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Soil Behaviour and Shear Strength Parameters of an Organic Alluvium Soil Using the CPTu and DMT

Guilherme H. S. Pinto[1#], *Bruna* Z. Hoch[2] and *Mauro* P. S. Junior[3]

[1,2] *Pimenta de Avila Consulting, Geotechnical Engineer, Alameda Oscar Niemeyer, 420 - Vale do Sereno, Nova Lima, Brazil*

[3] *Pimenta de Avila Consulting, Geotechnical Coordinator, Alameda Oscar Niemeyer, 420 - Vale do Sereno, Nova Lima, Brazil*

[#]*Corresponding author: guilherme.henrique@pimentadeavila.com.br*

ABSTRACT

The evaluation of in-situ behaviour, strength and compressibility of a soil profile is routinely performed by geotechnical engineers through field tests, such as the seismic piezocone penetration test (SCPTu), the flat dilatometer test (DMT) and the field vane shear test (FVT). This paper aims to compare the results of a CPTu, DMT and FVT to evaluate an organic alluvium soil in terms of: i) in-situ soil behaviour classification, ii) undrained shear strength and iii) stress history. To compare and complement the in-situ results, laboratory tests were carried out to determine the grain-size distribution, the Atterberg Limits, the pre-consolidation pressure, the organic content and the undrained shear strength under isotropic consolidation triaxial test (CIUC). The results showed that the soil evaluated herein exhibited a clay-like behaviour based on the classification system of both tests (DMT and SCPTu), which agrees with the laboratory characterization. Furthermore, the OCR (overconsolidation ratio) calculated from the SCPTu and DMT also shows a convergence with the values determined from laboratory tests. The SCPTu performed in this soil was predominantly undrained and enabled the calculation of undrained shear strength. Based on this, the methodologies based on N_{kt} and $N_{\Delta u}$ (from SCPTu) were compared with the undrained shear strength from the FVT and that obtained from DMT, based on the K_D parameter. Finally, a comparison is presented to discuss the influence of shear mode in the undrained shear strength and the applicability of the methodologies used to evaluate the soil behaviour and the stress history.

Keywords: CPTu; DMT; Vane Shear Test; Undrained shear strength.

1. Introduction

In geotechnical design, it has long been recognized that the assessment of soil properties and behaviour is the most important single task (Janbu 1985). To perform this evaluation, several approaches have been used in geotechnical engineering based on laboratory and field assessments.

The laboratory tests provide data under controlled conditions of the behaviour of geomaterials as well as complete characterization, including particle size distribution, plasticity, compressibility, and shear strength. However, the determination of such parameters depends on the technical team which performed the test and can be influenced by how representative the sample is, in addition to other factors such as storage, handling and transportation.

On the other hand, field tests allow the behaviour evaluation of these materials in the in-situ condition, considering properties such as stiffness and compressibility, which makes possible the evaluation of drainage conditions, porepressure profile and the estimation of the in-situ state of stress. Furthermore, through different correlations established in the literature, it is also possible to estimate shear strength parameters.

Regarding the field assessment, the piezocone penetration tests (CPTu) are now a cost-effective tool and an internationally recognized and established routine adopted in site characterization, soil profiling and assessment of the constitutive properties of geomaterials (Schnaid 2009). Also, the flat dilatometer test (DMT) has also been used to evaluate soil constitutive properties - such as deformability and stress history - and to estimate shear strength parameters (drained and/or undrained).

When contractive materials are studied, the positive induced porewater pressure in the shear process and the decrease of the effective shear strength turns the undrained condition into the most critical. In this case, the undrained shear strength must be assessed and, if possible, this must be based on different approaches to ensure the reliability of the value obtained.

As described by Lunne et al. (1997), the undrained shear strength (S_u) can be defined as the shear resistance of a geomaterial in a saturated or nearly saturated condition, which is mobilized under a fast loading without allowing any volumetric change. This parameter is commonly used to evaluate foundation bearing capacity, such as shallow footings, rafts foundation, and pilings, as well as an input to short-term stability analysis of excavations, slopes, and to determine the stability condition of embankments and tailings storage facilities (Mayne and Peuchen 2018).

However, differently from the effective parameters (such as the effective friction angle), the S_u values can be influenced by the complex effects of anisotropy, stress history, shear mode, direction of loading, boundary conditions, and other factors. Regarding the shear mode effects, different studies (Totani et al. 2001, Mayne 2016, Mayne and Peuchen 2018) have demonstrated the influence of this aspect in the final value obtained, as exemplified in Fig. (1) to the Bothkennar soft clay.

