# Settlement of Floating Bored Piles in Brasilia Porous Clay

W. Patrick Stewart, Renato P. Cunha, Neusa M.B. Mota

**Abstract.** The geotechnical graduate program of the University of Brasilia maintains a research site on the campus (to be discontinued for a new place). The site is underlain by the typically partly-saturated and potentially collapsible "porous clay" of the Federal District of Brazil. The soil conditions have been thoroughly evaluated using laboratory and *in situ* geotechnical tests (DMT, CPT, SPT, and PMT). Five bored piles were installed and tested at the site. Simplified analyses have been used so that the results of the tests can be easily compared. The various soil tests were used to estimate the pile settlements which were compared to the measured values and the results are discussed. It has been shown that simple elastic models can be routinely used in practice for the estimation of the settlement of bored floating piles on tropical unsaturated soils. Besides, the results tend to indicate that PMT tests provide the best ratios between predicted and measured data.

**Keywords:** *in situ* testing, Brasilia porous clay, pile settlement, elastic theory.

## 1. Introduction

Brasilia, the capital city of Brazil, was a pre-designed city, built to accommodate the federal government and the supporting population of staff and workers. Recently the size of the city has increased in both population and developed properties. Given the particular conditions of the local tropical subsoil, specific local solutions have been developed for foundation design. Recently more research-based solutions and techniques have been developed with the support of the University of Brasília "Foundation Group" (www.geotecnia.unb.br/gpfees), a joint academic-industry group. The good academic-industry interaction has not only allowed a better knowledge of the existing technologies, but also has stimulated a pioneering use of advanced *in situ* tests (such as the DMT, CPT, the standard penetration test with torque measurement, SPTT, and PMT) in the tropical soil of the city.

Brasília is located in the Central Plateau of Brazil, and is portrayed in Fig. 1 by an "airplane" shape like form. The University of Brasília (UnB) campus is located within the city of Brasília. The UnB foundation and *in situ* testing research site is marked on this figure.

Within the Federal District extensive areas are covered by a weathered latosoil of Tertiary-Quaternary age. This latosoil has been extensively subjected to a laterization process and has a variable thickness throughout the District, varying from a few centimetres to around 40 m. In this latosoil there is a predominance of the clay mineral kaolinite, and oxides and hydroxides of iron and aluminum (giving it a distinct reddish colour). The variability of the properties depends on several factors, such as the topography, the vegetation cover, and the parent rock. In localized areas of the Federal District the latosoil overlays a saprolitic/residual soil with a strong anisotropic mechanical behaviour and high (SPT) penetration resistance. The saprolite originated from a weathered, folded and foliated slate, the typical parent rock of the region.

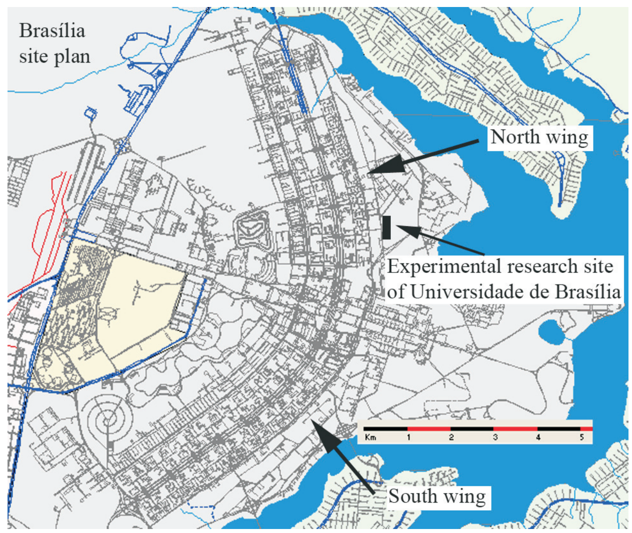


**Figure 1** - Site plan of Brasilia showing the research site of the UnB geotechnical group.

## 2. Site Characterization

The superficial latosoil is locally known as the Brasília "porous clay", forming a lateritic horizon of low unit weight and high void ratio, and often an extremely high coefficient of collapse (Cunha *et al.*, 1999). However the soil can vary from clay to silt and in the upper portion of this site, silty sand. By breaking down the structure with a

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deflocculating agent, the grain size curve of this soil shows a greater concentration of clay-size particles.

