The behaviour of Sarapuí soft organic clay

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ABSTRACT: This paper describes the geotechnical properties of very soft Brazilian clay, situated in a flat swampy and industrialized area, around Guanabara Bay, Rio de Janeiro City. The data reported in this paper is a summary of the geotechnical properties obtained from laboratory and in situ tests carried out in the last 30 years at the Sarapuí site. The deposit is described in terms of its geological origin, index properties, stress history, compressibility, consolidation and strength properties, and viscous behaviour. Field trials executed to assess the above properties and the clay behaviour in the field are also presented.

1 INTRODUCTION

The Sarapuí clay deposit, firstly studied by Pacheco Silva (1953), is situated at the riverside of Sarapuí River and flows through a very populated area, and then reaches Guanabara Bay, about 7 km from Rio de Janeiro City, as shown in Figure 1.

The region around Sarapuí site is quite industrialized with a large oil refinery, a number of petrochemical industries and an important access road to Rio de Janeiro City. A large landfill and a sewage treatment facility are also a few kilometres away from the site. From mid seventies till mid eighties, the Graduate School of Engineering of the Federal University of Rio de Janeiro COPPE/UFRJ, seconded by the Catholic University, PUC-Rio, carried out a number of studies in co-operation with the Transportation Research Institute of the Federal Highway Department, IPR-DNER. Other studies continued to be carried out with less intensity until the present days.

This consistent and comprehensive study transformed the Sarapuí site in the main Brazilian Reference Clay Site, and also provided geotechnical data for the design of a number of important projects in the region around the site.

The studies carried out in the seventies and eighties are the main focus of the present paper. A number of laboratory and in situ tests were carried out and two trial embankments were constructed, one built up to failure and the other with consolidation observations. A trial excavation was also executed. All three trials were analysed by means of analytical and numerical methods. At the end of 2001 an instrumented pile was installed at Sarapuí site and further studies are currently been carried out at COPPE/UFRJ.

2 SARAPUÍ CLAY DEPOSIT

2.1 Geology of Sarapuí clay deposit

Antunes (1978) performed a detailed study of the geology of deposits around Guanabara Bay. Sarapuí clay deposit is a quaternary deposit of fluvial/marine sediments deposited at the lower lands surrounding Guanabara Bay. The deposit was formed some 6000 years ago by sediments
of the erosion of the surrounding mountains carried by tributary rivers arriving at Guanabara Bay and by marine sediments, which were deposited due to sea regression.

Figure 1. Sarapuí experimental site localisation.

Sarapuí clay is an organic marine clay, dark gray, whose OM (organic matter) varies from 4.13 to 5.54 % (Ortigão 1975), and the average pH is 6.6. From mineralogy tests performed on samples collected at Sarapuí riverside, near the experimental site, it was observed that the clay fraction is mainly composed by kaolinite, although illite and/or smectite can also be found in a smaller proportion (Antunes 1978).

The soluble salts content of Sarapuí clay varies from 4.7g/l to 8.5 g/l, molecular relationship  is 2.7, and average values of SiO$_2$ and Al$_2$O$_3$ are 28% and 18.6%, as shown in Table 1.

Table 1. Chemical analysis results (apud Antunes 1978).

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Cations (mE/100g)</th>
<th>pH</th>
<th>Soluble salts (g/L)</th>
<th>Conductivity (S/cm)</th>
<th>Oxids Contents (%)</th>
<th>Ki</th>
<th>OM (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Ca$^{++}$ Mg$^{++}$ Na$^+$ K$^+$ KCL H$_2$O Ca Mg Na K</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>2.0 - 2.5</td>
<td>9.1</td>
<td>16.7</td>
<td>9.7</td>
<td>1.85</td>
<td>6.5</td>
<td>0.40</td>
<td>0.5</td>
</tr>
<tr>
<td>5.0 - 5.5</td>
<td>5.3</td>
<td>15.9</td>
<td>15.3</td>
<td>2.87</td>
<td>7.0</td>
<td>0.19</td>
<td>0.5</td>
</tr>
<tr>
<td>9.8 - 10.3</td>
<td>4.5</td>
<td>9.7</td>
<td>5.9</td>
<td>1.49</td>
<td>6.9</td>
<td>0.30</td>
<td>0.6</td>
</tr>
</tbody>
</table>

Barbosa (1994) carried out chemical and mineralogical analysis on samples collected 3 km downstream of the Sarapuí site. The average OM from those samples was 6%. The salinity of interstitial fluid was 35g/l and its pH varied from 7.6 to 7.8, average CEC (cation-exchange-capacity) of 50. Soil composition obtained from mineral analysis is shown in Table 2. The clay fraction is mainly composed by smectite, which differ from results obtained from mineralogical analysis carried out by Antunes (1978), apparently due to differences on sample localisation. There is consistency between the measured value of CEC and the clay minerals observed (Mitchell 1993).
Table 2. Mineralogical analysis results (apud Barbosa 1994).

<table>
<thead>
<tr>
<th>Depth</th>
<th>Material (%)</th>
<th>Minerals</th>
<th>Clay minerals</th>
<th>clay &lt;2mm</th>
</tr>
</thead>
<tbody>
<tr>
<td>(m)</td>
<td>NaCl OM</td>
<td>Quartz Carbonate</td>
<td>Feldspar</td>
<td>Pyrite</td>
</tr>
<tr>
<td>SPB2</td>
<td>5.0-5.3</td>
<td>18.5 2.5 12.0 1.0 14 12 30</td>
<td>56 74</td>
<td></td>
</tr>
<tr>
<td>SPB3</td>
<td>15.0 3.0</td>
<td>9.0 0.5 12 14 35</td>
<td>61 79</td>
<td></td>
</tr>
</tbody>
</table>

2.2 Soil and clay deposit characterization

Figure 2 shows the Sarapuí clay geotechnical profile based on investigation carried out near the trial embankments sites. An eleven-meter thick, very soft clay layer (NSPT <0), overlying fine to coarse sand layer composes the geotechnical profile. However, investigation carried out near the excavation site (Sayão 1980) showed a thinner layer of clay, of just 3.2m.

![Sarapuí clay profile](image)


Sarapuí clay composition is average of 69% of clay, 18% of silt and 13% of sand. Figure 2 shows its liquid limit (w<sub>L</sub>), plastic limit (w<sub>P</sub>) and natural water content (w<sub>n</sub>) profiles. Sarapuí clay water content is slightly superior to liquid limit, which is a typical behaviour of sensitive clays, however its sensitivity, measured in vane tests (Ortigão & Collet 1986), is around 4.4.