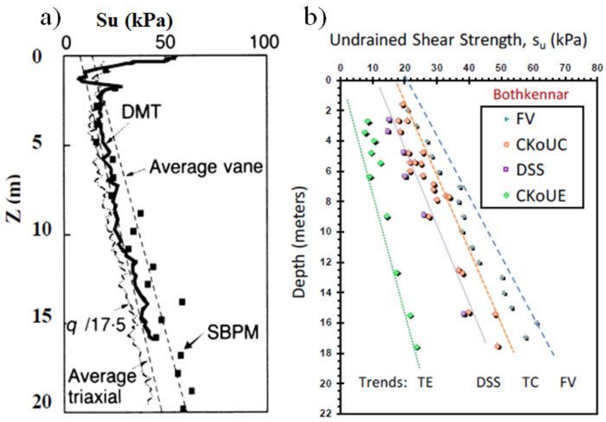

Figure 1. Undrained shear strength values depending on the shear mode of the Bothkennar soft clay: a) Adapted from Totani et al. 2001; b) Adapted from Mayne (2016).

Based on this, a family of S_u values must be considered and obtained through field and laboratory tests, instead of adopting one unique value/methodology (Brown and Giuliani, 2016). Also, this value must be compatible with the shear path under which the material can be mobilized.

This paper aims to compare the results of a Seismic Cone Penetration Test with porepressure measurement and dissipation test (SCPTu), Flat Dilatometer Test (DMT) and Field Vane Shear Test (FVST) to evaluate an organic alluvium soil in terms of i) in-situ soil behaviour classification; ii) stress history; iii) drainage conditions and iv) yield undrained shear strength. Also, the results obtained herein are compared and complemented with those obtained from laboratory tests: i) the grain-size distribution; ii) the Atterberg Limits; iii) the grain density (ρ_s) iv) the pre-consolidation pressure (σ'_p) assessed by oedometer test; v) the organic content; vi) the natural water content and vii) the undrained shear strength assessed by triaxial test under isotropic consolidation (CIUC).

2. Methodology and available data

The organic alluvium soil studied herein is located beneath the embankment of a tailings storage facility (TSF), which makes its characterization (in terms of strength and stiffness) of utmost importance when the TSF stability condition is evaluated.

A detailed program was developed to perform field and laboratory tests in the organic alluvium soil with the main purpose of determining: i) its current state/shear response (contractive/dilative) and ii) the shear strength parameters. All the field tests were performed in consecutive days and near to each other, allowing a straight comparison.

3. Soil characterization by laboratory tests

Figure 1 shows the Particle-Size Distribution (PSD) of the analysed soil according to the international standard D6913-04 (ASTM 2009). The results indicate that the soil is composed by 49.8% of clay, 28.6% of silt, 19.8% of sand and 1.8% of gravel. Also, the grain density was obtained as ρ_s = 2.585 g/cm³, according to the standard D792-20 (ASTM 2020).

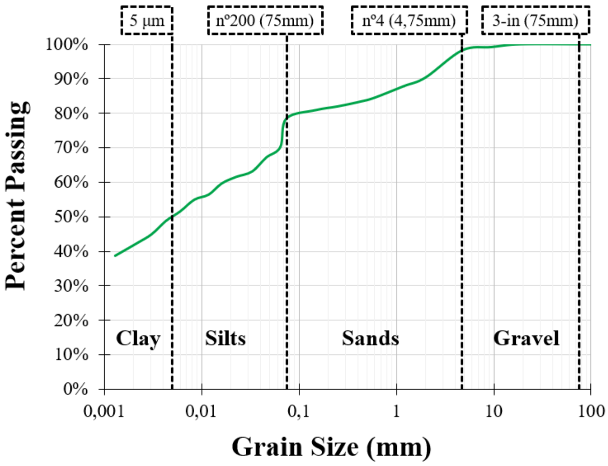

Figure 2. Organic alluvium PSD.

The Atterberg Limits were determined by the international standard D4318-17e1 (ASTM 2018) and the result is shown in Fig. (2), indicating that the organic alluvium soil can be characterized as highly plastic silt (MH) or highly plastic organic (OM).