Figure 2 contains a simplified profile of the deposit, characterized by a superficial lateritic layer overlying a transition zone and a saprolite formed by the native rock of the region. The figure also presents the average results of SPT blow counts, torque measurements, CPT tip resistance and lateral sleeve friction, for each meter depth at the site. Table 1 presents the geotechnical characterization of the site, based on soil classification tests, including

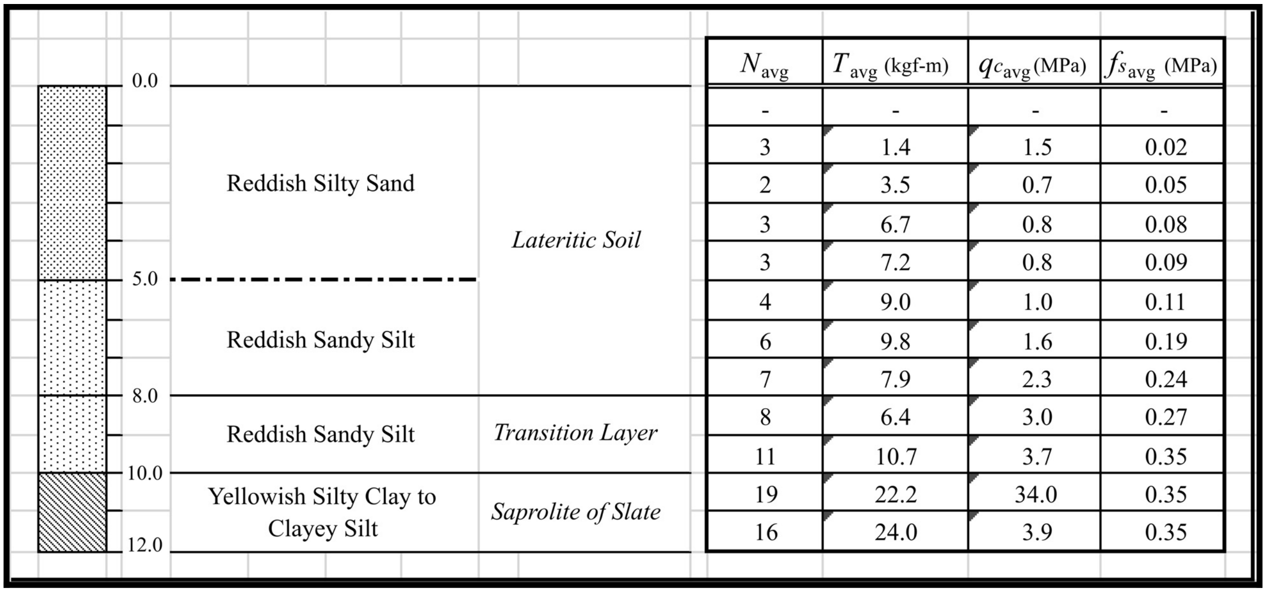


**Figure 2** - Simplified profile of the soil at the UnB research site.

**Table 1** - Geotechnical characterization of the soil of the UnB experimental site.

| Parameter | Depth (m) | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | 1 | 2 | 3 | 4 | 5 | 6 | 7 | 8 | 9 | 10 |
| $\gamma_s$ (kN/m$^3$) | 26.9 | 26.8 | 26.1 | 25.9 | 26.9 | 25.8 | 26.5 | 26.2 | 27.1 | 27.6 |
| $\gamma_d$ (kN/m$^3$) | 10.2 | 10.4 | 11.5 | 11.5 | 12.0 | 12.0 | 12.8 | 13.9 | 13.8 | 13.3 |
| $\gamma$ (kN/m$^3$) | 13.3 | 13.7 | 14.7 | 14.5 | 15.0 | 14.4 | 15.4 | 18.0 | 17.8 | 17.5 |
| $\gamma_{sat}$ (kN/m$^3$) | 16.5 | 16.5 | 17.1 | 17.0 | 17.5 | 17.3 | 17.8 | 18.6 | 18.8 | 18.5 |
| $G_s$ | 2.7 | 2.7 | 2.7 | 2.7 | 2.7 | 2.6 | 2.7 | 2.7 | 2.8 | 2.8 |
| $e$ | 1.6 | 1.57 | 1.27 | 1.27 | 1.25 | 1.15 | 1.07 | 0.89 | 0.96 | 1.08 |
| $n$ (%) | 61.6 | 61.1 | 56.0 | 55.9 | 55.6 | 53.5 | 51.7 | 47.2 | 49.0 | 51.9 |
| Gravel ND[1] | 0.2 | 0.2 | 0.7 | 0.8 | 1.4 | 2.1 | 4.3 | 3.6 | 0.6 | 0.0 |
| Sand ND | 56.2 | 56.2 | 53.2 | 53.0 | 49.2 | 34.9 | 30.1 | 42 | 10.2 | 1.4 |
| Silt ND | 51.4 | 35.9 | 34.2 | 43.1 | 48.6 | 61.4 | 61.9 | 51.9 | 86.8 | 79.5 |
| Clay ND | 2.2 | 7.7 | 11.9 | 3.1 | 0.8 | 1.6 | 3.7 | 2.5 | 2.4 | 19.1 |