The Sarapuí clay is very plastic and plastic index (I<sub>P</sub>) varies from 34 to 95. The I<sub>P</sub> of organic soft clays must be measured without over drying, since Brazilian experience shows that I<sub>P</sub> tends to drop from 10 to 30% with oven drying.

The initial voids ratio (e<sub>0</sub>) varies from 4.9 to 2.46; density from 2.49 to 2.68 and natural unit weight (γ<sub>n</sub>) varies from 12.5 to 14.5 kN/m<sup>3</sup>. Average parameter profiles proposed by Ortigão
(1980) are in good agreement with data obtained by characterization tests performed for different investigations.

The water table at Sarapuí River, just below ground surface, is influenced by the sea level at Guanabara Bay, thus due to water table variation, an average 1.5 to 3.5m thick unsaturated and desiccated clayey crust can be found at Sarapuí site (Gerscovich 1983).

When compared to the undisturbed clay beneath, the crust presents higher voids ratio and natural unit weight, higher coefficient of consolidation and permeability, but due to its heterogeneity results are quite scattered. The crust structure is similar to that of a disturbed sample and sampling and trimming this material requires especial care, since the clay is fissured. Down until the depth of 2m, the soil is heterogeneous, with shells, and concentration of decomposed roots (until 0.2m) and it becomes more homogeneous with depth (Gerscovich et al. 1986).

The compression curves of the soils at the crust present smaller $C_c$ than the undisturbed clay and passage from overconsolidated to normally consolidated domain is smooth, thus it is difficult to define preconsolidation pressure.

### 3 STRESS HISTORY

#### 3.1 Overconsolidation ratio (OCR)

Sarapuí clay is lightly overconsolidated due to aging and water table fluctuation. Below the crust, average $\sigma'_{p\text{conv}}$ varies linearly with depth, and the overconsolidation ratio (OCR) of the clayey deposit, determined after conventional oedometer tests, varies from 2.0 to 1.3. Values of OCR can be as high as 11 at the crust.

Figure 3 shows the in situ initial stress and preconsolidation pressure ($\sigma'_{p\text{conv}}$) determined after oedometer conventional tests. Initial in situ effective stress ($\sigma_{vo}$) was considered with an average $\gamma_h = 12.9 \text{kN/m}^3$ and water table at ground level, as piezometric data have indicated.

Continuous preconsolidation profile was also obtained from piezocone results ($\sigma'_{p\text{piezocone}}$) using cone factor $N_{so} = 3.4$, as follows:

$$N_{so} = \frac{q_t - \sigma_{vo}}{\sigma'_{p\text{piezocone}}} \tag{1}$$

where $q_t$ is the corrected point resistance and $\sigma_{vo}$ the total vertical stress.

The $\sigma'_{p\text{piezocone}}$ continuous profile is in good agreement with conventional oedometer results. Ortigão (1980) proposed average $\sigma'_{p\text{conv}}$ and OCR profiles that are in good agreement with test results shown in Figure 3.

#### 3.2 Coefficient of earth pressure

Figure 4 shows coefficient of earth pressure ($K_0$) data from: (a) dilatometer tests (Vieira 1994; Vieira et al. 1997) near Experimental Embankment II, using empirical correlation proposed by Marchetti & Crapps (1981); (b) hydraulic fracture tests (França 1976, Werneck et al. 1977); (c) the equation proposed by Mayne & Kulhawy (1982), $K_0 = (1 - \sin \phi'){(OCR)_{en\phi}}$, using the OCR profile shown in Figure 3, and different $\phi'$ value for crust ($\phi' = 30^\circ$) and below ($\phi' = 25^\circ$); (d) $K_0$ variation based on piezocone results (Bezerra 1996), using equation $K_0 = 0.5 + 0.11 \left(\frac{u_1 - u_2}{\gamma'_{vo}}\right) / \sigma'_{vo}$ (Sully & Campanella 1991).

$K_0$ values were obtained from hydraulic fracture tests with $R = \Delta u / \sigma'_{vo}$ being increased and decreased in 10 min. steps and according with Bjerrum’s method of gradual increasing head (Werneck et al. 1977). These results are in accordance with $K_0 \equiv 0.63$, from $K_0 = 0.34 (K_D)^m$, as proposed by Lacasse & Lunne (1988), where $K_D = 4$ and $m = 0.44$, for high plasticity clay.

The $K_0$ profile from laboratory tests, deduced from the empirical equation proposed by Mayne & Kulhawy (1982) also seems to be in accordance with values from hydraulic fracture tests. $K_0$ values from dilatometer tests decrease until 4m depth and then are constant with depth, $K_0$ of about 0.9, higher than values from the other tests.
Bezerra (1996) studied $K_0$ variation based on piezocone results and obtained average values, from 3 to 11 m depth, using different approaches as shown in Table 3. Soares et al. (1986) also obtained $K_0$ range from 0.7 to 0.9, from dilatometer tests performed below 4 m depth. A range of values between 0.6 and 0.9 can be found depending upon the method and test type used.

Discussions on $K_0$ variation with time was carried out by Lacerda (1977), Lacerda & Martins (1985) and Martins et al. (1997), based on tests results performed on Sarapuí clay. Those discussions were related to viscous behaviour of Sarapuí clay and are presented below.
Table 3. Average $K_0$ values from piezocone tests (Bezerra 1996): 3-11m depth.

<table>
<thead>
<tr>
<th>Method</th>
<th>Average $K_0$</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_0 = (1 - \text{sen}^f) \times \text{(OCR)}$ (Mayne &amp; Kulhawy 1982)</td>
<td>0.58</td>
</tr>
<tr>
<td>$K_0 = (K_D / 1.5)^{0.47} \times 0.6$ (Soares et al. 1986)</td>
<td>0.70-0.90</td>
</tr>
<tr>
<td>$K_0 = 0.1 (q_u - \sigma_{vo}) / \sigma_{vo}$ (Kulhawy &amp; Mayne 1990)</td>
<td>0.80</td>
</tr>
<tr>
<td>$K_0 = 0.5 + 0.11 (u_1 - u_2) / \sigma_{vo}$ (Sully &amp; Campanella 1991)</td>
<td>0.65</td>
</tr>
<tr>
<td>$K_0 = f \times (OCR, f_s / \sigma_{vo})$ (Masood &amp; Mitchell 1993)</td>
<td>0.70</td>
</tr>
</tbody>
</table>

4 COMPRESSION

4.1 Oedometer tests

Different authors (e.g. Ortigão 1980, Coutinho 1976, Duarte 1977, Vieira 1988, Barbosa 1990) have carried out consolidation tests in Sarapuí clay. As clay behaviour is affected by sampling disturbance, special care was generally oriented towards obtaining high quality samples. Sarapuí clay is a very compressible soil, which $C_c$ varies from 1.3 to 3.2 and $C_s/C_c$ is 0.12 as shown in Figure 5. The average $C_c/(1+e_0)$ relationship is 0.41, thus an indication that this clay is bound to present important viscous behaviour.

