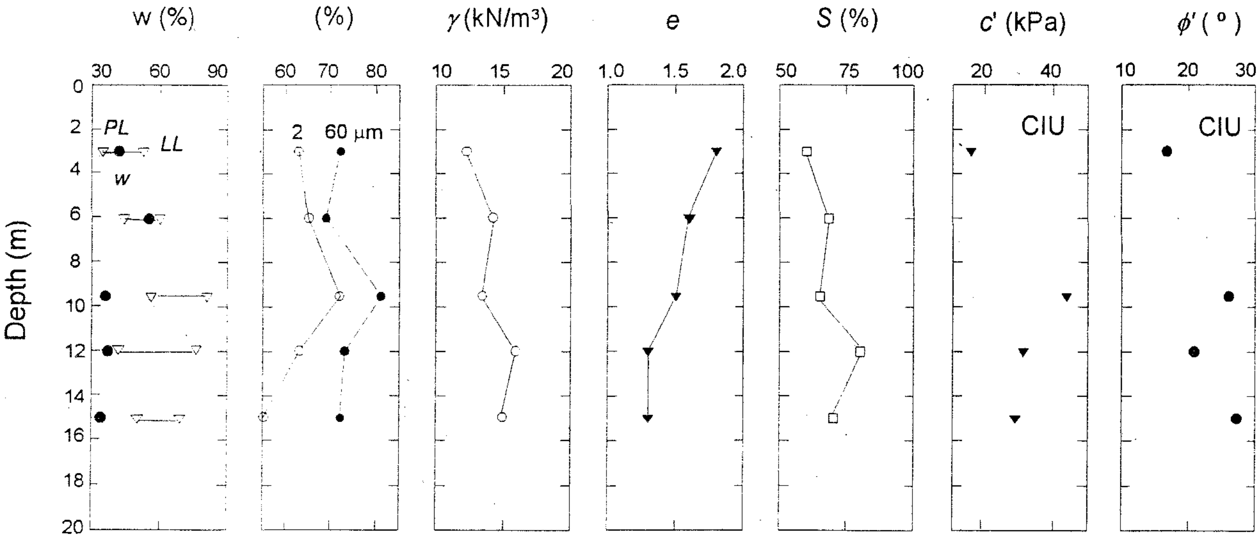


Fig. 2. Summary of laboratory test data. *w*, water content; PL, plastic limit; LL, liquid limit; γ, unit weight of water; *c*, void ratio; S, degree of saturation; *c*′, cohesion; ϕ', soil friction angle.



The bottom of the porous clay is clearly indicated by a sudden rise in the SPT blow count, as the residual soil from the slate or the siltstone or quartzite layers is encountered. These residual soils were investigated during excavations and show an inherent anisotropy as a dominant feature. Bedrock characteristics such as bedding and shear planes, remaining in the residual soil, control their behaviour.

Characteristics of the porous clay

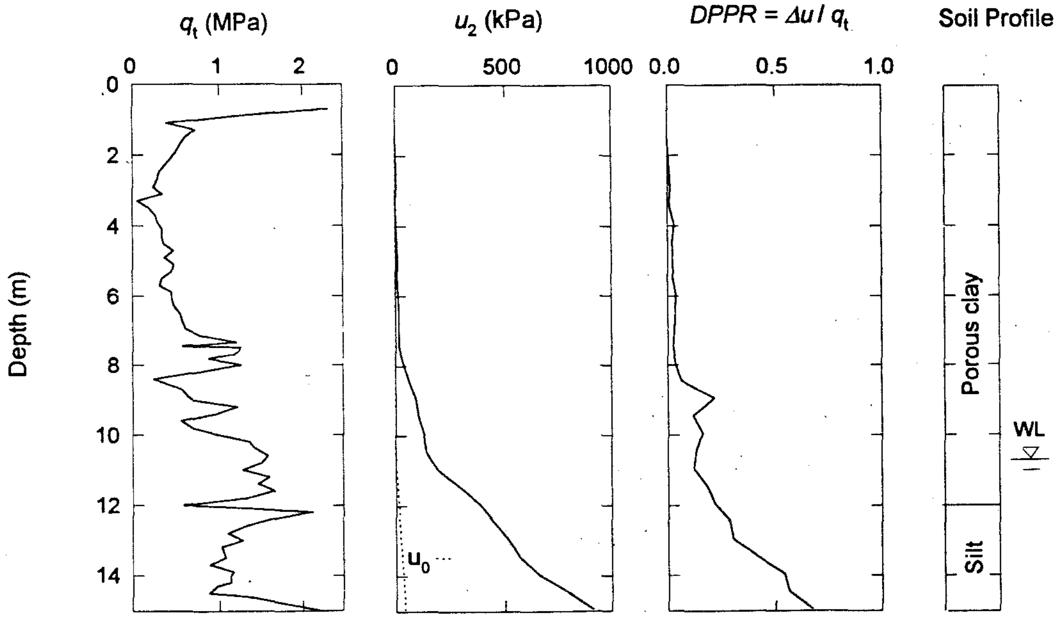
A summary of the laboratory test results on the porous clay is presented in Fig. 2. At the top, the porous clay layer presents a low unit weight and high void ratio. Foundation experience with this clay has proven that it is collapsible. Low-rise buildings on shallow foundations tend to crack 1 to 2 years after construction. A good practice is to adopt deep foundations consisting of small diameter bored piles, even for one-story buildings.

In situ test classes

Two different classes of in situ tests were carried out: logging and calibration tests. The first, encompassing SPT, CPT, and DMT, are those analysed by empirical methods only. They have to be calibrated against soil properties from laboratory tests or in situ tests such as the pressuremeter, in which test conditions (deformation or stress field) are supposed to be known and can be analysed by a rigorous method. These tests form the calibration class.

