CPT and T-bar Penetrometers for Site Investigation in Centrifuge Tests

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Abstract. Geotechnical design is based on site investigations which provide a reasonable overview of the soil profile and a realistic estimate of the geotechnical properties of each component layer. Nevertheless, when centrifuge modelling is involved, *in situ* tests become an additional challenge mainly because of limitations of in-flight procedures, but also due to the miniaturization of regular tools. As centrifuge modelling is becoming widespread, mostly as a result of decreasing electronic and computer costs, miniature site investigation tools are being designed to provide proper geotechnical information about model layers. This paper examines the development of site investigation tools to assess the strength of models during centrifuge tests. These tools are a T-bar penetrometer and a Cone Penetration Test (CPT) apparatus capable of measuring the resistance of clay and granular soils, respectively. These tools were used in a number of centrifuge tests on clay soils and silty tailings respectively. Both tools were tested and the results compared with centrifuge tests, *in situ* conventional tests, triaxial and direct shear laboratory tests showing an overall consistency and reliability of the measured data.

Keywords: penetrometer, CPT, T-bar, centrifuge test, soft clay, mine tailings.

1. Introduction

Physical modelling plays an important role in modern geotechnics as it aims to create a scaled model able to provide a physical understanding of a phenomenon associated with a real problem.

Within physical modelling, centrifuge modelling has becoming increasingly important due to its flexibility regarding the simulation of various engineering problems while keeping critical parameters invariable. The basic principle in centrifuge tests consists of submitting a reduced model (N times smaller than the prototype) to an acceleration N*g*, thus providing an inertial field similar to the gravitational field experienced by the prototype (Schofield, 1980).

Advances in centrifuge research have led to the need for a reliable resistance profile of the soil models, leading to the conception of in-flight penetration tests in order to describe the variation in the soil properties with depth.

A major difficulty in carrying out in-flight tests is the miniaturization of tools and their actuations. As a result, routine procedures such as SPT, for example, can become extremely complex. Early developments in in-flight site investigation in centrifuge tests made use of the vane test and cone penetration test (*e.g.*, Almeida & Parry, 1984, 1987; Esquivel & Ko, 1995) to measure the undrained strength of soil models and also sands (*e.g.*, Almeida, 1984; Bolton *et al.* 1999). Subsequent developments used the T-bar (Stewart & Randolph, 1991) to assess the undrained strength of clay soils. This paper presents the experience of the development of T-bar and CPT probes to measure the strength of models used in the mini-drum centrifuge at the Alberto Luiz Coimbra Institute – Graduate School and Research in Engineering (COPPE) in Rio de Janeiro. Test results and their interpretation are presented for clay and silty tailings soils.

2. Coppe'S Geotechnical Centrifuge

The COPPE geotechnical centrifuge (Gurung *et al.*, 1998), shown in Fig. 1, is a 1.0 m diameter mini-drum with a full load capacity of 90 *g*-ton. It comprises 20 slip rings, 16 data acquisition channels, a linear actuator, and a turntable on which the linear actuator is mounted.

The COPPE centrifuge has been used in studies on pipeline movements (Oliveira, 2005; Pacheco, 2006), stability of solid waste fills (Calle, 2007), and the behaviour of iron tailings materials (Motta, 2008).

A strongbox with dimensions 260 mm length, 210 mm width, and 178 mm height has been used for the tests performed so far in COPPE's centrifuge. Use of the whole centrifuge channel is also possible but such a procedure would require a very large amount of soil to be tested. A new, larger strongbox is now in use for pipeline studies.

Currently the main tool used in the centrifuge for clayey soils is the T-bar penetrometer. Its use is similar to that of the cone with the advantage that it does not need any area correction as the soil resistance is obtained directly from a simple equation. The use of the T-bar (Stewart & Randolph, 1991) is indicated only for clay soils, once the theoretical interpretation of the mobilized resistance was deduced solely for this type of soil. The tool considered

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Figure 1 - COPPE/UFRJ Geotechnical centrifuge.

most suitable to evaluate the behaviour of granular soils is the cone penetration test (CPT). This paper examines the use of the T-bar and the CPT for the site investigation of clay and granular type soils respectively.

3. T-Bar Tests In Clay Soils

A T-bar with a 15.2 mm diameter adapted to a rigid shaft was used in the present study (Fig. 2). The vertical force was measured using a manufactured 50 N tension and compression load cell, which compensates bending moments and thermal variations (Fig. 2). One pore-pressure transducer was positioned inside the soil layer for monitoring the consolidation phase.

3.1. Reconstituted clay and sample preparation

The natural clay from Guanabara Bay in Rio de Janeiro/RJ consists of lightly overconsolidated highly compressible soft clay with water content in the range 150%-200% and close to the liquid limit. A number of tests were used to assess the *in situ* undrained strength of the natural clay and it was found by Almeida *et al.* (2001) that the undrained strength profile (Fig. 3) could be well described by Eq. (1), in kPa, where z is the depth in m.

$$S_u = 0.126 + 1.373z \quad (1)$$

For the centrifuge tests described below the natural clay was collected *in situ* and transformed into slurry by increasing the water content up to 1.5 times the liquid limit. The slurry was then placed in-flight inside the strongbox through a specially designed rotating joint. After the consolidation, this process produced a smooth and regular surface, adequate for the shallow tests.