The behaviour of Sarapuí clay under uni-dimensional consolidation is heavily affected by sampling disturbance. Sample disturbance effect on test results is shown in Figure 5 for oedometer tests performed on good quality and bad quality samples and laboratory remolded samples of Sarapuí clay (Ferreira & Coutinho 1988, Coutinho et al. 1998).

The relationship between preconsolidation pressure between good and bad quality samples was found to be as high as 2, although initial voids ratio does not seem to be influenced by sampling quality (Coutinho et al. 1998). Decreases of both compression indexes $C_c$ and $C_s$ and of the coefficient of vertical consolidation $c_v$ were also observed as the soil is destructured with bad quality sampling. Those factors can lead to errors on settlement prediction: for stress state between 1.5 and 2.5 $\sigma_{pcnv}$ settlements calculated using Terzaghi theory are higher for low quality and remolded samples, however, at higher stresses the inverse occurs (Ferreira & Coutinho 1988).

Coutinho (1976) carried out conventional and special oedometer tests with radial drainage and measured coefficient of horizontal consolidation ($c_h$) of samples at normally consolidated domain, and observed decrease of $c_h$ with sample disturbance, and obtained $c_h\text{Undisturbed} / c_h\text{Remolded} = 1.42$ (at 5.8m) and 1.55 (at 6.8m).

However this relationship was found (Almeida et al. 1993a) to be higher for another site some few kilometres away from Sarapuí site, where a 24m high embankment was built on soft clay disturbed in situ. Contrary to planning, only part of the clay was removed, thus the average remaining 2.5m thick clay was disturbed. The $c_v$ variation of the undisturbed samples was $31 \times 10^3$ m$^2$/s to $4 \times 10^3$ m$^2$/s, from overconsolidated domain to just after preconsolidation pressure (from 6 to 200kPa). From tests performed on disturbed samples and from settlement analysis, $c_v$ was constant for this effective stress range, approximately $1 \times 10^3$ m$^2$/s. Only for stresses higher than 300 kPa the two samples presented the same $c_v$ values. This experience showed that the remoulding brought the sample to a normally consolidated state, with destructuration and leading to errors on prediction of settlement variation with time.

Experience at Sarapuí clay deposit shows that good quality samples can be achieved using stationary piston sampling with 5” diameter sampler. Extra care with transportation, storage - extra care with high temperatures at field - and extrusion of sample must be taken. Another recommendation is to wait at least 24h between sampler insertion and withdraw, to obtain better sample recuperation.

The $(e_{vo} - e_s) / e_s$ relationship, where $e_{vo}$ is voids ratio at in situ vertical stress, is an indication of sample quality. Lunne et al. (1997) proposed that for high quality samples this relationship is lower than 0.04, for an OCR between 1 and 2. Analysis of 63 oedometer tests carried out by Ortigão (1980) shows an average $(e_{vo} - e_s) / e_s$ relationship of 0.033 and for only 16% of samples this relationship is higher than 0.04.
Figure 5. Sampling effect on uni-dimensional compression of Sarapuí clay.

4.2 Special oedometer tests

The rate of load application and duration of tests are the main limitation of incremental (conventional) oedometer tests. In order to study consolidation of Sarapuí clay by means of short duration tests, restricted flow consolidation (RFC) tests (Almeida et al. 1993b) and CRS tests (Carvalho et al. 1993) were performed and compared with incremental tests.

In the restricted flow consolidation the total stress was measured at cell base, one face was drained and at the other face pore pressure is measured. When compared to incremental consolidation tests carried out for 10 days, 24 hour RFC tests presented similar compression curves and also similar preconsolidation pressure, as show in Figures 6 and 7.

CRS oedometer tests were also carried out at different strain rates (5.3, 2 and 1 x 10^{-6} s^{-1}). Carvalho et al. (1993) proposed a method to evaluate the strain-rate to be used in CRS tests, based on a single stage of an incremental test. The method, also reported by Almeida et al. (1995), was applied to Sarapuí clay and good agreement was obtained for the pore pressure ratio \( u_b / s_v \) estimated from oedometer tests with values measured in CRS tests. Good agreement of compression curves of the two test types was also obtained, as shown in Figure 6 (Almeida et al. 1995). Data of coefficients of consolidation were also in good agreement.

Even though the good accordance between conventional IL oedometer tests and CRS and RFC, it should be clear that this behaviour is strain-rate dependent and varies from clay to clay.
Figure 6. Compression curves of CRS and IL tests and RFC and IL tests.

Figure 7. Preconsolidation pressure versus depth - RFC, CRS and IL tests.

5 HYDRAULIC CONDUCTIVITY AND COEFFICIENT OF CONSOLIDATION

5.1 Laboratory tests

Conventional and special oedometer tests were carried out with radial drainage: inflow and outflow types, and inflow-outflow tests also described as double drainage, and with vertical drainage on samples at 90° with horizontal plane, in order to obtain coefficient of horizontal consolidation (c_h) value (Coutinho 1976, Lacerda et al 1977, 1995). For tests carried out with radial drainage with inflow and outflow, c_h values were computed using Baron (1948) and Scott (1961) solution, respectively, both with “equal strain” consideration. Figure 8 shows average vertical and horizontal coefficients of consolidation with stress, for those tests, carried out at the middle of the clay layer.

For higher stresses, showed in detail in Figure 8, the c_h range of inflow tests are systematically higher than outflow tests, maybe due to leaks or formation of smear zone around the external boundary of the sample, as vertical deformation develops. The outcome is that outflow tests and double drainage tests are quicker than inflow tests and are not affected by possible flow between top plate and the ring (Lacerda et al. 1995). The c_h/c_v relationship lies between 1.0 and 2.0 at normally consolidated range, which is typical for very soft clays.