The SPT is the site investigation tool for deep foundations in Brasília, reaching the hard strata below the soft clay layer. It is, however, of little use for obtaining soil properties in the porous clay, since available correlations for similar soils in São Paulo (Negro et al. 1992) show considerable data scatter for SPT blow count lower than 15 blows/30 cm. The site investigation programme included the in situ tests described below.

Fig. 3. CPTU results. WL, water level.



Piezocone test (CPTU)

Only one CPTU test was carried out. The test location is shown in Fig. 1. The piezocone was 20 cm^2 in cross-sectional area, 60° in apex angle, and the pore pressure was measured behind the tip, corresponding to the u_2 location, according to Robertson and Campanella (1988). The lateral friction was not measured because of electrical faults.

The data were processed with a cone program from The University of British Columbia (CPTINT; Campanella 1991), and are plotted in Fig. 3. This plot presents the corrected tip resistance q_t, the pore pressure u_2, and the differential pore pressure ratio (DPPR). Below the water table, positive DPPR values were measured. This is an indication of volume decrease during shear, typical of normally or slightly overconsolidated clays.

Dilatometer tests (DMT)

The dilatometer was developed in Italy by Marchetti (1980) and consists of a 14 mm thick, 95 mm wide, and 220 mm long blade which is driven or pushed into the soil. On one face there is a 60 mm diameter steel diaphragm capable of a lateral expansion of 1 mm under gas pressure.

Marchetti proposed a series of correlations based on Italian soils for estimating the soil type, the unit weight, the in situ stress ratio K_0, the overconsolidation ratio (OCR), the undrained strength for clays c_u, the friction angle for sands, and the one-dimensional compression modulus M. These correlations are based on a limited number of Italian soils, i.e., eight sand deposits and only two clay deposits. Lunne et al. (1989) added soil data from other countries and reviewed DMT correlations and proposed those used in this work.

Five DMT boreholes were carried out in the south wing of Brasília (Fig. 1), with some 20–30 tests per working day. The small size and portability of the equipment and the use of a local drilling rig led to high productivity and low costs per test (Ortigao 1994). The data were processed by a computer program and plotted as indicated in Figs. 4 and 5. The results indicate a reasonable repeatability in the porous clay, in which K_0 is close to 0.6, the friction angle varies in the range of 20–27°, and the dilatometer modulus (E_D) increases linearly with depth. This latter modulus represents a measure of the elasto-plastic response of sand (Campanella and Robertson 1989), since it is defined through the slope of the DMT expansion curve.

Several conclusions were obtained from Fig. 5. K_0 values are high for the first 2 m of depth and then decrease and lie in the 0.5–0.7 range, slightly decreasing with depth. Values of E_D increase from zero at the ground level and reach 15 MPa at 15 m depth, suggesting a normally consolidated behaviour. Notwithstanding, spurious results were obtained for OCR and will not be presented here. Considerable scatter in the compression modulus (M) is also noted.

The repeatability of the results is remarkable, especially regarding the friction angle that lies in the narrow range of 25–28° and does not vary with depth. It is noticed that DMT correlations yield reproducible and operator insensitive parameters, with a low variation for the results in the five boreholes analysed.

Horizontal plate loading tests (PLH)

Horizontal plate loading tests were proposed by the tunnel designer to be carried out in the test pits used for block sampling, as presented in Fig. 6*a*. These pits were 1.5 m square in plan and were excavated above the water table down to a depth of 10 m below ground level. At selected depths, two 300 mm × 300 mm × 50 mm steel plates were jacked against opposite sides of the pit walls. Loading was measured by means of an electric load cell, and lateral displacements were measured by two pairs of dial gauges. Displacements were measured on the steel plates, and

Fig. 4. Typical result from a DMT test (SP608).