During the 10 h consolidation flight, the clay slurry settled down from a 105 mm height into a 71 mm clay layer height. Figure 4 shows the measured pore pressures 10 mm above the clay layer bottom. A specially prepared program (Oliveira, 2005) has been developed, based on Terzaghi theory but combined with large deformation and centrifuge issues, in order to calculate total and effective stresses, pore pressure, water content and undrained shear strength variations, throughout the layer, during the tests at the centrifuge particular conditions. A pore pressure dissipation prediction curve has been added to Fig. 4, based on the clay parameters and the consolidation conditions.

The parameters of the reconstituted natural clay are summarized in Table 1. Figure 5 presents a final water content profile in prototype (real) scale.

Each test was divided into two phases: consolidation at 100 *g* followed by vertical and lateral actuations at 30 *g*. All samples reached around 90% consolidation during 10 h flight (Oliveira *et al.*, 2006). Enough time was allowed for pore-pressure dissipation during the centrifuge decelera-

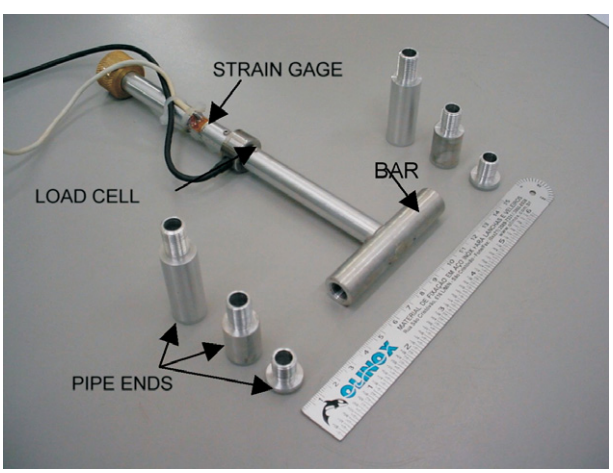

Figure 2 - Instrumented T-bar.

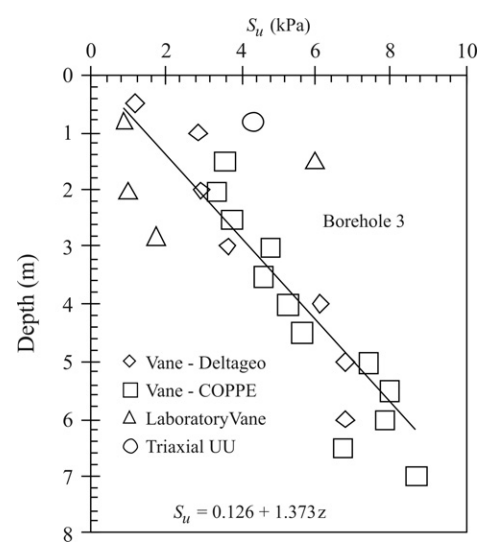

Figure 3 - Undrained strength profiles for a representative borehole (Almeida *et al.*, 2001).

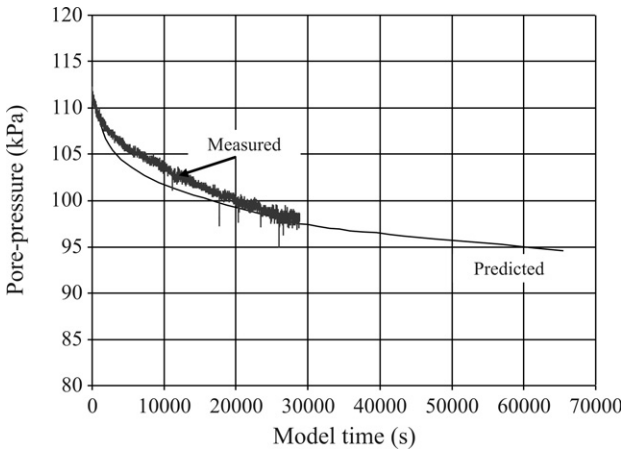

Figure 4 - Measured and predicted pore-pressure dissipation during consolidation.

tion from 100 *g* to 30 *g*. After that, the T-bar was driven into the soil (Fig. 6). The vertical penetration of the T-bar allowed the measurement of the undrained strength and this is described next.

3.2. Vertical actuation

As the bar penetrates the soil, the whole setup (Fig. 2) can be employed as a T-bar penetrometer (Stewart & Randolph, 1991), and the load cell measurements can be used to estimate the undrained shear strength of the soil profile. The following equation is used to obtain S_u from T-bar measurements:

$$S_u = \frac{V}{N_b \, D^* L} \tag{2}$$

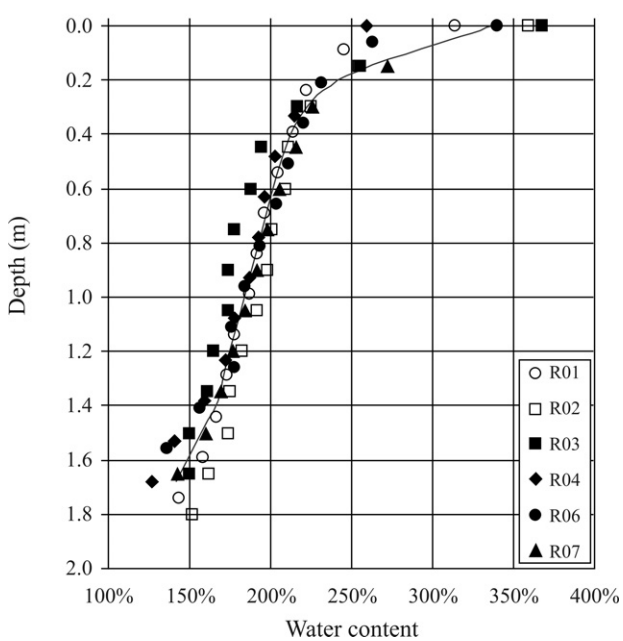

Figure 5 - Water Content Profile for clay centrifuge tests (prototype scale).