At the crust, average c_v values at overconsolidated domain are 6 x 10^{-6} m^2/s (z = 0.45m); 2 x 10^{-6} m^2/s (z = 0.75m) and 1 x 10^{-7} m^2/s (z = 1.5m), higher than those of the middle of the deposit presented at Figure 8 (Gerscovich et al. 1986).
Hydraulic conductivity at overconsolidated domain, $k_v$, at the middle of the clay deposit lies in the range $2 \times 10^{-9}$ to $5 \times 10^{-9}$ m/s (Coutinho 1976), which is much smaller than measured (Gerscovich et al. 1986) at the crust. On the overconsolidated domain, $k_v$ values measured at constant head tests carried out on triaxial cells lie in the range of $3 \times 10^{-9}$ m/s (at 1.5m) to $24 \times 10^{-8}$ m/s.

Figure 9 shows permeability variation with void index. At normally consolidated domain anisotropy permeability factor was $k_h/k_v = 1.6$ to 2.1 for void index between 3 and 2. Sandroni et al. (1997) found $k_{h0}/k_{v0}$ close to 2 for a Northwestern Brazilian organic clay. However, for inorganic eastern Canadian clays, for example, this factor is lower, of about 1.1 (Leroueil et al. 1983).

Figure 8. $C_h$ and $C_v$ values from laboratory tests at 5.5 - 6.0 and 6.5 - 7.0m depth (after Coutinho 1976).

Figure 9. Vertical and horizontal permeability variation with void index (Lacerda et al. 1977).
5.2 \textit{In situ tests}

The $c_h$ values deduced from piezocone dissipation tests performed at Sarapuí clay deposit are presented in Figure 10, with depth. Data from Danziger (1990) and Bezerra (1996) was analysed using Houlsby and Teh (1988)’s method, while Sills et al. (1988) used Baligh & Lebadoux (1980)’s method.

The figure also shows $c_h$ laboratory range of values obtained by Coutinho (1976) at overconsolidated domain. Laboratory range is in better accordance with data analysed with Houlsby and Teh’s method which, unlike Baligh’s method, take in consideration the soil rigidity index.

\textit{In situ} constant head permeability tests were carried out on piezometers, with $R = \Delta u/\sigma’_vo$ between 0.3 and 0.7, where $\Delta u$ is pore pressure excess (Werneck et al. 1977). Hydraulic conductivity $k_v$ was found to be between $1.4$ and $4.0 \times 10^{-9}$ m/s as shown in Figure 11. Values of $k_v$ obtained (Coutinho 1976, Duarte 1977) from laboratory tests at depths 1.5 - 8m at the \textit{in situ} stress state, are also plotted in Figure 11 and the overall agreement between laboratory and \textit{in situ} tests is good.

Figure 10. $C_h$ values with depth at overconsolidated range.

Figure 11. Hydraulic conductivity values with depth at overconsolidated domain.
Differences between laboratory and *in situ* test values of coefficient of consolidation are due to differences in the method used in the analysis, scale, stress state, anisotropy and heterogeneity of samples, and for Sarapuí clay tests it can be as high as one logarithmic cycle.

6 SOIL STRENGTH AND LIMIT STATE

6.1 Undrained strength

The undrained shear strength ($S_u$) of Sarapuí clay was extensively studied by means of both laboratory and *in situ* tests. $S_u$ data from 136 UU tests performed on 50mm and 63mm diameter samples (Ortigão 1975) were scattered and the values were lower than those of 138 UU tests performed on 127mm diameter samples (Costa Filho et al. 1977, Ortigão 1980), probably due to some degree of remoulding.

Figure 12 shows the average undrained strength ($S_u$) profile of Sarapuí clay obtained from UU, CK$_{0}$U and CIU (Shansep method). For tests performed using Shansep method, the samples were brought to consolidation until normally consolidated domain was reached, and then unloaded to an OCR value, and then undrained compression was carried out until rupture. It was observed that $S_u$ values obtained using Shansep method are underestimated, since this method may induce the destruction of the natural clay structure. $S_u$ from triaxial tests are lower, when compared to vane tests values.

The critical state strength profile (Fig. 12) was obtained using critical state parameters and OCR and $K_o$ values varying with depth (Almeida 1982). $S_u$ values predicted from isotropic consolidation are higher than those from anisotropic consolidation, the same behaviour observed when using Shansep method.

The undrained strength obtained from UU tests performed on samples of the crust lies between 4 and 7 kPa (Bressani 1983, Gerscovich 1983). As, thus lower than those measured by vane tests shown in Figure 13. It is possible that due to high permeability of the crust, strength measured at vane test at this level was partially drained.

Natural clays are anisotropic: strength parameters can change with direction, limit state curves (LSC) shape are different from cam-clay model LSC, which is based on isotropic clay. This behaviour can be amplified depending upon the type of test or method used in analysis. In order to study anisotropy vane tests using different sizes of vane blade ($H/D = 2, 0.5$ and $4$) were performed on Sarapuí clay (Collet 1978, Costa Filho et al. 1977). From these simplified studies, which limitations are nowadays recognized (Wroth 1984), it seemed that strength anisotropy of Sarapuí clay is not important.

![Figure 12. Average undrained strength profiles of Sarapuí clay.](image-url)
Collet (1978) carried out series of vane tests at Sarapuí site, and obtained the $S_u$ average profile shown in Figure 13. Ortigão & Collet (1986) improved the vane test equipment to decrease friction and obtained another $S_u$ average profile, with $S_u$ values higher than before (Fig. 13). Using the new vane data, the normalized $S_u/\sigma'_{pconv}$ average relationship is 0.35 (Almeida 1986), in accordance with the relationship proposed by Leroueil et al. (1983), $S_u/\sigma'_{pconv} = 0.2 + 0.0024 IP$, for eastern Canadian clays.

Values of $S_u$ deduced from dilatometer tests (Vieira 1994) are lower than $S_u$ values of vane tests. Differences from in situ tests are probably due to different stress paths and empirical factors of piezocone and dilatometer.

Normalized $S_u$ increases with OCR, in a different way for CIU and CK$_0$U tests (Fig. 14). For OCR = 1, the curves present the same $S_u/\sigma'_{vc}$, but as OCR increases, for CK$_0$U tests $S_u/\sigma'_{vc}$ relationship is lower. The variation of the normalized strength with OCR has been well predicted using critical state parameters ($\lambda$, $\kappa$, $M$) for both isotropic and anisotropic consolidation conditions (Almeida 1982). As expected, and seen in Figures 12 and 14, for triaxial tests carried out using Shanesp method, CIU tests presented systematically higher $S_u$ normalized value than CK$_0$U tests.