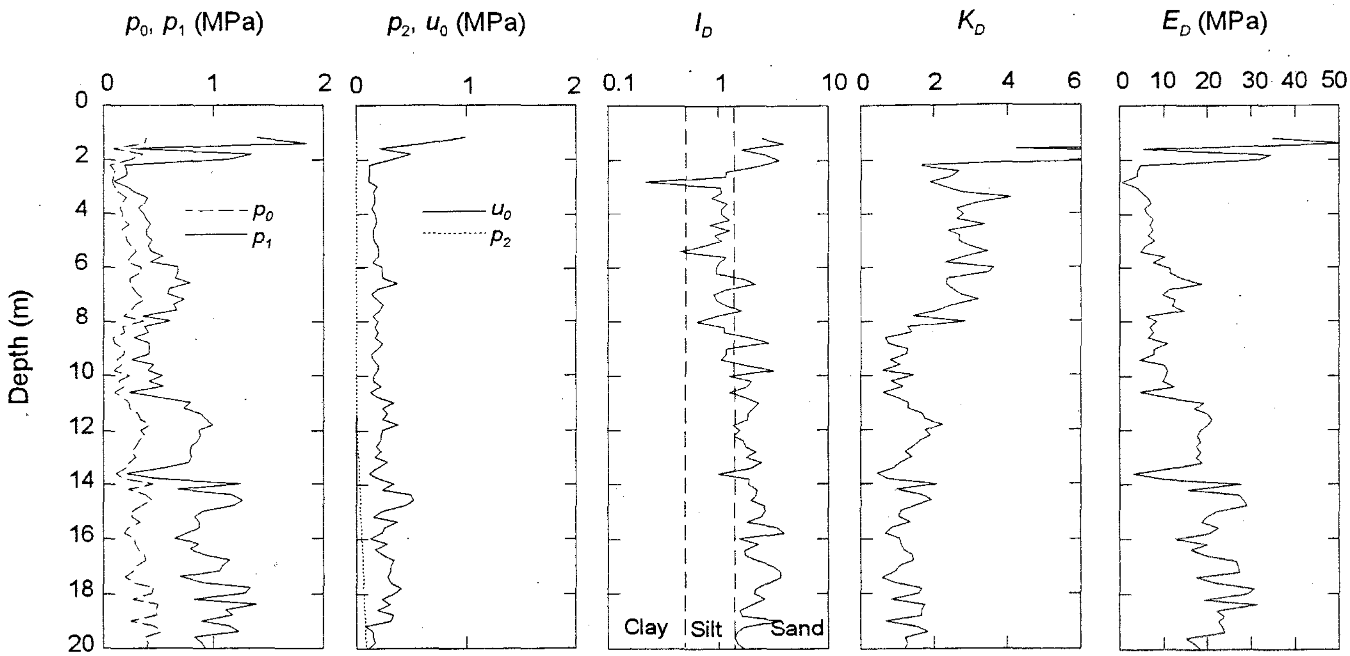
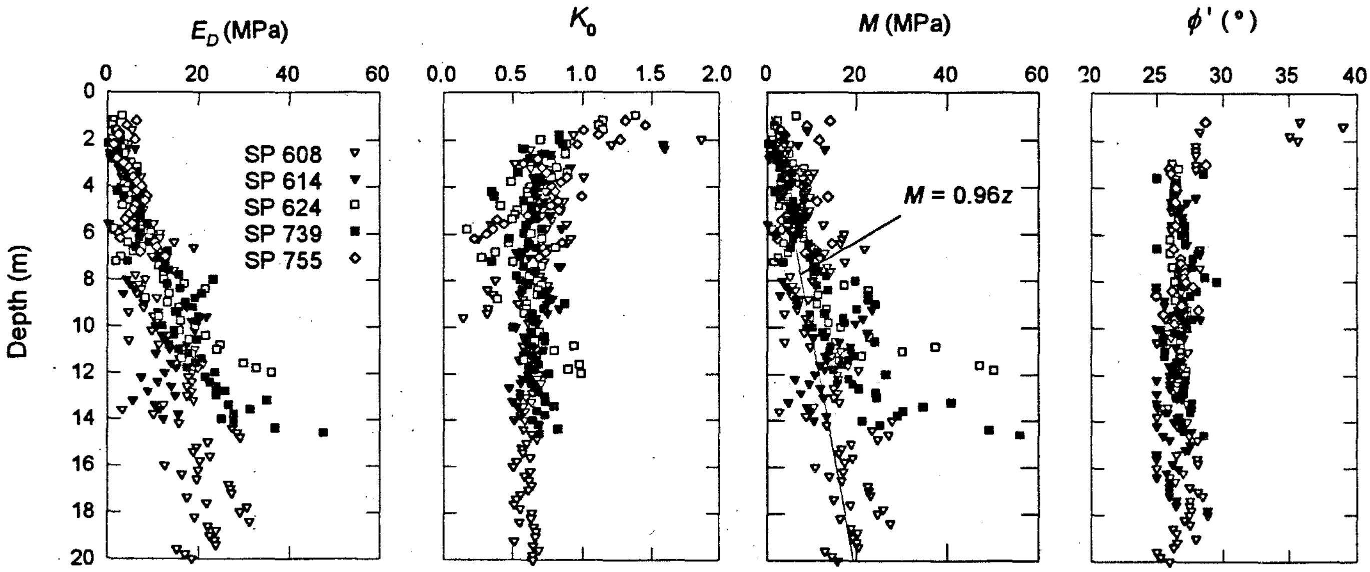


Fig. 5. Results from DMTs.

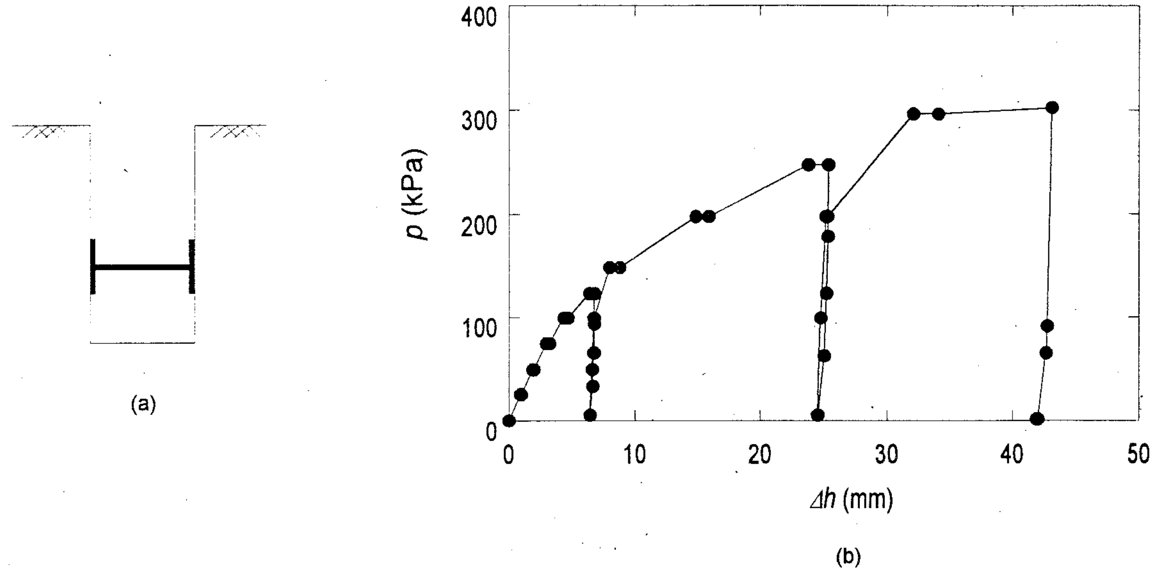


certainly bedding errors occurred. Better results would have been obtained if internal displacements were measured in the soil mass behind the steel plates. The test equipment rested on frictionless steel bearings on a wooden frame 1.5 m above the pit bottom.

Figure 6*b* presents a typical plot of the results, where the lateral pressure p on the steel plate is plotted against the horizontal displacement Δh. Two unloading–reloading cycles were performed, and jacking continued until the pressure p could no longer be increased.