Table 1 - Summary of the reconstituted clay properties.

Soft clay properties	Data
Liquid limit w_L	174%
Plasticity Index I_p	90%-120%
Solids specific weight G_s	2.60
Bulk weight (*)	12.0 kN/m^3
Voids ratio e	3.6-4.5
CR = C_c / (1 + e_0)	0.36
OCR	≈ 1.3
Coefficient of consolidation c_v	4.8 x 10^{-8} m^2/s

where V is the vertical force measured during penetration, L is the T-bar length, and N_b is the T-bar factor, with $N_b = 10.5$ the recommended value (Randolph & Houlsby, 1984; Randolph, 2004) for deep penetration. Shallow depths require different T-bar factors for each depth. In addition, the contact area increases as the bar is pushed into the soil, also increasing the amount of material involved in the failure process. For deep bar penetration D^* is the bar diameter D. For shallow embedment ratios a modified bar diameter D^* is used once the contact area between bar and soil varies with depth, and it has a direct relationship with strength. To take this variation into account, the following relation was used to express the horizontal projection of the contact area of the bottom half of the bar with the soil (Fig. 7).

$$D^* = 2\sqrt{H(D-H)} \tag{3}$$

where H is the distance between the soil surface and the bottom of the bar.

When the bar is just touching the soil surface, the cylindrical shape can be assumed to have a flat plate foundation, therefore associating this condition with Terzaghi's bearing capacity factor $N_b = 5.14$ for a purely cohesive ma-

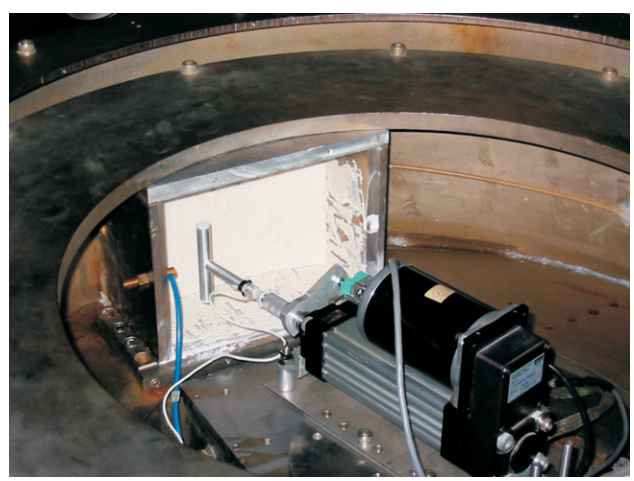

Figure 6 - T-bar in position for actuation phase.

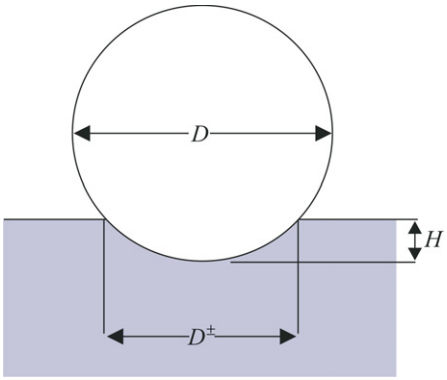

Figure 7 - Horizontal projection of the bar bottom contact area with soil.

terial. However, it is important to evaluate the burial depth at which the T-bar factor reaches its full value and how this variation develops. A numerical approach is proposed below.

3.3. Numerical simulation to evaluate N_b

The vertical penetration phase was numerically simulated (Oliveira, 2005) in order to evaluate the variation in the T-bar factor N_b with the embedment ratio H/D. In this way, numerical analyses have been carried out with the embedment ratio H/D varying from 17% to 600%. In each case, the T-bar was pushed into the soil until yield of the soil was achieved.

A finite element code for geotechnical applications (Costa, 1984), incorporating geometric and physical non-linearities with iterative-incremental integration algorithms, was used. For the soft clay soil the elastic perfectly plastic Von Mises model was adopted, with $E_u = 300\ S_u$ (Almeida & Marques, 2002), $\nu = 0.5$, and a unitary strength profile of $S_u = 1$ kPa, which makes it easier to normalize the vertical force against S_u. In these finite element analyses using the code developed by Costa (1984), three adhesion factors α were used to simulate the soil-bar interface: $\alpha = 1.0$, $\alpha = 0.5$, and $\alpha = 0.2$. Figure 8 shows the displacement vectors output for the numerical simulation of a fully buried T-bar penetrometer.

Figure 9 presents the numerical results of the T-bar factor (N_b) for the three adhesion factors computed with Eq. (2) using the numerical value of V, S_u equal to unity, and D^* defined by Eq. (3). The results in Fig. 9 indicate that the T-bar factors show a major variation for H/D in the range 0%-300% and just a minor increase for H/D greater than 400%. The initial value is around 5.24, which is close to the expected Terzaghi's bearing capacity factor, 5.14. The final value for the same adhesion factor is around 10.5, which is the same value as that proposed by Randolph (2004). A similar range of values has also been obtained by Barboza-Cruz & Randolph (2004) using a remeshing technique for smooth and rough cylinder surfaces.