| Gravel WD[2] | 0.2 | 0.2 | 0.7 | 0.8 | 1.4 | 2.1 | 4.3 | 3.6 | 0.6 | 0.0 |
| Sand WD | 41.5 | 41.5 | 41.6 | 33.7 | 31.6 | 25.7 | 22.7 | 33.8 | 10.2 | 3.4 |
| Silt WD | 24.9 | 29.2 | 25.7 | 26.3 | 26.5 | 22.9 | 24.6 | 27.4 | 80.4 | 93.2 |
| Clay WD | 33.4 | 29.1 | 32.0 | 39.2 | 40.5 | 49.3 | 48.4 | 35.2 | 8.8 | 3.4 |
| $w_L$ (%) | 38 | 36 | 39 | 41 | 45 | 44 | 46 | 43 | 44 | 46 |
| $w_P$ (%) | 28 | 26 | 29 | 29 | 34 | 33 | 35 | 34 | 26 | 30 |
| $PI$ (%) | 10 | 10 | 10 | 12 | 11 | 11 | 11 | 9 | 18 | 16 |

[1]Gravel portion with no deflocculating agent. [2]Gravel portion with deflocculating agent.

grain size proportions both without and with a deflocculating agent.

## 3. Field and Laboratory Tests

In support of the foundation testing, a series of field and laboratory tests have been completed at the site (for more details see Mota, 2003). Table 2 summarizes the field tests. Figure 3 presents the layout of the field testing and test piles.

The dilatometer tests (DMT) were carried out with a standard Marchetti apparatus pushed into the soil with a 200 kN hydraulic field rig (until the maximum resistance was met). The tests were done in accordance with ASTM D-6635-01, using nitrogen gas to expand the membrane. Measurements were done at 20 cm intervals, and the dilatometer was pushed at 2 cm/s. The measured pressures were corrected using lab calibrations. Typical examples are shown in Fig. 4.

**Table 2** - Summary of field tests at UnB experimental site.

| Test type | Total no. of borings | Depths (m) at end of test | Comments |
|---|---|---|---|
| DMT | 12 | 12.0-18.2 | Hydraulic Field Rig |
| CPT | 17 | 12.1-18.0 | Hydraulic Field Rig |
| SPT-T | 5 | 10.5-12.5 | Manual Procedure |
| PMT | 3 | 7.6-9.6 | Done in sequence together with SPT's |