The results were analysed assuming isotropic linear-elastic behaviour and a Poisson's ratio of $\upsilon = 0.3$. Using the initial slope at the beginning of the test, it is possible to obtain the modulus E_i. Then, for a loading cycle, the unloading–reloading modulus E_{ur} is obtained. The results are plotted in Fig. 7, where it can be noticed that the unload–reload moduli from the plate load tests are higher than moduli measured by the initial slope of the test. This is mainly caused by the stress level dependency on the soil modulus, as noted by many authors (Robertson 1982; Bellotti et al. 1989). The unload–reload modulus is carried out at a stage where an expanding elasto-plastic zone exists around the pushing plate, and hence, the average level of mean normal stress is higher than the original stress level of the deposit.

Fig. 6. (*a*) Horizontal plate loading test; (*b*) PLH expansion curve.



Ménard pressuremeter (PMT)

Two pressuremeter boreholes were carried out employing a Ménard-type probe. A detailed description of the pressuremeter and its application is found elsewhere (e.g., Baguelin et al. 1978). The productivity achieved was relatively low, a maximum of six PMTs were carried out in a working day. The authors preferred way for plotting a PMT test is to relate strain to inflation pressure, as shown in Fig. 8. Values of circumferential strain are obtained from volume change measurements through the equation

$$\varepsilon_\theta = \frac{1}{2}\frac{\Delta V}{2V_0}$$

If elastic conditions are assumed, the average elastic shear modulus G_i and the unloading–reloading modulus G_{ur}, obtained at a pressure cycle, are given by the slope of the inflation curve as

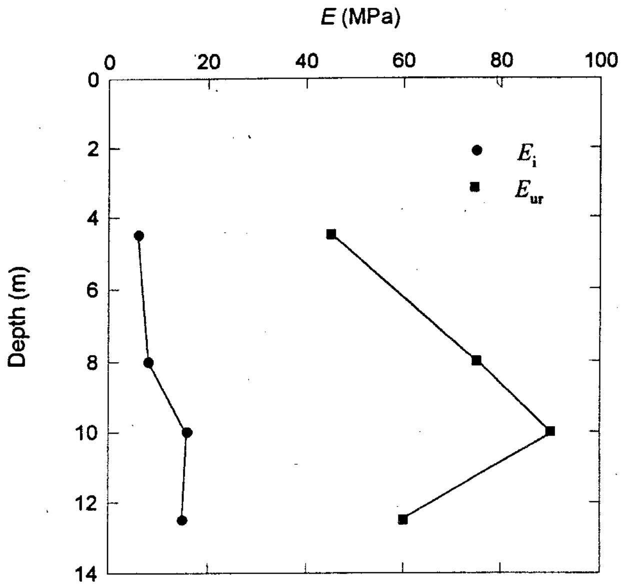
$$G = \frac{1}{2}\frac{\Delta p}{\Delta\varepsilon_\theta}$$

At the end of the expansion, the inflation reaches the limit pressure p_{lim}, corresponding to $\Delta V = V_0$ or to a circumferential strain of $\varepsilon_\theta = 50\%$.

A disadvantage of the Ménard pressuremeter is that the insertion method through a prebored hole disturbs the soil, and it is impossible to reproduce the preexisting soil stress condition. Therefore, the initial part of the curve in Fig. 8 is usually disturbed and can not be used to infer the original lateral stress of the deposit. An effort was made to obtain the shear modulus by another manner.

Hughes (1982) found that the unload–reload modulus, G_{ur}, from loops during the expansion phase of a test is much less sensitive to disturbance than the one measured at the initial stage of test, also confirmed by Bellotti et al. (1989). However, as quoted earlier in this paper for PLHs, G_{ur} is related to a stress level higher than the original stress of the soil. This led Robertson (1982) and Bellotti et al. (1989) to propose expressions to derive the average stress level surrounding the probe (within the idealised plastic zone) that exists prior to the loop stage, and hence obtain a modulus that is corrected for stress level with Janbu's (1963) equation. Therefore, all the unload–reload moduli, G_{ur}, were corrected to stress level based on the Bellotti's equations, assuming $K_0 = 0.55$ from average DMT's results.

Fig. 7. Young's moduli from PLH tests.



The analysis of the pressuremeter data by the traditional interpretation methodology, as discussed in Baguelin et al. (1978) leads to the parameters presented in Fig. 9. The Young modulus E_m is obtained through G_i with a Poisson's ratio of 0.3, as previously estimated by the plate loading tests. The limit pressure p_{lim} is directly obtained with the extrapolation of the pressuremeter testing curve for a volume change equal to the initial volume of the cavity.

Fig. 8. PMT expansion curve.