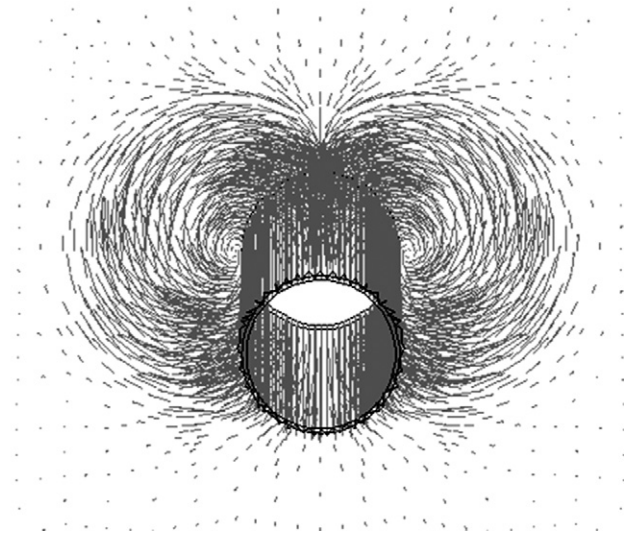

Figure 8 - Numerical displacement vectors for a 500% buried T-bar (Borges *et al.*, 2005).

3.4. Undrained strength from T-bar tests

T-bar tests were interpreted using Eq. (2), with N_b factors provided by Fig. 9, as shown in Fig. 10. The penetration rate used in all tests was 0.50 mm/s. The linear fit through S_u data of the five tests with H/D varying between 17% and 124% is given by the equation below (Oliveira *et al.*, 2006) with a linear coefficient of correlation greater than 0.99.

$$S_u = 0.1002 + 1.283z \tag{4}$$

with S_u in kPa and depth z in m. Values of S_u obtained according to Eq. (4) compare reasonably well with the *in situ* values given by Eq. (1), although the penetration depth of the model tests (around 1.0 m) is much smaller than the penetration depth of the field tests (around 6.0 m). As the sample has been consolidated at 100 g, and the penetration phase has been done at 30 g, the S_u/σ'_{v0} ratio of 0.64 is compatible with that of the Rio de Janeiro clays for an OCR ≈ 3 (Almeida, 1982).

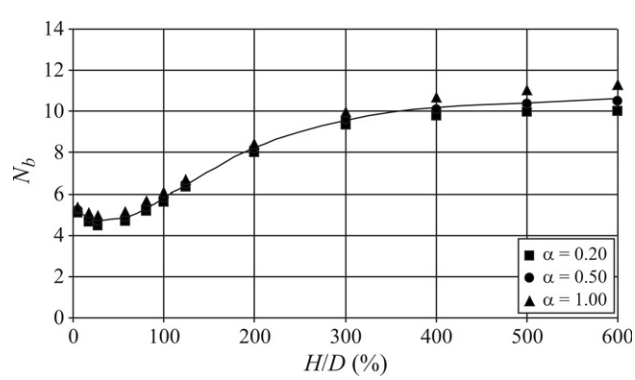

Figure 9 - Variation of the T-bar factor (N_b) with burial depth.

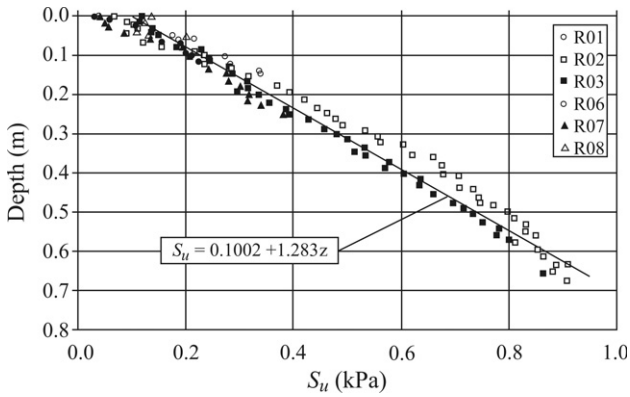

Figure 10 - T-bar tests data in prototype scale.

In an attempt to overview the whole S_u behaviour, all *in situ* and laboratorial undrained strength results on undisturbed samples have been plotted against their respective water content values on Fig. 11. In addition, centrifuge tests strength profiles on reconstituted samples have also been included in the same plot.

However, some corrections were necessary, since field samples are undisturbed, whereas centrifuges samples were reconstituted and tested in 15 h. Almeida & Marques (2002) report sensitivity values up to 4.4 measured in vane tests. Reconstituted samples used in the centrifuge are consolidated, *i.e.*, some restructuration is allowed, which needs to be taken into account. Therefore an average sensitivity value of 2.0 was adopted as a multiplying factor for the centrifuge values.

Using the critical state soil mechanics equation, which associates undrained shear strength with water content, and adopting the critical state parameters, for the same soil, obtained by Almeida (1982), a theoretical curve, based on Eq. (5), was plotted on the chart of Fig. 11.

$$S_u = 0.59e^{\left(\frac{5.80-2.60w}{0.90}\right)} \quad (5)$$

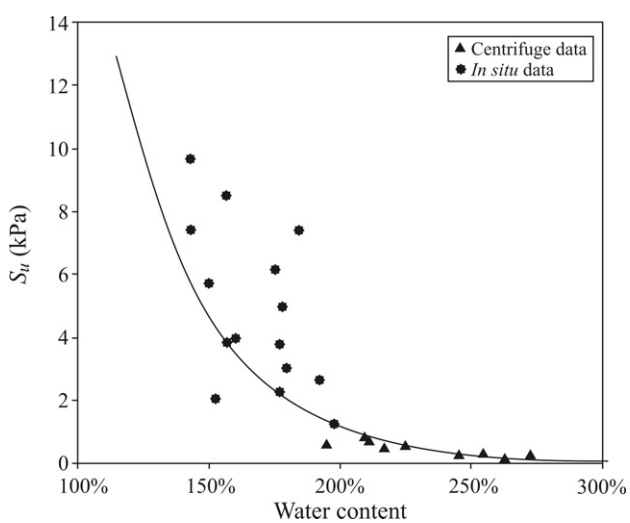

Figure 11 - Consolidation of centrifuge and *in situ* undrained shear strength versus water content data.