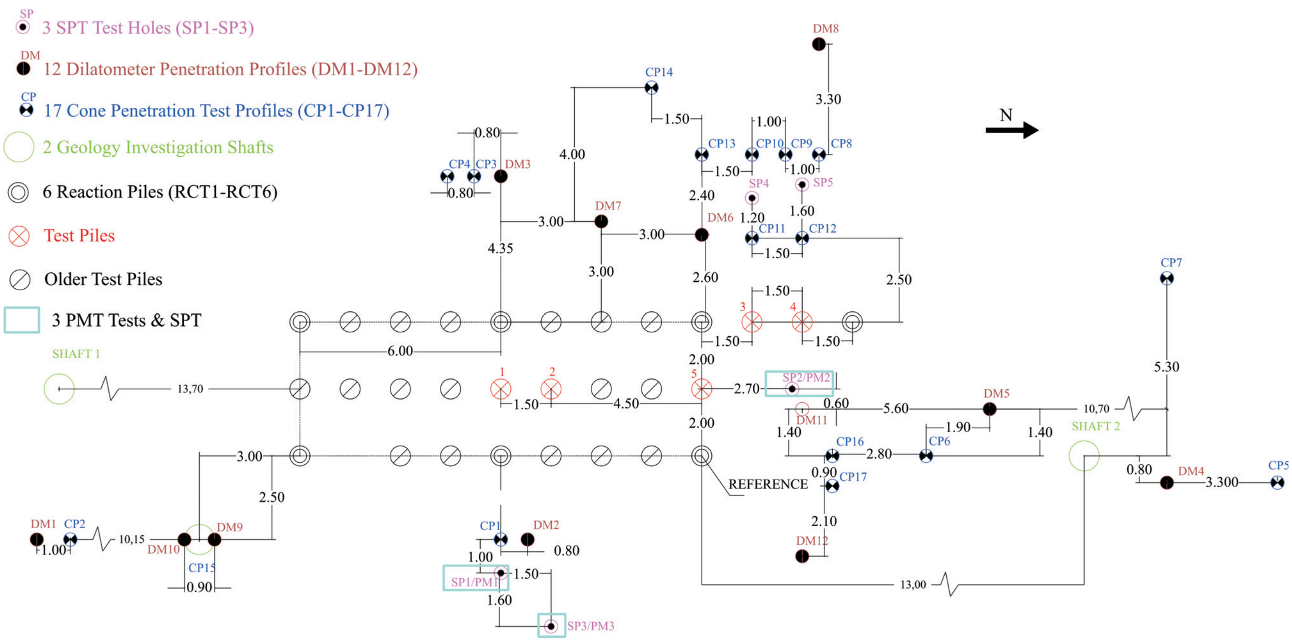


**Figure 3** - Layout of *in situ* testing and test piles.

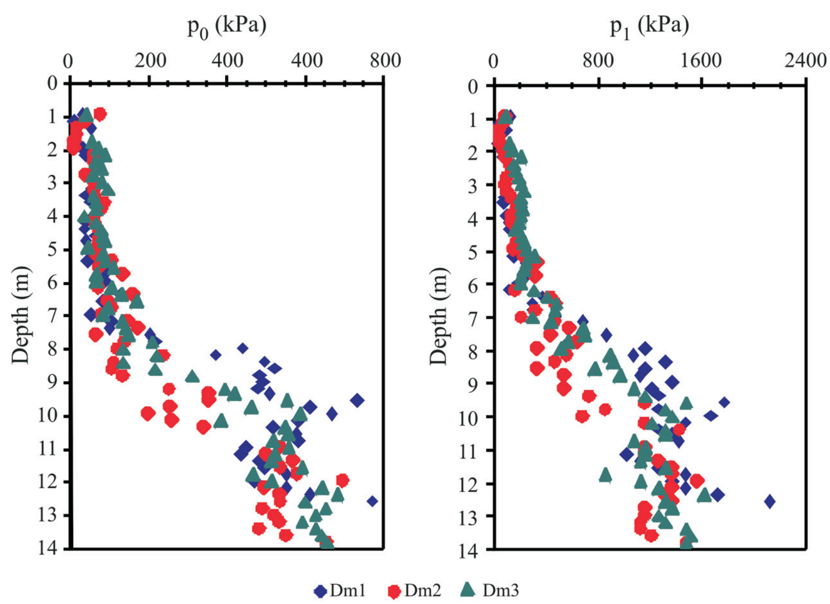


**Figure 4** - Typical DMT results.

The cone penetration tests (CPT) were advanced with the same hydraulic rig. CPTs 1-14 were conducted with a standard electronic cone - 60° tip with area of 10 $cm^2$ - and CPTs 15-17 were conducted with a piezo-cone. The tests were conducted with a penetration rate of 2 cm/s (ASTM D-5778). The inclination was measured and the test was stopped if it became excessive (above 15 degrees). The cones were calibrated at the national Laboratory of Furnas in Goiânia-GO. Results of 4 CPT tests are given in Fig. 5.

The standard penetration tests (SPT-T) were conducted according to NBR-6484, and a manual hammer was used. The test used a four-legged frame with a winch on one side. The hammer was a long (H ≈ 2D) pin-guided type that was raised by 2 labourers pulling on the cables used to lift the hammer. After the SPT, the torque was measured with a calibrated torque wrench at a set rate (for both the peak and residual values) as presented for the typical peak result in Fig. 2. Results of the number of blow counts of all SPT tests are presented in Fig. 6.

The Menard pressuremeter tests (PMT) were conducted according to ASTM D-4719 to obtain a pressure-deflection curve and gave the strength and deformability parameters of the soil, as well as the insitu horizontal stress. The test was usually run in increments of 25 kPa. An example test is shown in Fig. 7, based on a "curve matching" procedure (see Mota, 2003 and Fontaine *et al.* 2005).