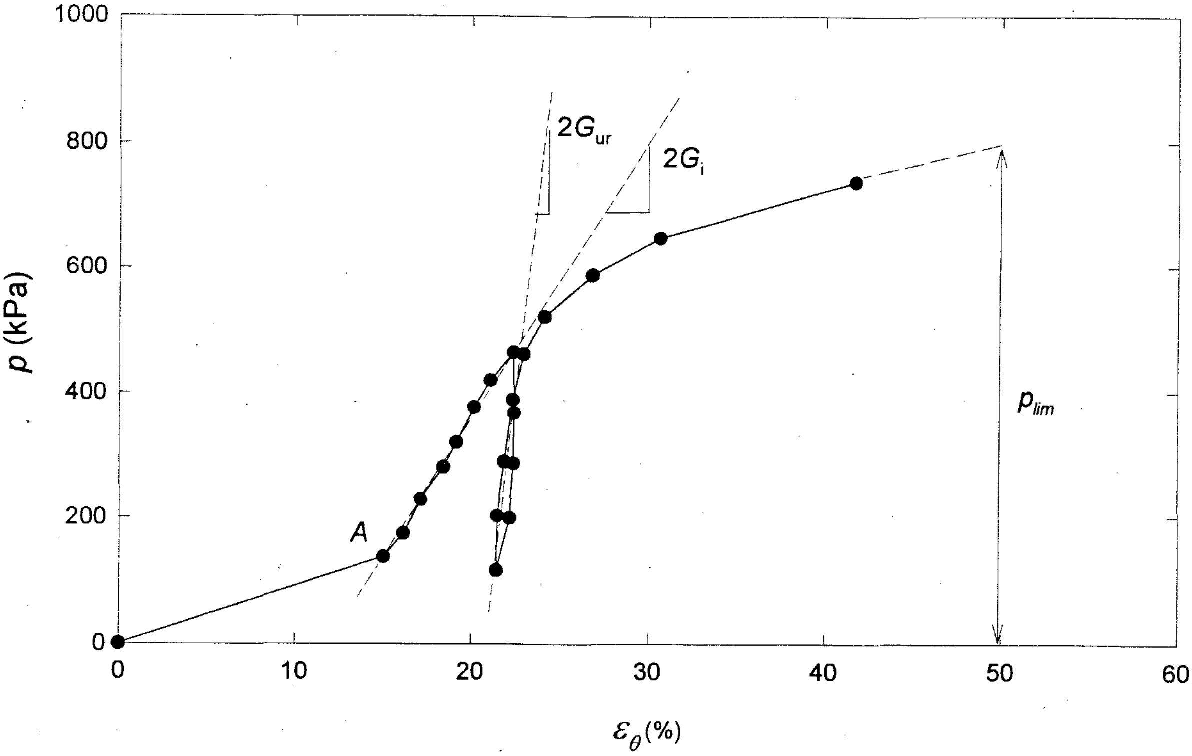
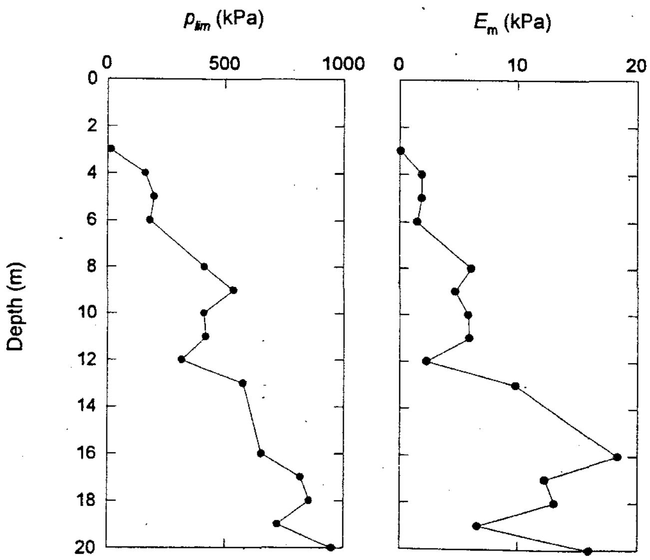


Fig. 9. PMT results.



Comparison of results

Results from different in situ tests were compared with each other and with laboratory test data. The following observations can be made:

The limit pressure is usually defined for the PMT only. However, the DMT interpretation by Campanella and Robertson (1989) enables one to define an equivalent p_{lim} for the DMT equal to the p_1. Additionally, it is also possible to obtain p_{lim} from PLHs similarly to the procedure used on the PMTs. Figure 10 shows all results. There is a reasonable agreement of p_{lim} values from different tests, which is not at all a surprise because p_{lim} is obtained for very large deformation and is beyond any influence of soil disturbance by an insertion method. Some scatter of the results is, however, observed.

Figure 11 plots Young's moduli from all in situ tests. Due to nonlinearity in soil behaviour, the secant modulus depends on the strain or stress level. This should be considered when comparing moduli from different tests, and a reference strain level should be obtained for each test. However, the establishment of a relevant stress and strain level for in situ tests is not an easy task, since the boundary conditions in the field are unknown.

Data in Fig. 11 show that E_m (PMT) is less than E_D (DMT), which in turn is in the same order of magnitude of a few E_i (PLH) data. Soil disturbance during borehole drilling and probe insertion is the main explanation for the low PMT's moduli. The agreement of resutls between PLH and DMT moduli is related to the fact that both moduli are measured at relatively large strains, where the soil behaves in an elasto-plastic (nonlinear) manner. Indeed, for both DMT and PLH, the modulus is defined in the shear strain range of the testing curve, equal to and above 0.1%.

The low values given by the PMT were disappointing and led to an additional effort in understanding the causes. Jefferies (1988) and others (e.g., Cunha 1994; Ferreira 1992) advocated an improved method of analysis for self-boring pressuremeter tests. It consists of theoretically generating a expansion curve using a set of material parameters which are varied until an agreement with the PMT data is obtained.

Fig. 10. Limit pressure p_{lim} from PMT, DMT, and PLH.