The centrifuge and the *in situ* data shows good agreement with the critical state theory curve, indicating that the procedures adopted for the shear strength analysis of the T-Bar measurement con-ducted to a set of reasonable values.

Figure 12 shows the centrifuge and the *in situ* liquidity index and shear strength data. These values compared well with Wood & Wroth (1978) equation keeping a clear linearity.

These clay beds have been subjected to lateral actuation of pipelines and the results of these tests are shown in Oliveira *et al.* (2005) and Oliveira *et al.* (2010).

4. Cpt Tests In Fine Tailings

4.1. Characteristics of the fine tailings

The fine tailings studied in this work came from the exploitation of iron ore by Samarco Mineração S.A., located in the city of Mariana, State of Minas Gerais, Brazil. The main minerals present are haematite, goethite (limonite), and magnetite. For the purpose of the tests carried out here the fine tailings were dried in an oven and homogenized to obtain representative samples. The grain size analysis resulted in the following percentages: clay 7%, silt 71%, and fine sand 22%. The X-ray diffraction indicated predominance of haematite Fe_2O_3 and quartz silica, confirmed by the chemical analysis, which resulted in 40.9% Fe_2O_3 and 53.6% silica.

The fine iron tailings were found to be non-plastic. Geotechnical properties of the studied soil are: specific gravity $G_s = 3.22$; minimum dry density = 1.36 g/cm^3; maximum dry density = 2.16 g/cm^3; field density = 1.6 to 2.2 g/cm^3 (average 1.97 g/cm^3); coefficient of consolidation c_v = 0.5-3.0 x 10^{-6} m^2/s (average 1.4 x 10^{-6} m^2/s); coefficient of permeability = 5-8 x 10^{-6} m/s; and compression ratio CR = $C_c/(1 + e_o)$ = 0.05, characteristic of a low compressibility soil.

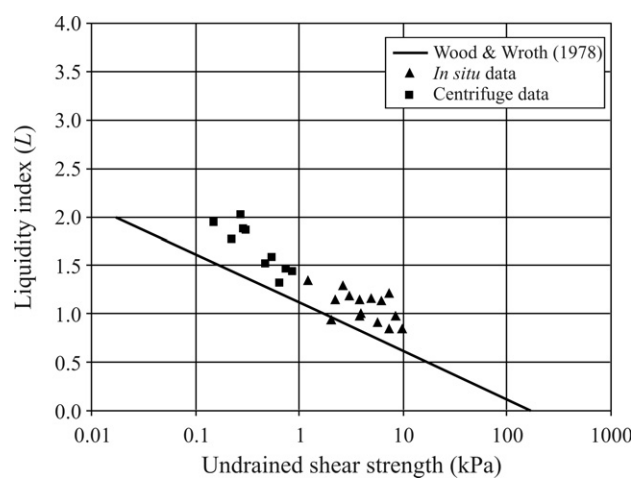

Figure 12 - Comparison between liquidity limit and undrained shear strength.

4.2. CPT design and assembly

The design of the mini-CPT penetrometer had to take into account a number of factors such as the maximum driving capacity of the radial centrifuge actuator (2000 N), the measuring capacity of the load cells (125 N), the maximum travel length of the tool (less than 18 cm), and the high resistance of the soil material to be tested. Also, the tool should be as light as possible, but still capable of measuring the soil resistance.

The mini-CPT was designed to measure the point load Q_b plus the total load Q_t which is the sum of Q_b and Q_s, the lateral load. Thus, the mini-CPT was designed with a 9 mm cone diameter, a 5 mm internal shaft diameter, 165 mm total length (including total load cell), and approximately 70 mm of free driving shaft (Fig. 13). These dimensions took into account the possibility of buckling of the CPT shaft, predicted according to Euler's formulation. The total weight of the mini-CPT including load cells was 323 g.

A general view of the developed mini-CPT can be seen in Fig. 14, where the location of the load cells installed in the equipment is shown. The point load cell positioned inside the metallic body is attached to the inner shaft and the cone tip. The total load cell bears both the point load and the shaft load.

4.3. Modelling of models

New centrifuge tests tools are usually verified using the "modeling of models technique" (Schofield, 1980).