![Figure 13. Undrained shear strength profile.](image)

![Figure 14. $S_u/\sigma'_{vc}$ variation with OCR from CIU and CK$_0$U tests (Ortigão 1980).](image)
6.2 Effective strength parameters

Figure 15 shows the stress path of CIU tests carried out at different consolidation pressure ($\sigma'_c$) (Ortigão 1980); stress state at rupture from CK, U tests (Ortigão 1980) and CIU tests (Costa Filho et al. 1977), all performed on samples below crust. The average strength envelope was obtained from $t_{\text{max}} = ((\sigma'_1 - \sigma'_3)/2)_{\text{max}}$ values. The friction angle of Sarapuí clay lies between 25 and 30°, as shown in Figure 15.

For CIU and UU tests performed on samples at 4 - 5m depth, at low stress level, the strength envelope presents $c' = 1.5$ kPa and $\phi' = 25^\circ$, as shown in detail in Figure 15 (Costa Filho et al. 1985, Gerscovich et al. 1986).

The strength envelope obtained after CIU and UU tests (Gerscovich 1983, Bressani 1983), performed on samples of the crust is shown in Figure 16. The friction angle of the dessicated crust is higher than that of the soft clay below. At low stress level, the strength envelope is curve, but can be approximately described by $c' = 1.5$ kPa, and $\phi' = 30^\circ$. For higher stress level the strength parameters become $c' = 0$ and $\phi' = 31^\circ$.

Figure 15. Strength envelopes of Sarapuí soft clay.

Figure 16. Strength envelopes of Sarapuí crust.
6.3 Piezocone (CPTu)

Piezocone tests were performed on Sarapuí clay with different types of equipments (Soares et al. 1987, Danziger et al. 1997). Figure 17 shows the $q_t$ (corrected point resistance), the pore pressures measurements during driving (cone face, $u_1$ and shoulder, $u_2$) and frictional resistance of a piezocone test carried out at Sarapuí deposit (Bezerra 1996). The last generation of piezocone equipment developed by COPPE/UFRJ can measure point resistance, side friction, and pore-pressure at both tip and shoulder of the cone, besides monitoring of the cone inclination.

Figure 18 shows average piezocone tests factors, $N_{kt}$, $N_{ke}$ and $N_{du}$, based on vane strength data, for some Brazilian soft clays, in comparison with Sarapuí clay (Sandroni et al. 1997). The cone factors shown in Figure 18 illustrate clearly the importance of using regional piezocone factors in analysis.

Figure 17. Piezocone results of Sarapuí deposit (Bezerra 1996)

Figure 18. Empirical cone factors for Brazilian soft clays (Sandroni et al. 1997).
\( N_{k_t} \) values obtained by Bezerra (1996) lie between 11 and 16, obtained from \( S_u \) results of vane borer (Ortigao & Collet 1986). Undrained strength was estimated after piezocone tests (Bezerra 1996) using \( N_{k_t}=15 \) (Fig. 18).

The friction angle with depth deduced from piezocone tests by Rocha Filho (1988) were in good accordance with values obtained by Costa Filho et al. (1977) from CIU triaxial tests. Below crust, the friction angle seems to increase slightly with depth, and at crust, as discussed before, triaxial tests showed that friction angle is higher (Costa Filho et al. 1985).

Danziger & Schnaid (2000), in a state-of-the-art report on the Brazilian experience on piezocone tests, showed the need of improving soil classification method (Robertsson et al. 1986) in order to encompass Brazilian organic clays.

### 6.4 Limit State Curve

Natural clays, contrary to laboratory prepared clays are anisotropic, have different horizontal and vertical properties, due to their formation process, thus limit state curves (LSC) of natural clays are not centred on isotropic axis (Diaz-Rodriguez et al. 1992). Figure 19 shows normalized LSC of five natural soft clays localized at Rio de Janeiro State (Futai 1999), Sarapuí clay included.

![Figure 19. Limit state curves of five Rio de Janeiro soft clays (Futai 1999).](image_url)

Both \( s' \) and \( t \) were normalized with relation to preconsolidation pressure, however \( t \) was also normalized with relation to the critical state parameter \( M \), defined as follow.

\[
M = \frac{6 \sin \phi'}{3 - \sin \phi'}
\]

using different values of \( \phi' \) (friction angle), as function of the tests depth.

The limit state curve of Sarapuí clay, as well as the others, is not centred on isotropic axis, due to its anisotropic properties. The general shape of the LSC of soft clays at in situ natural state is shown in curve 1 (Fig. 20) and it depends mainly on the clay friction angle (Diaz-Rodriguez et al. 1992). As isotropic consolidation is performed, the LSC obtained after those tests goes from state 1 to state 4, when the natural structure of the soil is already broken, and the LSC is centred on isotropic axis, as would a laboratory prepared clay, as stated on Cam-clay model.

When using Shansep method the soil is loaded well beyond consolidation pressure, then unloaded until an OCR is achieved, and then undrained sheared. In this case, the soil loses its
structure, which can explain the divergences when using Shansep method to obtain soil parameters and LSC, for natural clays (Futai et al. 2001).

![Critical state line](image)

Figure 20. Limit state curves variation with isotropic loading (Futai et al. 2001).

6.5 Deformation data

Rupture of Sarapuí clay is plastic, and strain at rupture varies from 7 to 9%. Young Modulus, $E_u$, obtained at 50% of stress, after 127mm diameter UU (Ortigão 1980) was found to increase with depth (Fig. 21a), however, average $E_u/S_u$ relationship seems to be constant, $E_u/S_u = 265$ (Fig. 21b).

For CIU and CK0U tests (Ortigão 1980) using Shansep method, average $E_u/S_u \equiv 100$, is lower since the soil looses its structure due to the method (Fig. 22). The variation of deformation moduli with strain of Sarapuí clay has not yet been measured in laboratory with internal instrumentation.

Francisco (1997) performed seismic cone penetration tests at Sarapuí site using geophones at cone tip in order to obtain maximum shear stiffness ($G_{\text{max}}$) variation with depth. These results were compared with results from UU triaxial tests, $G = E_u/3$ (Ortigão 1980) and are showed in Figure 23. A $G/G_{\text{max}}$ profile was obtained, and thus $G$ values can be inferred from $G_{\text{max}}$ values.

![E\_u and E\_u/S\_u profiles from UU tests](image)

Figure 21. $E_u$ and $E_u/S_u$ profiles from UU tests (Ortigão 1980).
7 VISCOUS BEHAVIOUR

Studies on the rheological behaviour of Sarapuí clay have been carried since the beginning of the early days and the majority of studies were based on 1-D compression of the clay (Martins et al. 1997).