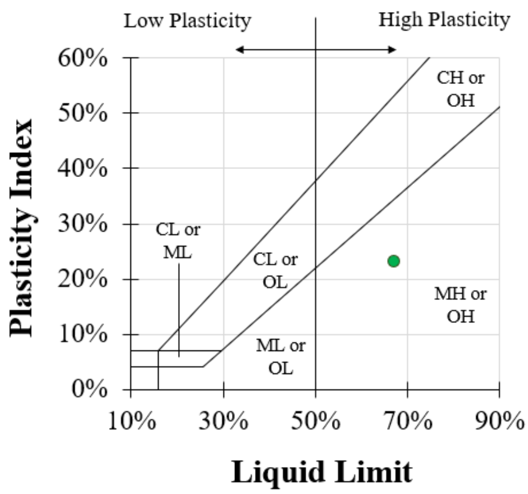

Figure 3. Plasticity chart determined for the organic alluvium soil.

The natural water content in the organic alluvium was 50.3% and was determined following the procedures outlined in the standard D2216-19 (ASTM 2019). This value is high if compared to non-organic soils. Other researchers have found similar values, as shown by Merani et al. (2016).

Also, the organic content was determined following the American standard D2974-14 (ASTM 2020) resulting in an average value of 20.5% (average of three tests), with less than 1% of variation between the measurements.

4. In-situ behaviour

The in-situ behaviour evaluation was performed using the Seismc Cone Penetration Test with porepressure dissipation and the Flat Dilatometer Test.

4.1. Seismic Cone Penetration Test (SCPTu)

To evaluate the in-situ behaviour based on the SCPTu data, one test was performed according to international standard 22476-1 (ISO 2022). The SCPTu with dissipation test was performed using a 10 cm² cone pushed at 2.0 ± 0.5 cm/s and readings were taken at every 5 cm. The test provided five independent parameters: (i) the cone tip resistance (q_c), which characterizes the soil resistance to cone penetration; (ii) the sleeve friction (f_s), which represents the soil adhesion to the friction sleeve; (iii) the porewater pressure (u), commonly measured behind the cone tip (u_2 location); (iv) the shear wave velocity (V_s) which represents the velocity that the shear wave propagates into the soil mass; and (v) the equilibrium in-situ porewater pressure (u_0), by the dissipation test.

The measured cone resistance was corrected to the total cone resistance (q_t) by the equation $q_t = q_c + u_2$ (1-a), where the "a" value was approximately 0.79 (as described by the cone certificate). This correction was performed to account the porepressure action in unequal end areas (Lunne et al. 1997).

As detailed by Robertson (1990, 2016) and Robertson & Wride (1998), since most of the soils are essentially frictional and strength and stiffness increase with depth, the normalized parameters are more consistent and reliable to evaluate the in-situ soil behaviour. Based on this, the authors use the normalized porepressure (B_q), the normalized Friction Ratio (F_R), the normalized Cone Resistance (Q_t) and the normalized cone resistance by the atmospheric pressure (Q_{tn}) in complement to cone measurements. These parameters can be calculated by equations 1 to 4 respectively.

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \tag{1}$$

$$F_R = \frac{f_s}{q_t - \sigma_{v0}}\ x\ 100\% \tag{2}$$

$$Q_t = \left(\frac{q_t - \sigma_{v0}}{\sigma'_{v0}}\right) \tag{3}$$

$$Q_{tn} = \left(\frac{q_t - \sigma_{v0}}{Pa}\right)\left(\frac{P_a}{\sigma'_{v0}}\right)^n \tag{4}$$

Where σ_{v0} and σ'_{v0} are the total and the effective vertical stress, Pa is the atmospheric pressure, and the exponent "n" is obtained by Eq. (5) as a function of the material behaviour index ($I_{CR\&W}$) obtained by Eq. (6).

$$n = 0.381 I_{CR\&W} + 0.05\frac{\sigma'_{v0}}{P_a} - 0.15 \tag{5}$$

Figure 4 shows the basic parameters from the analysed CPTu, and it is possible to observe four different materials: (i) the compacted landfill; (ii) the non-compacted landfill (no compaction control); (iii) the alluvium with organics; and (iv) the foundation clay. The focus herein is the organic alluvium, which presents a low cone tip resistance (below 2 MPa), low sleeve friction, as expected to soft materials (Lunne et al. 1997), and a high generation of porepressure (higher than 200 kPa).