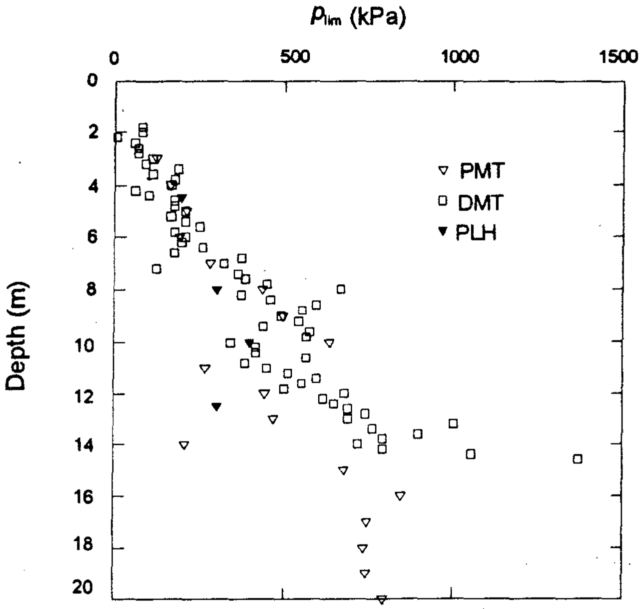
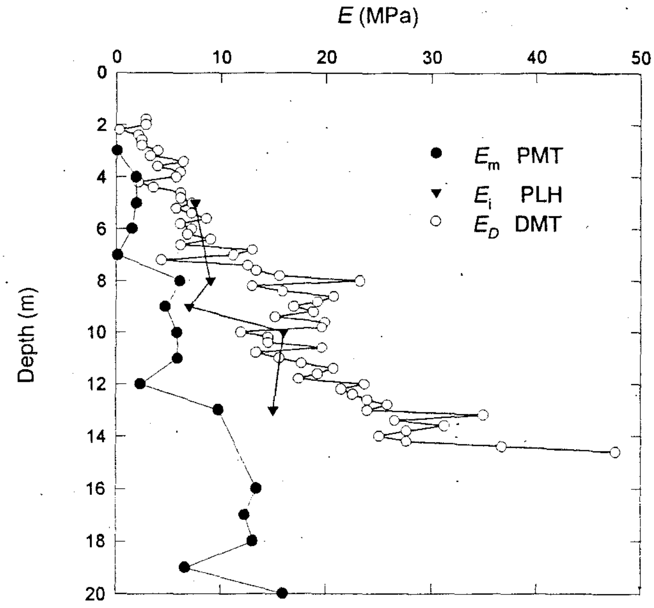


Fig. 11. Young's moduli from PMT, PLH, and DMT.



This analysis requires the choice of an expansion model. The theory of Carter et al. (1986) was selected because it is a closed form solution for a cylindrical cavity in cohesive-frictional material. This theory assumes infinitesimal deformations, plane strain conditions, elasto-plastic behaviour with Hooke's law in the elastic phase until the expansion reaches Mohr-Coulomb's failure criterion. The soil can be considered dilative or contractive, depending on the values assumed for the peak friction angle ϕ'_p, and the critical state friction angle ϕ'_{cr}. A computer program named PMT was written to read field data and generate a theoretical curve from a set of six given parameters: σ'_{h0}, G, υ, c, ϕ'_p, and ϕ'_{cr}, where c is the cohesion and the other parameters were previously defined. The results are then displayed, as shown in Fig. 12. The user varies the parameters until an agreement is obtained. Three or four runs are necessary to reach a good agreement.

This method of analysis leads to a set of *coupled* soil parameters, i.e., they are dependent on each other. This means that, for instance, if soil modulus G is increased, another parameter should be decreased to give the same agreement with the field curve. As a result, it is expected that errors are better distributed among all parameters.

Unlike self-boring pressuremeter tests, the origin of the inflation curve of a Ménard pressuremeter is undefined. This kind of analysis requires that the user estimates the origin, as shown by point A in Fig. 8.

In the present analysis ϕ'_{cr} was not known, in the absence of adequate large strain laboratory tests. Therefore, ϕ'_{cr} was assumed to vary within reasonable limits (30–35°) that would encompass the value of this clay. However, the results show a low sensitivity to variations in ϕ'_{cr}, suggesting that estimates of this parameter can be done in the lack of a laboratory input value. The same conclusion was obtained by Cunha (1994) for sands.

Cohesion was inferred from laboratory data and assumed to be 30 kPa, constant with depth. Its value has little effect on the results for depths greater than 3 m. Poisson's ratio was taken as 0.3 and kept constant.

The analysis consisted of varying ϕ'_p, the shear modulus G, and the effective horizontal stress ϕ'_{h0} until the field and theoretical curves agree. The curve match was sought in the large strain range, above the circumferential strain of 30%, since this range of the testing curve is less affected by the initial disturbance of the soil. Moreover, at this strain range, the surrounding plastic zone has expanded sufficiently enough to encompass the disturbed annulus of soil around the cavity, caused by the initial pressure relief around the cavity.

The results of the curve-fitting technique are presented in Fig. 13, where the following can be noted:

(1) There is a slight resemblance between the shapes of the obtained curves (parameters versus depth) and the cone profile for this deposit. This indicates that the curve-fitting was capable of obtaining soil parameters that followed the natural stratigraphic variability of this profile.

(2) The coefficient of lateral stress does not vary significantly with depth, with a mean value close to 0.5. Similar behaviour was noticed with the DMT data and may suggest that the geological processes involved with the formation of this clay did not considerably affect the stress regime along depth.

(3) The stiffness of the clay increases almost linearly with depth due to the increase in the mean normal stress along depth.

(4) The range of friction angle results is larger than the range obtained for the other parameters. A mean value close to 26° was obtained for this clay.

The comparison among peak friction angles predicted by the DMT, the PMT (curve-fitting technique), and the measured laboratory values is presented in Fig. 14. It is noticed in this figure that the range defined by the curve-fitting technique of the PMT data lies close and above the

Fig. 12. Example of PMT program curve fitting.