Two shafts were excavated for geological investigations of the soil profile as depicted in Fig. 3. Triaxial tests were conducted earlier on block samples from depths of 3 m, 6 m, and 9 m. At each depth $CK_oD$ tests were conducted at cell pressures of 50 kPa, 100 kPa, and 200 kPa and the values of the initial modulus $E_i$ and the tangent modulus at 50% of the failure stress $E_{50}$ were found. These values were interpolated to the stress conditions at each depth. For this paper these three values were averaged, giving $E_i$ = 6.6 MPa and $E_{50}$ = 3.7 MPa.

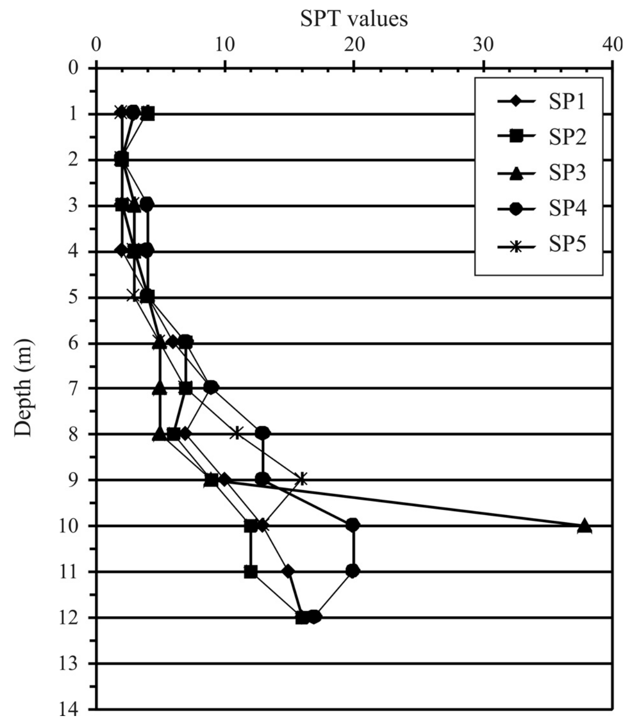


**Figure 6** - Summary of SPT blow count results.

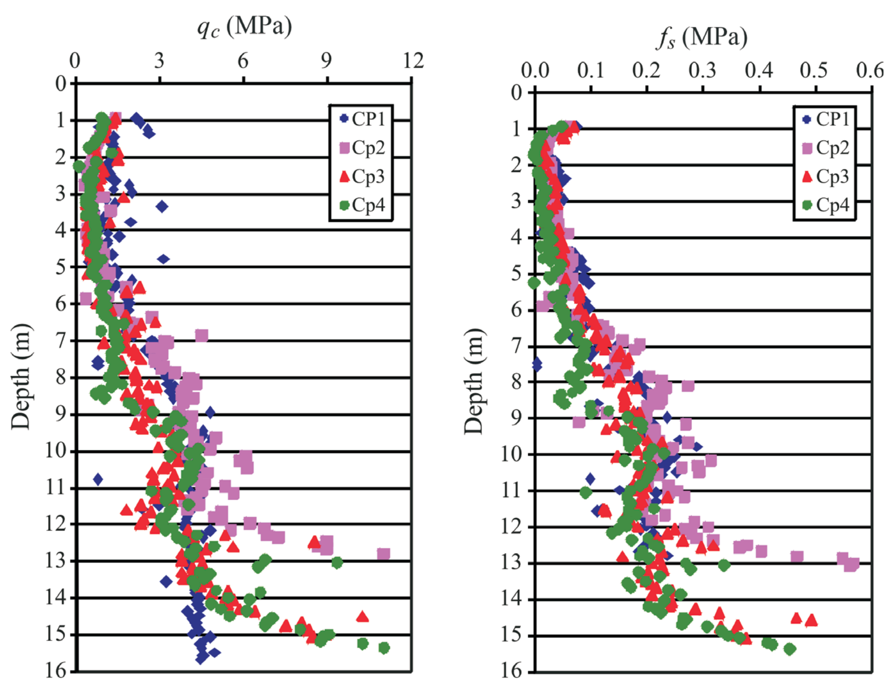


**Figure 5** - Results of four CPT profiles.

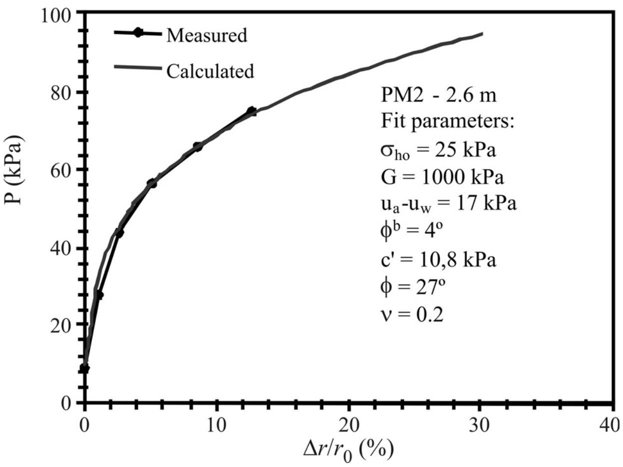


**Figure 7** - Example PMT test result and analytical fitting analysis.

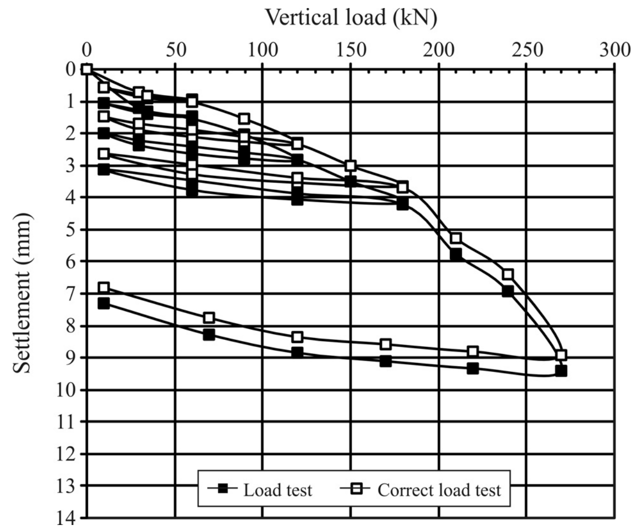


**Figure 8** - Example of load-settlement curve of test pile.

## 4. Pile Load Tests

An earlier series of test piles had been conducted at this site, also following the NBR-12131. For this paper a series of five piles were constructed and are noted as E-1 to E-5 on Fig. 3. It was planned to install internal instrumentation (strain gauges in all of the piles and load cells at the base of E-1, 2 & 4). However the soil squeezed inwards at the base of piles E-2 & 4, and the instrumentation could not be installed. The difficulties have been noted for later tests. For piles E-3 & 5, the instrumentation was not installed. The instrumentation in pile E-1 provided reasonable results, proving that this pile, and by analogy the others, behaved as a *floating* foundation.