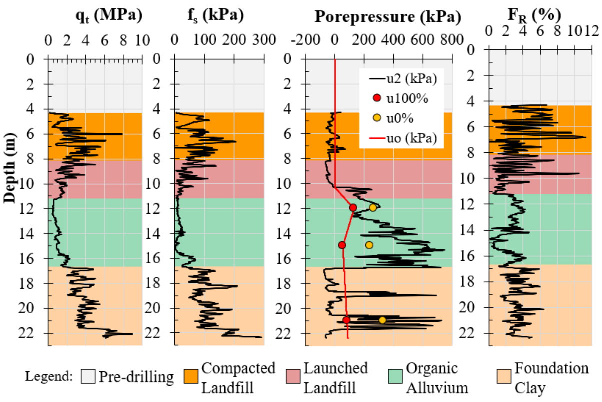

Figure 4. CPTu basic parameters: a) q_t; b) f_s; c) u_2; and d) F_R.

To perform the material behaviour classification, Robertson & Wride (1998) presented the $I_{CR\&W}$ index (calculated by Eq. 6) which classifies the material into six different groups, based on this behaviour: (i) Gravelly sands, when $I_C < 1.31$; (ii) Sands: clean to silty when $1.31 < I_C < 2.05$; (iii) Silty sand to sandy silt when $2.05 < I_C < 2.60$; (iv) Clayey silt to silty clay when $2.60 < I_C < 2.95$; (v) Clays when $2.95 < I_C < 3.60$; and (vi) Organics soils $I_C > 3.60$.

$$I_{CR\&W} = \sqrt{[(3{,}47 - \log Q_t)^2 + (\log F_r + 1{,}22)^2]} \tag{6}$$

Figure 5 shows the $I_{CR\&W}$ classification and the normalized parameters. As can be seen, the organic alluvium shows a normalized Q_{tn} around 5 and an $I_{CR\&W}$ value between 2.95 and 3.60, which classifies the material as clay-like behaviour. The other materials show higher values of Q_{tn} but the $I_{CR\&W}$ varies from 2.05 to 3.60, corresponding to a transitional behaviour (silty materials) to clay-like material.

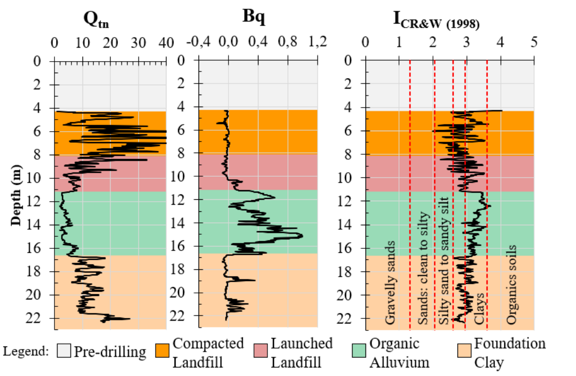

Figure 5. CPTu normalized parameters and soil classification: a) Q_{tn}; b) B_q; and c) $I_{CR\&W}$ (1998).

One of the major factors that influences the soil behaviour classification suggested by Robertson & Wride (1998) and Robertson (2016) is the soil microstructure, such as bonding or aging. To take this occurrence into consideration, Robertson (2016) suggests an evaluation based on the shear wave velocity (V_s). As can be noted in Fig. (6), the V_s values of the organic alluvium are lower than 200 m/s, based on the two tests. Considering the methodology proposed by Robertson (2016), the organic alluvium has no significant microstructure. Based on this, the available methodologies presented in literature to evaluate parameters such as OCR or S_u can be applied.

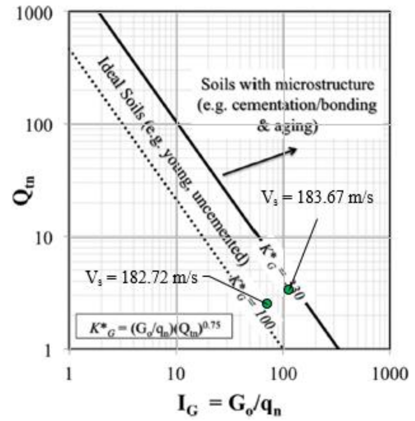

Figure 6. Microstructure evaluation by the shear wave velocity (adapted from Robertson 2016).