Ortigão (1980) studied uni-dimensional secondary compression at normally consolidated domain. Three conventional oedometer tests were carried out until 67 kPa - the equivalent stress level of a 2.7m fill - then the samples were allowed to consolidate during 70 days, under constant effective stress. After this period of time, the sample was again loaded as a conventional oedometer test, as shown in Figure 24a.

The analysis of deformation curve with time showed that the coefficient of secondary compression changed with time (Fig. 24b). Martins & Lacerda (1985) stated a theory to explain this behaviour and proposed that the average degree of consolidation $\bar{U}$ as follows:
\[ \bar{U}(T) = \frac{\Delta\sigma}{\sigma} \left[ 1 - \frac{2}{M_0} \sum_{m=1}^{\infty} \frac{2}{3} M_0^2 e^{-TM_0^2} \right] + \frac{2}{3} (1 - K_{0n}) \left[ 1 - e^{-\theta T} \right] \]

\[ M = (2m+1)(\pi/2) \]

\[ \theta = \lambda t/T \]

where, \( T \) is the time factor as defined in the classical theory and \( K_{0n} \) is the coefficient of earth pressure at normally consolidated domain.

In order to verify this behaviour Vieira (1988) performed four long-term oedometer tests, carried out in samples at 1.25m depth. The samples were loaded until 75, 100, 200 and 400 kPa, and monitored during almost a year. Figure 25 shows two tests, which the secondary compression was monitored when the sample was loaded from 50 to 200 kPa (test A) and from 50 to 400 kPa (test B).

Figure 24. Secondary compression of Sarapuí clay (Ortigão 1980).
The theoretical behaviour proposed by Martins & Lacerda (1985) is also presented at Figure 25, and it seems that for lower values of $\theta$, slower is the secondary compression. As for loading, the lower $\Delta\sigma/\sigma$ is, higher is the effect of secondary compression, when compared with classical Terzaghi $U \times T$ curve.

Undrained shear strength changes from 5 to 20% per logarithm cycle of strain rate, and typically changes by 10% per logarithm cycle of strain rate (Leroueil & Marques 1996). Evidence of strain rate effect on Sarapuí clay normalized $S_u$ was presented by Ortigão (1980), when an average change of 15% per logarithm cycle of strain rate was measured on normally consolidated samples, after CIU tests as shown in Fig. 26.

Feijó (1991) carried out long-term oedometer tests, under controlled temperature, in order to study the secondary expansion of samples submitted to different values of OCR. Figure 27 shows volumetric strains variation with time. First, the samples were loaded in the overconsolidated range and then unloaded to different values of OCR. For OCR between 1.5 and 2 after some expansion, the sample begins to compress again. For OCR of 8 and 12 there is secondary expansion during the test.

For OCR between 2 and 6, secondary swelling is not significant and the sample is supposed to be at an equilibrated state, where strain rate is almost zero (Fig. 28a). Under this OCR range,
the coefficient of earth pressure lies between 0.77 and 1.23, which is the range of $K_0$ at equilibrium state (Fig. 28b).

This behaviour is quite important for design of pre-loading of embankments, for it is possible to decrease secondary compression of clay, when unloading from this OCR range.

Figure 26. Variation of normalized undrained strength with strain-rate (Ortigão 1980).

Figure 27. Volumetric strains observed after unloading of Sarapuí clay (Feijó 1991).
Lima (1993) also observed evidence of viscous nature of Sarapuí clay, with the results of uni-dimensional relaxation tests carried out in a special cell with drainage control. The samples were subjected to back-pressures in stages of 25 kPa to 100 kPa each. The end of primary consolidation (EOP) was determined in advance, during each loading stage by Taylor’s method. Under constant vertical stress, at normally consolidated domain, the drainage was stopped at the EOP. At a loading stage from 75 to 150 kPa, the pore pressure increased up to 30 kPa, after drainage was closed (Fig. 29a). In another test, loading stage from 100 kPa to 800 kPa, the pore pressure increased until 490 kPa, nearly 70% of the previous effective stress variation due to loading (Fig 29b).

This mechanism was well discussed by Lacerda & Martins (1985). When neither volumetric strain nor distortions were allowed on those tests, the strain rate dropped to zero. Since viscous resistance of soils is strain rate dependent, under constant total stress, the pore pressure increased in a relaxation process. In this case, the effective stress path goes from $K_0$ line to the $K_{ol}$ line (shown in Fig. 28b).

Figure 28. Zone of indifferent equilibrium (Feijó & Martins 1993).

Figure 29. Increase of pore pressure after closing the drainage at EOP (Lima 1993).
Although the in situ temperature of about 23°C is similar to those usually used in laboratory tests, Sarapuí clay is very sensitive to temperature variation. In another series of tests, Feijó (1991) observed an increase in swelling strain rate at the end of another long-term test, after stabilization was achieved. This was in fact noticed due to a problem in the temperature-controlled room, in the middle of summer, when ambient temperature can reach 40°C. Therefore, extra care with temperature control must be taken when tests are running or during sampling.

Guimarães (2000) performed stepped-creep tests on samples collected at Sarapuí deposit, not far from the experimental site. The tests were carried out inside a temperature-controlled bath, at 30°C and it was observed that a variation of 0.5°C could mask results for this kind of test.

Undrained shear strength and preconsolidation pressure changes typically by 10% with a change of temperature of 12°C (Leroueil & Marques 1996). Soil temperature at Sarapuí site is affected by ambient temperature down from 3m to 4m depths, but remains constant below this depth (Sandroni 1977).

8 FIELD STUDIES

Two experimental embankments were built in the 70' as shown in Figure 30. The Embankment I was built until rupture, 30 days after beginning of loading in December/1977. Embankment II was built in November/1980 and heightened five years later and monitored during almost 10 years since first loading. The majority of the studies described above were associated with the design of these trials, and an extensive comparative analysis between laboratory and in situ test results and field results were performed (Ortigão et al. 1983, Almeida et al. 1989).

8.1 Embankment I

Three sections of the embankment were instrumented with hydraulic piezometers, inclinometers, horizontal extensometers, settlement plates and surfaces marks (Ortigão et al. 1983).