The soil was excavated with mechanical augers to a diameter of 30 cm. The pile lengths were 7.25-7.85 m. After a re-bar cage was installed, the borings were filled with ready-mix concrete. Cylinder samples were obtained for later testing. After the concrete had hardened, a smooth-faced concrete block was installed at the top of each pile. Pile Echo tests (PET) with a new acquired equipment were recently conducted on each of the piles to confirm the absence of voids or reductions in cross-section.

Reaction piles with a diameter of 0.5 m and depth of 10 m were installed to hold the metal beams that provided the support for the load tests. For the tests a hydraulic jack, a load cell and extensometers were attached to the head of the pile. Six extensometers were used, each with a travel of 0.05 m and a sensitivity of $10^{-5}$ m. Static load tests were carried out in progressive stages. The load-settlement plots were manually adjusted for any apparent settlement of the loading equipment. A typical example is shown in Fig. 8.

## 5. Analysis of Pile Settlement

Modulus values were selected for each type of field test. The values of the field measurements varied with depth, but were averaged (neglecting extreme values). Most correlations consider sand and clay values separately. Since the soil is partly saturated, undrained (clay) values were not used. Many authors consider sand correlations to vary widely depending on stress history and age of the sand deposit. The soil dates from Tertiary-Quaternary era and is aged soil. The water table is below the pile depth and the soil is likely somewhat overconsolidated due to variations in the soil suction. Simple correlations were adopted from references:

• Baldi *et al.* (1986) indicate that theoretically $E_{25} = (1 - \nu^2)E_D$, but gave an empirical relation of $E = 0.88E_D$, where $E_D$ is the dilatometer modulus.