Robertson (2016) updated the soil behaviour type index ($I_{CR\&W}$) introducing the I_B, which classifies the soils in three major groups: (i) for $I_B < 22$ the materials are classified with clay-like behaviour; (ii) for $22 < I_B < 32$ materials are classified as transitional (silts in general); and (iii) for $I_B > 32$ the materials exhibit a sand-like behaviour. Also, the author proposed the parameter Contractive/Dilative (CD) to evaluate shear response. CD values higher than 70 indicate dilative response and values lower than 70 indicate contractive behaviour. The CD equal to 70 boundary was developed based on an isoline of OCR = 4 for clay-like materials and a state parameter equal to -0.05 ($\psi > -0.05$) for sand-like materials. The I_B and CD can be calculated by Eq. (7) and Eq. (8) respectively.

$$I_B = \frac{100(Q_{tn}+10)}{(Q_{tn}F_R+70)} \tag{7}$$

$$CD = (Q_{tn} - 11)(1 + 0{,}06F_R)^{17} \tag{8}$$

Figure 7 shows the SBTn proposed to the organic alluvium, the focus of this study. As can be seen, the material has clay-like and contractive behaviour. Based on this, and since the material is probably saturated due the high rate of pore pressure generation (as shown in Fig. (4)), the next step is to verify if the CPTu was performed in an undrained condition.

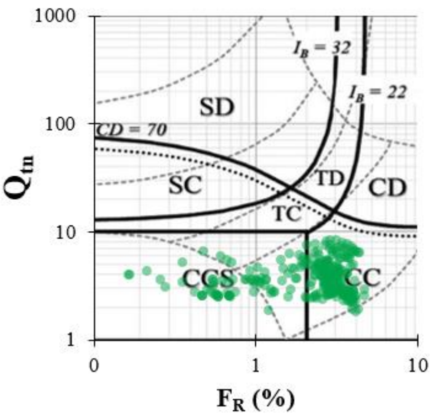

Figure 7. SBTn proposed by Robertson (2016) to the organic alluvium.

4.2. Drainage condition based on the SCPTu

To evaluate the drainage conditions of the organic alluvium, the $I_{Q\text{-}Bq}$ parameter proposed by Mayne et al. (2023) was applied. Figure 8 shows the results and the data were plotted in the undrained behaviour area ($I_{Q\text{-}Bq} < 4$). Based on the results, the undrained shear strength can be evaluated.

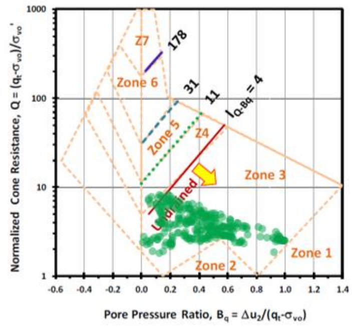

Figure 8. Drainage conditions of the organic alluvium by the $I_{Q\text{-}Bq}$ index (Adapted from Mayne et al. 2023).

4.3. Soil behaviour evaluation by the DMT

As described by Schnaid (2009), the flat dilatometer consists of a stainless-steel blade with a circular, thin flat steel membrane placed in one face. The blade is pushed vertically into the soil using pushing rods which can be adapted from those used in the CPTu. Penetration is halted every 20 cm and the test is performed by inflating the membrane and taking a series of pressure readings at prescribed displacements of 1.1 mm.

The A pressure is required to begin to move the membrane ("lift-off"), and the B pressure is required to move the membrane 1.1 mm against the soil (Totani et al. 2001). These pressures are then corrected to assess the first and second readings, p_0 and p_1 respectively (Schnaid 2009).

To evaluate the in-situ soil behaviour, the Eq. (9) presented by Tot*ani* et al. (2001) was applied. As described by the authors, the material has: (i) clay-like behaviour if $I_D < 0.60$; (ii) transitional/silty behaviour if $0.60 < I_D < 1.80$; and (iii) sand-like behaviour if $I_D > 1.80$. Since the DMT performed did not measure the p_2, the equilibrium porepressure (u_0) used in Eq. (9) was those obtained in the dissipation test from the SCPTu.

$$I_D = \frac{p_1 - p_0}{p_1 - u_0} \tag{9}$$

Is important to emphasize that the parameter I_D is based on the soil behaviour (similar to I_B from the CPTu) and the contours proposed by Totani et al. (2001) can sometimes mislead the classification of clay-like and transitional/silty behaviours due the drainage conditions as detailed by Schnaid et al. (2016). Also, this can be extended to the I_B evaluation from the CPTu.