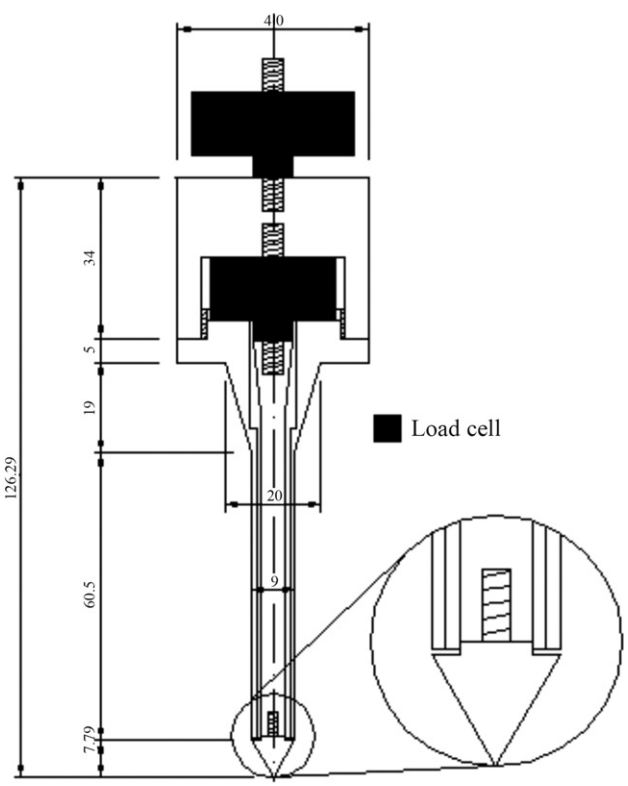

Figure 13 - Mini-CPT schematics (measurements in mm).

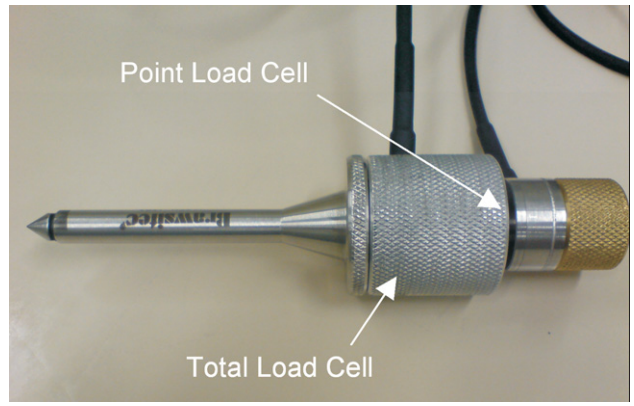

Figure 14 - General view of mini-CPT.

This procedure is realized by carrying out tests with different accelerations.

The tests were carried out using the tool in the silty tailings materials. The soil layers were moulded inside the strongbox with the centrifuge fully stopped. After that, the equipment was set to spin and the samples were consolidated at 50 *g* for 30 min, which is enough time to allow full consolidation. The final layers had a total height of 9 cm and average dry density equal to 1.80 g/cm^3 (relative density of 55%).

These test penetration velocities were standardized with the normalized velocity $V = 0.5$, defined by the expression (Finnie & Randolph, 1994):

$$V = \frac{vD}{c_v} \tag{6}$$

where v is the rate of cone penetration, D is the cone diameter, and c_v is the coefficient of consolidation. This value of $V = 0.5$ assures a fully undrained behaviour. The maximum penetration in the model was 6 cm, which corresponds to prototype depths of 1.5 m, 3.0 m, and 4.5 m for the 25 *g*, 50 *g*, and 75 *g* accelerations respectively.

Figure 15(a) shows the model scale point resistance profiles for test accelerations of 25 *g*, 50 *g*, and 75 *g*. Figure 15(b) presents the same point resistance profiles in prototype scale. All tests show good agreement, except for a slight deviation to the right in the 25 *g* test. This apparent increase in soil resistance is related to the overconsolidation conditions for the 25 *g* test, once the consolidation phase took place at 50 *g*. The good agreement observed in Fig. 15 is in accordance with a successful modelling of the model's procedure. Similar results were previously obtained for sands (Almeida, 1984).

Almeida (1984) used an in-flight mini-CPT apparatus to assess the strength profile of a Leighton Buzzard 30/52 uniform medium sand layer at centrifuge accelerations of 25 *g*, 50 *g* and 100 *g*. The samples were carefully prepared by placing the sand with a scoop, in submerge conditions, reaching a relative density of 47.8% and a void's ratio of 0.668. Figure 16 shows the results obtained by Almeida

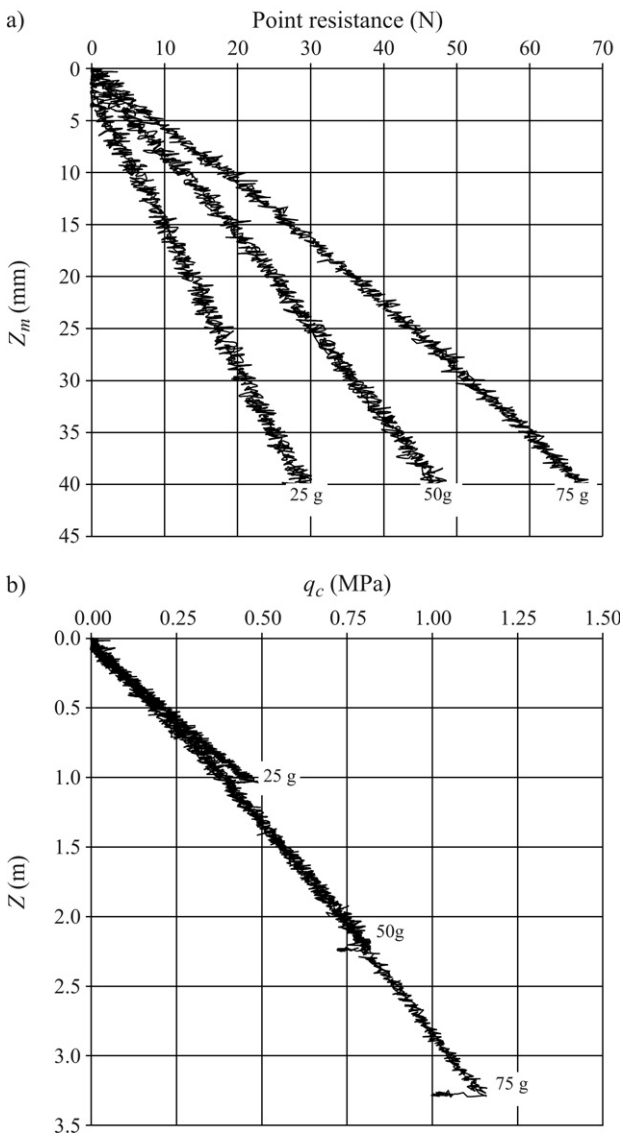

Figure 15 - Mini-CPT modelling of models tests with silty tailings: (a) point resistance in model scale and (b) q_c in prototype.