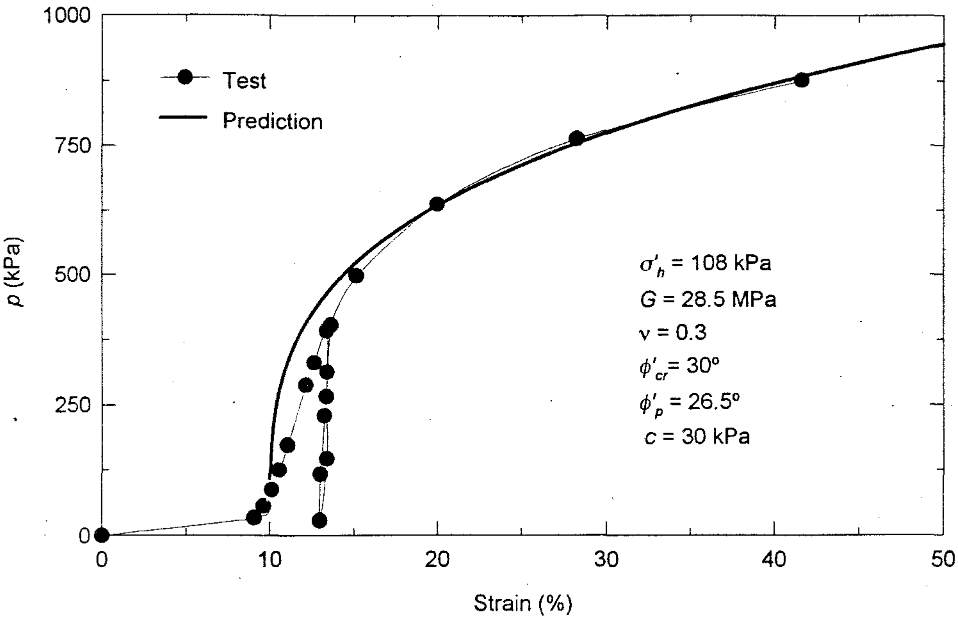


Fig. 13. PMT results via curve-fitting technique.

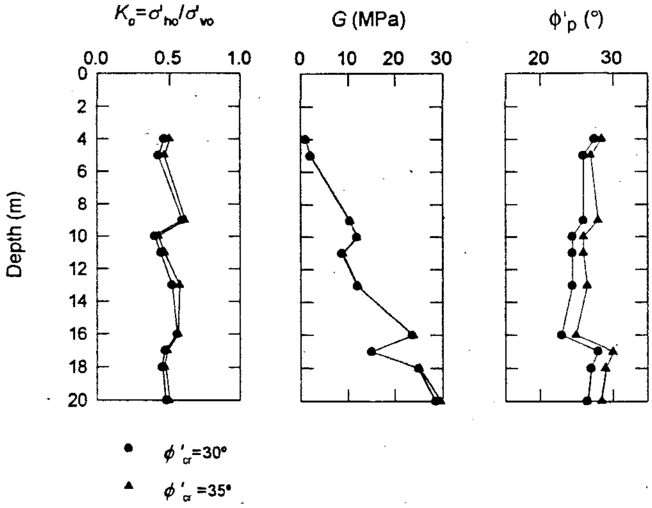
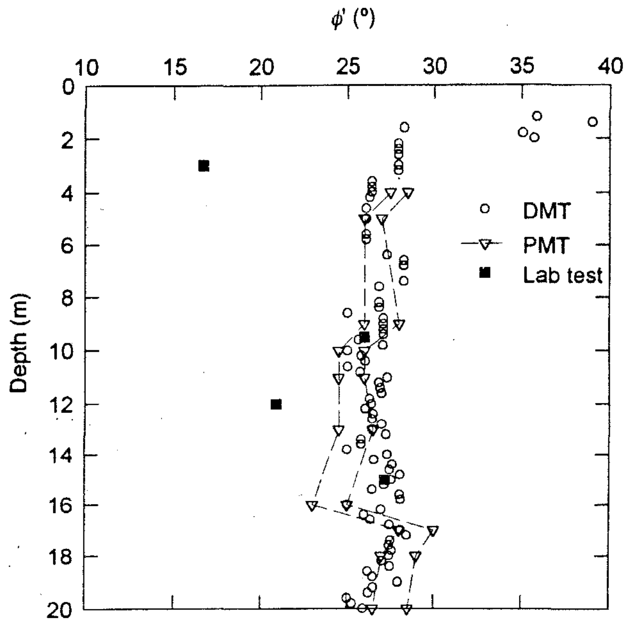


Fig. 14. Comparison of friction angles.



few data points representative of isotropically consolidated triaxial values. Indeed, an agreement among results should not be expected because the PMT data refers to a drained shearing mechanism that differs from those of the triaxial samples. Wood and Wroth (1977) observed that the mode of shear deformation imposed by pressuremeter tests corresponds to the deformation that could be imposed in a plane strain test with the sample in horizontal position, i.e., the plane where the vertical stress acts in the field would be the laboratory intermediate stress. This differential mode of deformation has been shown to impose differences in the mechanical properties of sands measured during shear (Oda 1981).

The average value for the friction angle $\phi' = 26°$ given by the DMTs, with remarkable repeatability, is reasonably close to the range defined by the PMT analysis, suggesting that the empirical correlation employed in the DMT analysis can be used for preliminary assessment of the strength of the Brasília porous clay. Note that ϕ' given by the DMTs was based on correlations for sands.

The comparison among coefficients of lateral stress K_0 for the DMT, the PMT, and the Jáky (1994) formula for normally consolidated soils is presented in Fig. 15. The results indicate that this coefficient does not vary significantly with depth and that the porous clay seems to have a normally consolidated behaviour, where the Jáky expression can be used.

As stated before, the DMT values are close to and slightly above the range defined with the curve-fitting

Fig. 15. Comparison of lateral stress predictions.