Totani et al. also suggest the Eq. (10) to assess the Horizontal Stress Index (K_D) and evaluate the contractive/dilative behaviour. Values of K_D approximately equal to 2 ($K_D \approx 2$) indicates a normally consolidated clay and values higher than that indicate an overconsolidated behaviour. In addition to this, it is detailed by Totani et al. (2001) that values of $K_D \approx 5$ are equivalent to OCR = 4, which is the same value used by Robertson (2016) to separate contractive and dilative behaviour of clay-like soils in the SBTn chart.

$$K_D = \frac{p_1 - u_0}{\sigma'_{v0}} \tag{10}$$

Figure 9 shows the data measured by DMT and the in-situ behaviour by I_D and K_D index. All materials show a clay-like behaviour except the foundation clay which shows a silty behaviour (similar as found by the CPTu test in Fig. (5)). Based on the K_D values, all materials have a contractive behaviour (considering the boundary of OCR > 4 to a dilative shear response).

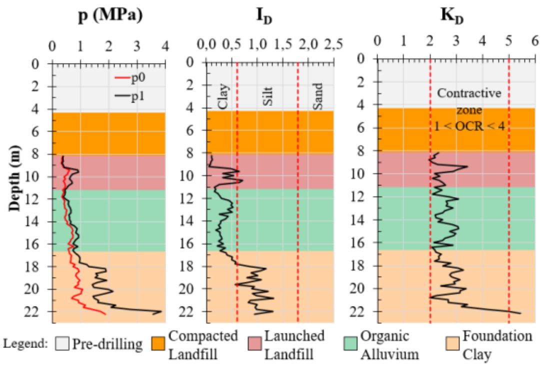

Figure 9. DMT soil behaviour evaluation: a) P_0 and P_1; b) I_D; and c) K_D.

Comparing the DMT result with the CPTu classification (Fig. (5)) it is possibly to note the same conclusion by the $I_{CR\&W}$ and I_B, specially to the organic alluvium (focus of this study). Regarding the shear response, both in-situ tests show a contractive behaviour.

5. Stress history evaluation

The stress history evaluation was performed considering the methodologies based on the cone penetration and the dilatometer data and the result was compared with two oedometer tests.

5.1. Oedometer test

To evaluate the stress history, two oedometer tests were conducted according to the standard D2435-04 (ASTM 2011). Figure 10 shows the results of the oedometer tests, where the mean pre-consolidation pressure (σ'_p) was determined to be equal to 127 kPa using the Pacheco Silva (1970) methodology.

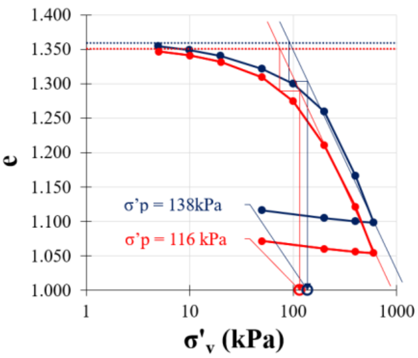

Figure 10. Pre-consolidation pressure by the oedometer tests.

5.2. In-situ tests (DMT and SCPTu)

The stress history evaluation was performed based on the DMT test applying the Eq. (11) detailed by Totani et al. (2001), valid to $I_D < 1.2$ (i.e., clay-like behaviour) and using the CPTu test by Eq. (12) proposed by Chen and Mayne (1996).

$$OCR_{DMT} = 0.5K_D^{1.56} \tag{11}$$

$$OCR_{CPTu} = 0.317\left(\frac{q_t - \sigma_{v0}}{\sigma'_{v0}}\right) \tag{12}$$

Figure 11 shows the result obtained considering the region of the organic alluvium only (focus of this paper). As can be noted, the material has a normally consolidated behaviour and the OCR values vary between 1 and 2. Also, based on the pre-consolidation pressure obtained by the oedometer tests, the point with OCR equal to 1 (comparing the pre-consolidation pressure and the effective stress state) is in the beginning of the profile, indicating a normally consolidated behaviour under this elevation.

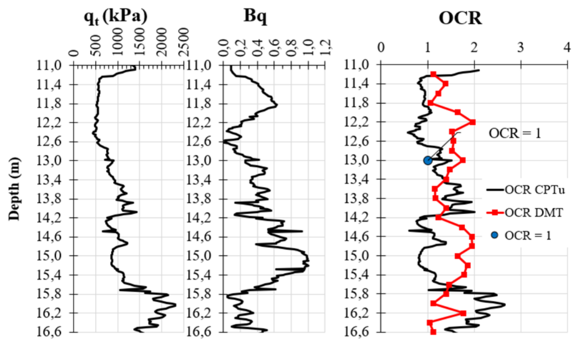

Figure 11. Stress history comparison by the in-situ tests and the oedometer test to the organic alluvium.