• Robertson and Campanella (1988) suggested $E = 6$ to $10q_c$ and a value of $8q_c$ was used, where $q_c$ is the cone tip resistance.

• Poulos (1998) related SPT N-values to the modulus along and below a pile as 3 N, where N is the SPT blow counts for 30 cm.

• After standard corrections, the PMT data was plotted and curve "matched" via the methodology and cavity expansion model proposed by Cunha (1996). This original model was later modified for cohesive-frictional materials by Fontaine *et al.* (2005), and the model used herein is this modified version. Hence, a number of soil parameters were fit into their model and were adjusted to match the field curve, giving a modulus value, E, for each test. This modulus is derived from the shear modulus G obtained for the elastic zone around the pressuremeter. The E values were averaged for the pile analysis.

• Values of $E_i$ and $E_{50}$ from the triaxial tests were used directly in the analysis.

As required, the averaged field data were converted to $E$ values, using the correlations and techniques given in Table 3.

These modulus values for each soil test were then used in the Poulos & Davies (1990) solution to calculate

**Table 3** - Correlations & techniques used for moduli assessment.

| Test | Reference | Formulation |
|---|---|---|
| DMT | Baldi *et al.* (1986) | $E_{25} = 0.88E_D$ |
| CPT | Robertson & Campanella (1988) | $E = 8q_c$ |
| SPT | Poulos (1998) | E(MPa) = 3 N |
| PMT | Fontaine *et al.* (2005) | Curve fitting |
| Laboratory | Triaxial CK0D tests | $E_i$ and $E_{50}$ |

settlement. The predicted values were then compared to the measured values. The ratio of the predicted values to the measured values are presented in Table 4 (together with working loads and settlements) and plotted in Fig. 9.

The simple (elastic) model used allowed a straightforward comparison of the settlement predictions. In Table 4, it can be seen that the pressuremeter and SPT modulus values seem to provide the best estimates of the pile settlements, followed by the CPT. The DMT and lab values over-predict considerably the pile settlements.

## 6. Conclusions

Simple elastic models can be routinely used in practice for the estimation of the settlement of bored floating piles on tropical unsaturated soils. Although limited in terms of data, the results tend to indicate that PMT tests provide the best ratios between predicted and measured data. As a general conclusion it can be said that more research emphasis must be placed on this matter, so that this versatile *in situ* tool becomes more readily used in practice.

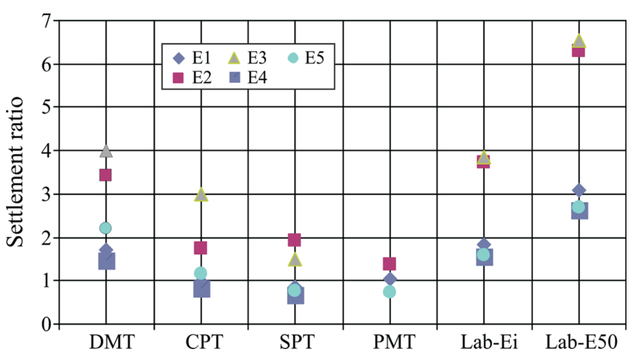


**Figure 9** - Plot of settlement ratio for various tests used for modulus.

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**Table 4** - Measured and predicted settlements using Poulos & Davies (1990) solution.

| Pile | $P_{load}$ (kN) | $\delta_{measured}$ (mm) | Settlement ratio (predicted/measured) | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | DMT | CPT | SPT | PMT | Lab-$E_i$ | Lab-$E_{50}$ |
| E1 | 135 | 2.4 | 1.72 | 0.98 | 0.86 | 1.03 | 1.82 | 3.08 |
| E2 | 180 | 1.56 | 3.43 | 1.73 | 1.92 | 1.39 | 3.73 | 6.31 |
| E3 | 135 | 1.13 | 3.99 | 3.00 | 1.49 | - | 3.86 | 6.53 |
| E4 | 130 | 2.70 | 1.47 | 0.84 | 0.66 | - | 1.56 | 2.63 |
| E5 | 155 | 3.14 | 2.21 | 1.15 | 0.75 | 0.72 | 1.59 | 2.70 |
| Average | | | 2.56 | 1.54 | 1.14 | 1.05 | 2.51 | 4.25 |

Note: $P_{load}$ = working load, $\delta_{measured}$ = measured settlement at working load.

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