Beginning of fissuring occurred when embankment was at the height of 2.5m. At this time the strain rate increased rapidly and at 2.8m height fissures were 5cm wide and at 3.1m the rupture was generalized. Subsequent analysis (Almeida 1985, Sandroni 1993) of this case history indicated that the actual failure took place when the embankment was the height of 2.5m. It is well known that for design purposes a Bjerrum type correction has to be applied to the $S_u$ value measured in vane tests. The re-analysis of Embankment I failure using a 3D failure surface has shown (Sandroni 1993, Souza Pinto 1992) that the correction factor for Sarapuí clay is $\mu = 0.70$, which lies slightly above (for a typical $I_P = 80$) the $\mu$ versus IP relationship proposed by Azzouz et al. (1983), for this type of failure.

![Figure 30. Experimental site layout.](image-url)
Figure 31 shows the settlement and Embankment I height with time, from settlement plates at the centerline of the embankment. At the beginning of fissures (h = 2.5m) the embankment was not maintained at this height for more than one day, therefore there was no time for deformations due to this loading to develop. The failure could have occurred at this height, if it was maintained for a longer period, before increasing to 2.8m. The rupture developed slowly, in a progressive fashion, so it may have mobilized larger strains, at smaller $S_u$ values, going to critical state.

![Figure 31. Settlement curve - Embankment I.](image)

During construction time (30 days) the clay deposit was partially drained, particularly the crust (Ortigão et al. 1983, Gerscovich et al. 1986). This consideration implies in significant increase of the crust’s $S_u$, when back-analysing the clay failure.

The hypothesis of partial drainage is consistent with piezometric measures at the middle of the embankment and at the top of the deposit. There was pore-pressure dissipation during early stages of construction.

Embankment I was analysed numerically (Almeida 1981, Almeida & Ortigão 1982, Fontenelle 1987) using the modified Cam-clay model. As shown in Figure 19, the yield surface of Sarapuí clay is not far from isotropic. Moreover, this model has provided good insight for other case histories in Brazil (Brugger et al. 1997, Antunes Filho 1996) and was therefore also used here. Analyses were performed for both, undrained and partially drained conditions (using Biot 2D consolidation theory) using the same critical state parameters, but with vertical and horizontal hydraulic conductivities for the partially drained case. Good overall agreement between measured and numerical values of base settlements and pore pressures was observed just for the partially drained case (coupled consolidation - run c), but not for the undrained condition, as shown in Figure 32, which suggests that even for one month long embankment loading some drainage may have taken place.

One other field trial to be mentioned is the study on the performance of reinforcement under an access road to Embankment I (Palmeira, 1983; Ortigão and Palmeira, XXXX) , which presented conclusions on a number of schemes and types of geosynthetics used.

### 8.2 Embankment II

Embankment II was built 35m wide and 315m long, divided in seven-instrumented sub-areas, where different kinds of drains were installed, as described in Table 5.

Loading was applied in two main stages and the final height was about $h = 3.6$ m. The first stage of loading was applied during 1981, in two steps. The first step ($h = 0.7$m) during a month
and the second, 200 days after, lasted approximately 2.5 months (h = 2.0m). The second stage, in 1986, was applied only from section B to G, in one week, until final height.

Pore pressure dissipation showed that top layer had a higher coefficient of consolidation, as expected from tests on this layer (Gerscovich et al. 1986).

Piezometric measurements (Ferreira 1991) suggested that sand drains presented better performance than pre-fabricated drains. Drains installed with jetted sand presented better performance because of fissures created by the installation process, decreasing even more drainage distance. Between the two types of prefabricated drains installed, the geotextile-prefabricated drains had better performance than the fibro-chemical.

Table 5. Characteristics of sub-areas - Embankment II (Terra 1988, Almeida et al. 1989).

<table>
<thead>
<tr>
<th>Section</th>
<th>Clay thickness (m)</th>
<th>Vertical drain type</th>
<th>Installation type</th>
<th>Diameter (mm)</th>
<th>Drain distance (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>10.5</td>
<td>none</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>B</td>
<td>10.5</td>
<td>close ended mandrel/sand</td>
<td>displacement</td>
<td>400</td>
<td>2.5</td>
</tr>
<tr>
<td>C</td>
<td>10.5</td>
<td>open ended mandrel/sand</td>
<td>lowdisplacement</td>
<td>40</td>
<td>2.5</td>
</tr>
<tr>
<td>D</td>
<td>10.5</td>
<td>jetted/sand</td>
<td>non displacement</td>
<td>40</td>
<td>2.5</td>
</tr>
<tr>
<td>E</td>
<td>10.2</td>
<td>prefabricated (polyetilen)</td>
<td>displacement or lowdisplacement</td>
<td>2.8 x 100</td>
<td>1.7</td>
</tr>
<tr>
<td>F</td>
<td>10.0</td>
<td>prefabricated (polyester)</td>
<td>displacement or lowdisplacement</td>
<td>4.5 x 210</td>
<td>2.0</td>
</tr>
<tr>
<td>G</td>
<td>9.0</td>
<td>none</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

For the first stage of loading, when range of vertical effective stress was going from 15 to 60 kPa, the c_v from field data was difficult to analyse and compare. At this stress range there is an
important variation of $c_v$ when soil goes from overconsolidated state to normally consolidated state, as shown in oedometer results presented on Figure 8.

For the second stage of loading, from 60 to 80 kPa, at normally consolidation range, the $c_v$ from oedometer tests remained almost constant with vertical stress (Fig. 8).

Table 6 shows average coefficient of vertical and horizontal consolidation obtained from laboratory, *in situ* tests and back-calculated from settlement data using Asaoka (1978) method, for the second stage of loading. From those results it was observed that $c_v$(field) / $c_v$(laboratory) relationship range lies between 20-30.

<table>
<thead>
<tr>
<th>Test or data</th>
<th>Depth (m)</th>
<th>Method</th>
<th>Reference</th>
<th>$c_v$ $10^{-8}$ m$^2$/s</th>
<th>$c_h$ $10^{-8}$ m$^2$/s</th>
</tr>
</thead>
<tbody>
<tr>
<td>Oedometer test</td>
<td>5-6</td>
<td>Taylor</td>
<td>Coutinho (1976)</td>
<td>1.2</td>
<td>2.4</td>
</tr>
<tr>
<td></td>
<td>2.2-8.2</td>
<td>Houby &amp; Teh (1988)</td>
<td>Danziger (1990)</td>
<td>1.6-4.4</td>
<td>3.1-8.7</td>
</tr>
<tr>
<td>Piezocone</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Field instrumentation:</td>
<td>whole layer</td>
<td>Asaoka (1978)</td>
<td>Schmidt (1992)</td>
<td>17.8</td>
<td>3.1-4.4***</td>
</tr>
<tr>
<td>settlement plates</td>
<td>whole layer</td>
<td>Asaoka (1978)</td>
<td>Almeida et al. (1989)</td>
<td>22.6</td>
<td>4.2-8.1****</td>
</tr>
<tr>
<td>Field instrumentation:</td>
<td>3.3-8.3</td>
<td>Orleach (1983)</td>
<td>Ferreira (1991)</td>
<td>2.2-4.5</td>
<td>1.2-2.8</td>
</tr>
<tr>
<td>top settlement</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>magnetic gages</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Field instrumentation:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Casagrande piezometers</td>
<td></td>
<td></td>
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</tr>
</tbody>
</table>

*average value of central settlement plates.
**average value of central settlement plates (no smear, vertical drainage considered).
***smear but no vertical drainage considered.