6. Undrained shear Strength evaluation

The undrained shear strength evaluation was performed considering field tests (SCPTu, DMT and the FVST) and compared with a triaxial test with isotropic consolidation and undrained shear phase (CIUC).

6.1. Triaxial test Interpretation

To evaluate the shear strength based on the triaxial compression test, four samples were collected in depth using a Shelby sampler (thin wall). The samples were saturated achieving a minimum of 95% B-value as suggested by standard D4767-11 (ASTM 2020). After the saturation phase, isotropic confinements were applied of 50 kPa, 100 kPa, 200 kPa and 400 kPa. Figure 12 shows the deviator stress versus axial strain which is noted a ductile behaviour occurred (no strain softening).

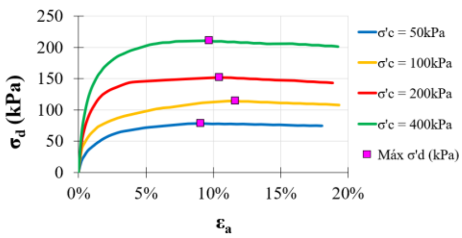

Figure 12. Triaxial tests: deviator stress.

Figure 13 shows the induced pore water pressure due the shear process, where the values increases with the increasing confining stress. In the same figure it is possible to note that the samples achieved the critical state, since no expressive variation of the porewater pressure is observed over the axial strain.

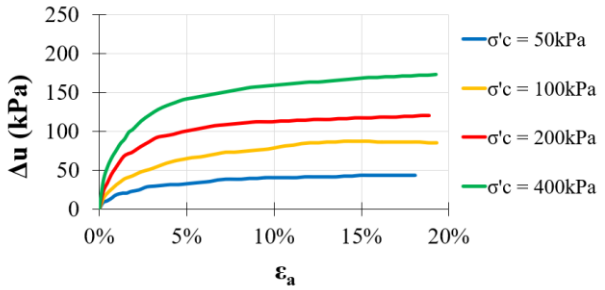

Figure 13. Triaxial tests: induced porewater pressure excess.

To evaluate the shear strength and observe if there are any influence of the stress history, the undrained shear strength ratio (S_u/σ'_c) was calculated over the axial strain, as shown in Fig. (14). The highest shear strength ratio ($S_u/\sigma'_c \approx 0.80$) is observed with the lowest confining stress (50 kPa) and lowest shear strength ratio ($S_u/\sigma'_c \approx 0.20$) is associated with the highest confining stress (400 kPa).

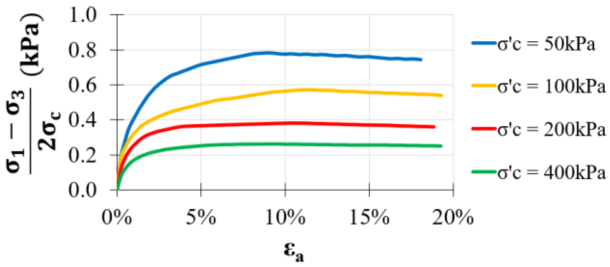

Figure 14. Triaxial tests: undrained shear strength ratio versus the axial strain.

As described by Ladd & Foot (1974), this behaviour is associated with the stress history since the highest undrained shear strength ratio occurs at the highest OCR (equal to 2.5 considering the average pre-consolidation stress calculated by the oedometer test) and the lowest shear strength parameter is obtained when the samples are normally consolidated (OCR = 1). Based on this, a simple model with a constant value of the undrained shear strength ratio (S_u/σ'_{v0}) may not represent the material parameter or contemplate the stress history.

To better fit the shear strength profile of the organic alluvium, the model developed by Ladd and Foot (1974), named as Stress History And Normalized Soil Engineering Properties (SHANSEP), was applied according to Eq. (13).

$$S_u = A + \sigma'_{v0}\, S\, OCR^m \quad (13)$$

Where A is the S_u value with no confining stress, S is normally consolidated shear stress ratio and m is the models' exponent, typically between to 0.70 to 0.90 (Ladd and Foot, 1974).