(1984), which are very similar to those described in this work, despite of the different nature of the tested materials.

4.4. Strength parameters of the silt tailings

Strength parameters obtained from the mini-CPT tests are compared herein with the strength parameters measured in direct shear tests and triaxial tests, which are described first.

4.4.1. Direct shear tests on centrifuge samples

The preparation of centrifuge samples was similar to preparation of the samples for the mini-CPT test. After the consolidation stage and the complete drainage of the water, the centrifuge was halted and placed in a 90° position so that five test specimens could be extracted as shown in Fig. 17. The direct shear test moulds were inserted at a

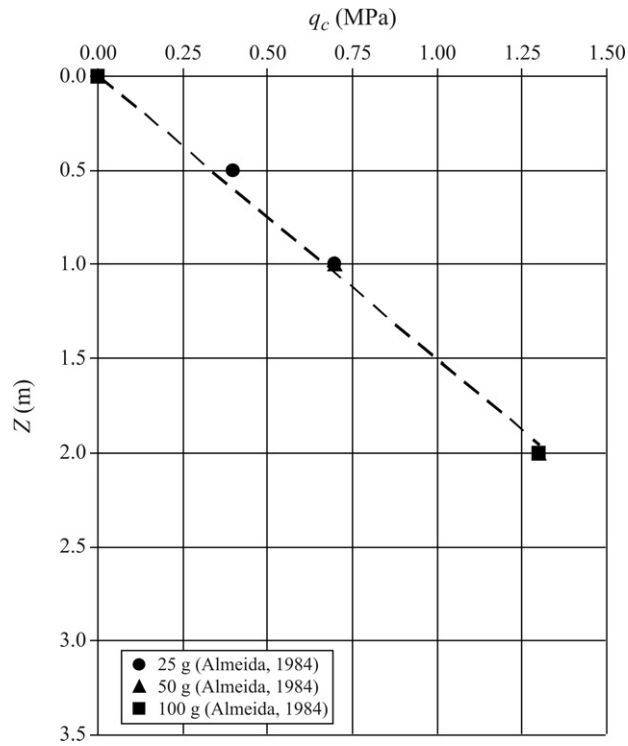

Figure 16 - Mini-CPT modelling of models tests for Leighton Buzzard sand (after Almeida, 1984).

depth corresponding to twice their height in order to avoid surface interference.

The direct shear tests were carried out at vertical stresses of 100, 200, 300, and 400 kPa on saturated specimens and the results are shown in Table 2. These data presented negligible dilation and a mean effective friction angle $\phi' = 31.5°$. However strength data at low stress levels relevant to the centrifuge tests give $\phi' = 34.6°$ and this is the value to be compared with friction angles obtained from CPT tests performed in the tailings soil.

4.4.2. Triaxial tests

A set of CD triaxial tests were undertaken in silty tailings samples statically compacted at the optimum water content. The mean dry density obtained with this process was $\rho = 2.09$ g/cm^3. The confining pressures adopted for the tests were 100, 200, 300 and 400 kPa. The volumetric strain behaviour shows an overconsolidated material with an initial increase in volume followed by a small decrease. Figu-

Table 2 - Data from direct shear tests in the tailings soil.

Test	σ (kPa)	τ (kPa)	τ/σ	ϕ' (º)
1	115	80	0.69	34.6
2	232	147	0.63	32.4
3	350	197	0.56	29.4
4	462	260	0.56	29.4

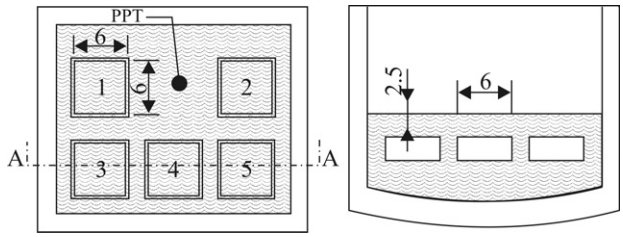

Figure 17 - Location of the direct shear specimens: (a) plan view and (b) cross section AA (dimensions in cm).

re 18 presents the triaxial CD results, displaying an internal friction angle of 41° and no cohesion.

4.4.3. Friction angles from CPT tests

A number of authors have developed theories or correlations between CPT tests and friction angles for non-cohesive soils. The methods proposed by Durgunoglu and Mitchell (1975) and Robertson & Campanella (1983) are quite often used (Schnaid, 2009; Lunne *et al.*, 1997) to estimate friction angles from CPT data. The method by Durgunoglu & Mitchell (1975) is based on bearing capacity theory. The method developed by Robertson & Campanella (1983) is based on correlations with CPT tests performed in calibration chambers on normally consolidated sands of medium compressibility. Both methods are based on the ratio between the measured point resistance q_c and the vertical *in situ* stress σ_{vo}. For $c' = 0$ soils, the bearing capacity factor N_q is equal to q_c/σ'_{v0}, which is the ratio between the measured point resistance q_c and the vertical effective stress σ'_{v0}. Chen & Huang (1996) have expressed these two methods using the equation

$$\tan\phi' = \frac{1}{C_1}\ln\left(\frac{\frac{q_c}{\sigma'_V}}{C_2}\right) \tag{7}$$

where the coefficients C_1 and C_2 are expressed as shown in Table 3.