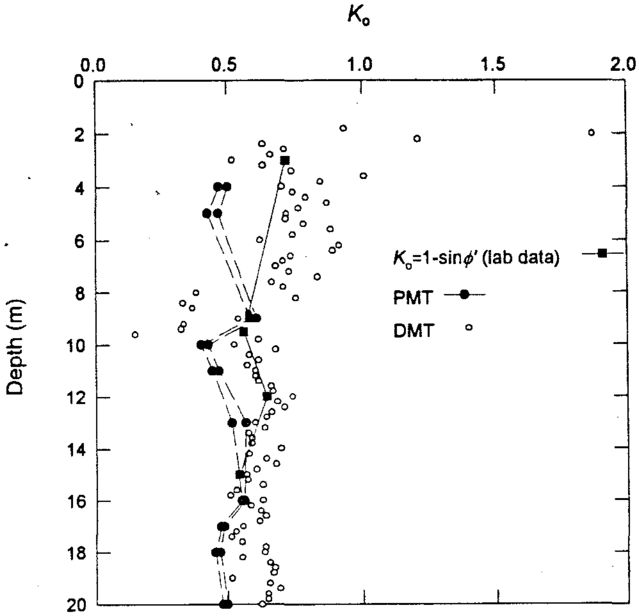
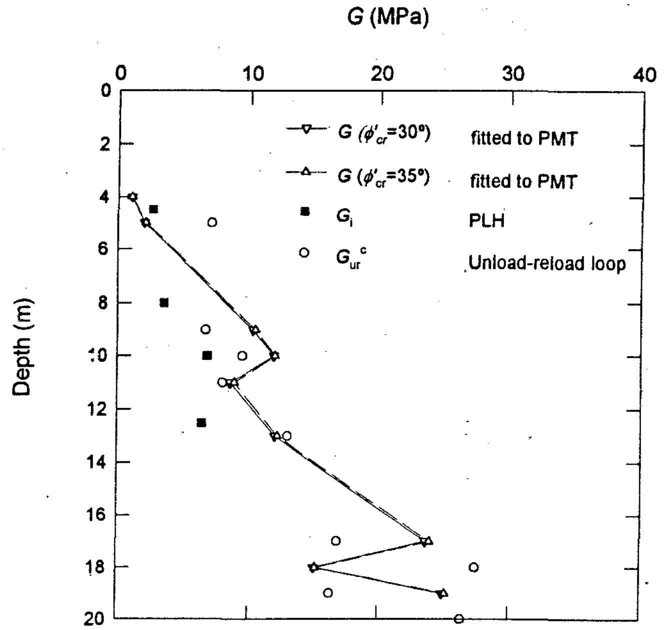


Fig. 16. Comparison of stiffness from PMT and PLH.



technique of the PMT data. This is due to the natural variability of the deposits considered in the data base used by Lunne et al. (1989) to define the empirical equation for young clays between K_0 and K_D of the DMT. Lunne's equation can be further calibrated with PMT data, in order to establish a unique correlation for the Brasília porous clay. For this purpose a correction factor in the range of 0.8–0.9 shall be multiplied to the original Lunne's equation.

The comparison between the shear moduli obtained by the curve-fitting technique of the PMT data, the stress level corrected value of G_{ur}, and the plate load tests converted moduli (from E_i, using a Poisson's ratio of 0.3) is presented in Fig. 16. The comparison between the stress level corrected values, G^c_{ur}, and G indicates an average ratio G^c_{ur}/G in the range of unity when all the testing depths are considered. Since both moduli are related to similar shearing conditions and the same value of average lateral effective stress, they should be indeed close. Localized differences between G^c_{ur} and G in some depths may be given by slight differences in the amplitude of shear strain that are related to each of these moduli.

The moduli obtained by the PMT test, either using the unload–reload loop or using a curve-fitting technique, is higher than the converted moduli from the horizontal plate loading tests. As noted earlier, PLH testing data appears to be related to the very large strain behaviour of the soil, where plastic strains are present and the soil behaves in an elasto-plastic manner.

The shear moduli obtained by the curve-fitting technique represents an index to the stiffness of the material within the idealized elastic zone around the cavity, where the strain level is within and below 0.1%. This value was estimated based on an extensive data base obtained by Cunha (1994) with PMT tests in sands.

Hence, although PMT and PLH impose a similar shearing mechanism in the surrounding material, the obtained results can be vastly different depending on the strain level adopted to estimate the PLH moduli. Therefore, depending on the level of strain imposed by the engineering work in the soil, one or the other moduli will be more appropriate for the design. Further research in this area is being undertaken at the University of Brasília.

Conclusions

This paper addressed laboratory tests, field investigation, and parameter evaluation for the Brasília porous clay. The laboratory tests confirmed the drained behaviour of this cohesive material during shear, suggesting that empirical equations developed exclusively for sands can be used to assess the parameters with in situ testing data.

The initial conclusion from in the in situ testing programme carried out in this deposit is that there is no useful correlation between SPTs and deformation moduli. The SPT index is not at all sensitive to the low stiffness of the porous clay. The time-consuming PLH tests yielded moduli results that agree with the DMT predictions but that are lower than the values estimated by the PMT via unload–reload loop or a curve-fitting technique. Both PLH and DMT moduli are related to the soil behaviour in a higher strain level than those imposed by the PMT tests during the unload (loop) stage or within the elastic zone around the cavity. Therefore, the PMT moduli are an index of the soil stiffness when sheared in a pseudo-elastic manner with strains below 0.1% the horizontal direction, whereas the PLH and DMT refer to an elasto-plastic soil behaviour.

The friction angle of the porous clay was well defined by either the curve-fitting technique of the PMT tests or the empirical evaluation of the DMT data. These values, however, were slightly above the laboratory results of the isotropically consolidated triaxial tests. Differences in the mode of shear and the possible anisotrophy of the clay are used to explain the differences. The porous clay of

Brasília also seems to behave in a normally consolidated condition, based on the results of lateral stress. The curve-fitting technique of the PMT data yielded K results close to those predicted by the classical Jáky (1944) equation with the laboratory triaxial data. The DMT predictions of K_0 were close to and slightly above the PMT predictions, suggesting that the equation employed to infer K_0 from K_D can be further calibrated against the PMT data for a customized future DMT interpretation of this same deposit.

The traditional interpretation of the PMT data yielded unreliable and spurious results and had to be disregarded. The curve-fitting technique adopted in this paper and advocated by many authors (Jefferies 1988; Ferreira 1992; Cunha 1994), can be used to obtain reliable soil parameters from the Ménard (or any other) pressuremeter test.

It is also concluded that the DMT can be efficiently used in engineering works provided that at least one PMT testing borehole is carried out for calibration purposes. The DMT led to good results in terms of strength and lateral stress, and presented a very easy deployment and operation, low cost, and high productivity.

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