Schmidt (1992) showed that low value of $\sigma'_p/\sigma'_w$ can lead to error on $c_v$ determination with Asaoka’s method, since secondary consolidation is very important. As discussed in item 7, the lower $\Delta \sigma/\sigma$ is, higher is the effect of secondary compression on settlement curve. The good agreement between $c_v$ values obtained from laboratory and *in situ* tests, and from settlement plates analysis discussed by Almeida et al. (1993a) for another site not far from Sarapuí was due to the high $\Delta \sigma'$, (24m height fill), since almost no secondary compression was observed.

Pinto (2001) discussed the validity of Asaoka’s method and observed that $c_v$ values, as well as final settlement value, computed using the method are very susceptible to time of monitoring. From series of data analysed with different periods of observation, the serie with 100 days of observation the $c_v$ computed was 0.082 m$^2$/day, while after 450 days, the $c_v$ was 0.005 m$^2$/day.

An instrumented pile was driven near at the site and dissipation tests were performed at the bottom, at 6.6m depth. The values of $c_h$ obtained from those tests were well above laboratory values (Soares & Dias 1989, Dias 1988). This behaviour was caused mainly by uncertainty due to initial pore-pressure caused by pile installation; difficulty of analysis due to consolidation of clay surrounding the pile, disturbance of clay, anisotropy and creep.

8.3 *Experimental excavation*

An experimental excavation was also carried out at Sarapuí site. Figure 33 shows the main geotechnical characteristics of the deposit at excavation site (Sayão 1980) compared to average values of embankments site (Ortigão 1980). Even though the excavation site was near the embankments, geotechnical characteristics of the deposit were quite different. Due to an existing 1.4m fill at the excavation site, the clay deposit was normally consolidated and values of $\sigma'_p$ are similar to those obtained close to the embankments. The natural unit weight is higher and the natural water content is lower, thus $\Sigma u$ values from UU tests and vane tests are also higher.
9 CONCLUSIONS

Sarapuí clay is a 6000 years old fluvial/marine deposit, typically about 10m thick. It is highly plastic clay with water content higher than the liquid limit, but with low sensitivity. The main clay mineral is kaolinite, but illite and smectite are also present in a smaller proportion. It is an organic clay and a typical value of the organic matter content is 5%. The clay layer is quite homogeneous, but it has a 3m thick crust resulting from water level fluctuation and desiccation. The geotechnical parameters of the crust differ from the rest of clay layer. Stress history and compressible behaviour were defined by means of good quality laboratory specimens obtained from stationary piston samples. Below the crust the clay is overconsolidated due to aging and water table fluctuation with values of overconsolidation ratio OCR varying from 2.0 to 1.3. The compression ratio of Sarapuí clay $C_c/(1+e_0)$ is equal to 0.41, thus the clay is not just highly compressible but also presents an important viscous behaviour.

Values of hydraulic conductivity obtained from laboratory and in situ tests were in the same range and $3.0 \times 10^{-9}$ m/s can be considered a representative value for mid depth of the deposit. Coefficient of consolidation in vertical and horizontal directions have been obtained by laboratory and in situ tests and also assessed by means of a trial embankment. The ratio $c_h/c_v$ obtained with oedometer tests was about 2 with $c_h = 2.4 \times 10^{-8}$ m$^2$/s. Piezocone values of $c_v$ and $c_h$ were at least 50% higher than laboratory values. Field values of $c_h$ from the trial embankment were in the same magnitude of piezocone values. However, field values of $c_v$ from the trial embankment were badly influenced by the secondary consolidation observed.

The clay undrained strength increases with depth and is in the range 8-20 kPa. Best estimates of $S_u$ were made with large diameter UU specimens and with the vane borer equipment. Critical state strength agreed quite well with UU data. A trial embankment taken to failure confirmed the need to apply a correction the in situ vane data. Measured effective strength parameters of Sarapuí clay were $c' = 0$ and $\phi' = 25^\circ$. However $c'$ and $\phi'$ at the crust measured at low stress levels for were greater than below the crust. Hydraulic conductivity and consolidation parameters were also observed to increase at the crust, apparently to its higher fibre content. As most natural clays, the limit state curve of Sarapuí clay is not centred on isotropic axis. As expected, Young’s modulus measured at 50% of the maximum deviator stress increase with depth, but after normalizing it with undrained strength the ratio $E_u/S_u$ gets about constant with depth. The resulting rigidity index $I_r = G/S_u$ is equal to about 90, which is a typical figure for soft clays.

Sarapuí clay viscous behaviour was observed from laboratory experimental data. Sarapuí clay showed high sensibility to temperature and strain-rate variation and important deformation due
viscous behaviour was also observed from settlements analysis, which affected field $c_v$ calculations.

The piezocone was shown to be a useful tool for providing a continuous estimated profile of a number of geotechnical parameters, such as undrained strength, OCR, $c_h$ and $K_0$. Other in situ tests, such as dilatometer and hydraulic fracture tests were also carried out and provided insight on the in situ clay behaviour.

The trial embankments were useful in providing the actual clay behaviour in the field. The embankment taken to failure showed that some drainage took place during the one-month embankment loading and confirmed the need for correction of the vane strength. The embankment under consolidation with and without vertical drains was useful to evaluate the field coefficients of consolidation. It was also useful to illustrate the important secondary consolidation in situ, previously advanced by means of laboratory tests.

10 ACKNOWLEDGEMENTS

The geotechnical properties of the Sarapuí clay, summarized herewith, result from a large number of studies, particularly master and PhD dissertations. The authors are very much indebted to all involved in these studies, which made the preparation of this paper possible. The authors would like to acknowledge Leidimar Bezerra for lending piezocone data, to Ayla Romano for valuable help on data analysis.

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