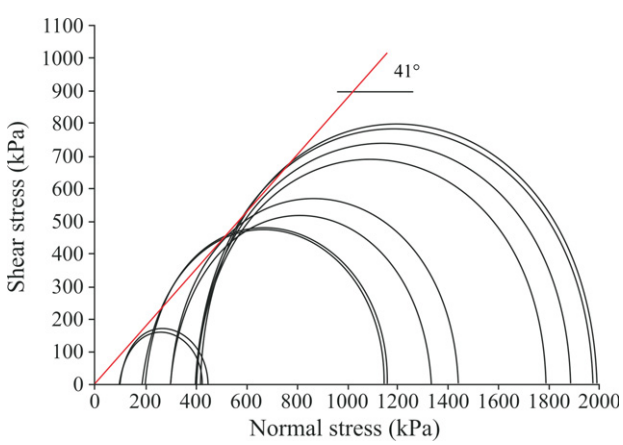

Figure 18 - Triaxial CD tests in silty tailings samples.

Table 3 - Coefficients for q_c-tan f ' correlations.

Method	C_1	C_2
Durgunoglu & Mitchell (1975)	7.629	0.194
Robertson & Campanella (1983)	6.820	0.266

According to Chen & Huang (1996), Durgunoglu & Mitchell's method (1975) is suitable for low compressibility sands and Robertson & Campanella's method (1983) is suitable for medium compressibility sands. Although the above methods have been developed for sands, they will be applied for the non-plastic silty soil studied here. This is a soil with granular behaviour ($c' = 0$), and thus this application appears to be reasonable.

The 50 *g* test was used to estimate the soil friction angle. Table 4 summarizes the values of point resistance and σ'_{v0} obtained at the prototype depths 0.5, 1, 1.5, and 2 m. Values of σ'_{v0} considered in Table 4 take into account the small non-linearity of σ'_{v0} with the centrifuge radius (Schofield, 1980) important in small centrifuges. Data of $N_q = q_c/\sigma'_{v0}$ are also shown in Table 4.

Values of friction angles using the two methods are presented in Table 5. It is observed in Table 5 that values of ϕ' appear to decrease slightly with depth and the average friction angles ϕ' obtained by the two methods are quite close and are also in agreement with $\phi' = 34.6°$ obtained in direct shear tests at low stress levels. Higher values in CPT tests for loose samples are expected once a relative compaction of the soil ahead of the cone can increase the friction angle.

Friction angle values associated with the triaxial tests (41°) are higher than those from direct shear and CPT tests.

Table 4 - Point resistance, vertical stresses and bearing capacity factors.

Depth (m)	q_c (kPa)	σ'_V (kPa)	$N_q = q_c/\sigma'_{v0}$
0.5	190.3	5.3	36
1.0	390.1	10.8	36
1.5	579.0	16.4	35
2.0	738.2	22.1	35

Table 5 - Data for friction angles from CPT tests.

Depth (m)	ϕ' (º) - D&M	ϕ' (º) - R&C
0.5	34.4	35.7
1.0	34.4	35.8
1.5	34.3	35.6
2.0	34.0	35.3
Average values	34.3	35.6

This result is probably related with the high relative density (91%) obtained with the static compaction procedure, which is much higher than those obtained in the centrifuge tests (55%).

5. Conclusions

The design and development of instrumentation for centrifuge geotechnical purposes requires the best tools to measure the desired parameter. Also it demands that the variables involved, such as the materials to be tested and equipment limitations, are very well known and controlled.

The T-bar penetrometer developed for this research was used in centrifuge tests to measure the undrained strength of reconstituted samples of Guanabara Bay clay.

Numerical analyses were carried out with the aim of obtaining a variation in the factor N_b with the normalized depth, which was shown to be consistent with the extreme values available in the literature for shallow and deep cases. Based on this formulation the undrained strength profile was calculated for a number of tested samples. The T-bar centrifuge test data carried out for high water content values were complemented by measurements of water content values for the clay.

The measured centrifuge S_u profile agreed well with the field profile obtained by a vane and triaxial tests. Also the relationship between water content and S_u seemed to be coherent with the *in situ* measurements.

Additionally, a comparison between liquidity limit and undrained shear strength show a clear linear behaviour that is very close to the line proposed by Wood & Wroth (1978). All these evidences confirm the overall consistency of the measured data.

The mini-CPT apparatus developed for this research was designed for the specific purpose of testing embankments in silty tailings materials. The tests showed the efficiency of the miniature tool, as well as the possibility of assessing a continuous resistance profile of in-flight layers.

The modelling of the models technique, a highly established procedure to evaluate consistency in centrifuge modelling simulations, was applied in 3 different *g* levels leading to coherent results. Strength parameters were also obtained from the mini-CPT tests and compared with strength parameters measured in direct shear tests and triaxial, leading to consistent results.

The strength profile was also compared with other centrifuge mini-CPT tests in sand soil confirming the expected behaviour.

Finally, both tools developed for in flight strength profile measurements in clayey and sandy soils were tested and the results compared with conventional tests showing an overall consistency and reliability of the measured data.

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