Figure 15a shows the SHANSEP calibration, where S is equal to 0.35 and the m exponent is equal to 0.94 (upper limit suggested by Ladd and Foot 1974). The R^2 (coefficient of determination) obtained is approximately 0.83, indicating a good fitting of the model into the triaxial data. The A value was assumed equal to zero.

Figure 15b shows the SHANSEP application to evaluate the undrained shear strength considering the criteria of the maximum deviator stress. As can be noted, the model shows adherence to almost all points of the triaxial test, except to the ones submitted to the highest confining stresses.

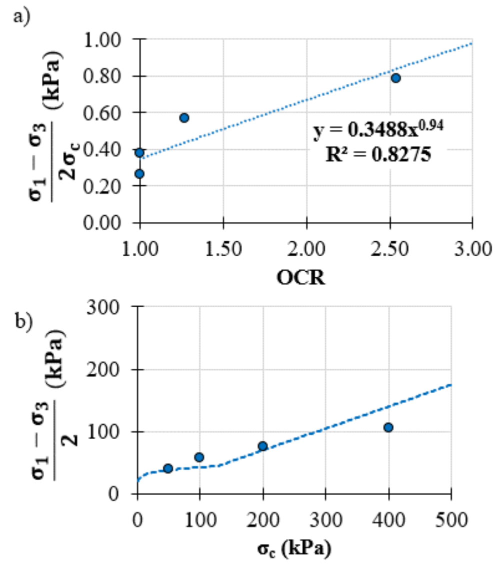

Figure 15. Triaxial tests undrained shear strength evaluation: a) SHANSEP calibration; and b) undrained shear strength envelop.

Considering the SHANSEP parameters (S, and m, since the A value is equal to 0) the next step was to consider the OCR assessed by the in-situ test and obtain the undrained shear strength profile. This was performed using Eq.12 coupled with an OCR estimated using the SCPTu.

6.2. In-situ evaluation by the vane test

To calculate the yield undrained shear strength (S_u) from the field vane shear test (FVST), Eq. (14) was applied according to the international standard D2573-08 (ASTM, 2015). The test was performed considering a rectangular vane blade with dimensions of 13cm x 6.5cm (diameter x high) and the standard velocity rotation of 6 degree per minute.

$$S_{u\,yield} = \frac{6}{7}\frac{M-R}{\pi . D^3} \quad (14)$$

Where M is maximum torque, R is the rod friction, and D the plate diameter.

Regarding the Bjerrum (1973) correction, since the plasticity index obtained in the Atterberg Limits test is approximately 20% (as shown in Fig. (3)), the correction factor μ_r is approximately equal to one and no corrections need to be made.

6.3. In-situ evaluation by the SCPTu

The undrained shear strength evaluation by the SCPTu test can be performed by the bearing capacity factor theory, applying the Eq. (15) and Eq. (16)

presented by Lunne et al. (1997), based on net cone resistance (N_{kt}) and excess porewater pressure ($N_{\Delta u}$).

$$S_{u\ yield} = \frac{q_t - \sigma_{v0}}{N_{kt}} \tag{15}$$

$$S_{u\ yield} = \frac{u_2 - u_0}{N_{\Delta u}} \tag{16}$$

The N_{kt} estimation was performed considering the Eq. (17) suggested by Mayne and Peuchen (2018) which correlates the undrained shear strength of anisotropic triaxial test (CAUC) and CPTu parameters.

$$N_{kt} = 10.5 - 4.6 \ln(B_q + 0.1) \tag{17}$$

The calculation of the $N_{\Delta u}$ was done using Eq. (18) proposed by Battaglio et al. (1997).

$$N_{\Delta u} = 4 + 6\, B_q \tag{18}$$

Also, based on the undrained shear strength obtained by FVST, the correspondent N_{kt} relative to the vane shear mode was calculated by using Eq. (14). Considering the N_{kt} value obtained, the undrained shear strength was extrapolated to entire profile.

6.4. In-situ evaluation by the DMT

The yield undrained shear strength obtained by the DMT was calculated by applying Eq. (19) presented by Totani et al. (2001).

$$S_{u-DMT} = 0.22\, \sigma'_{v0} (0.5 K_D)^{1.25} \tag{19}$$

Valid only to $I_D < 1.2$ (e.g., clay-like behaviour)

6.5. Undrained shear strength comparison

Figure 16 shows the summary of the yield undrained shear strength of the organic alluvium, considering Eq. (13) to Eq. (19) to different shear modes.

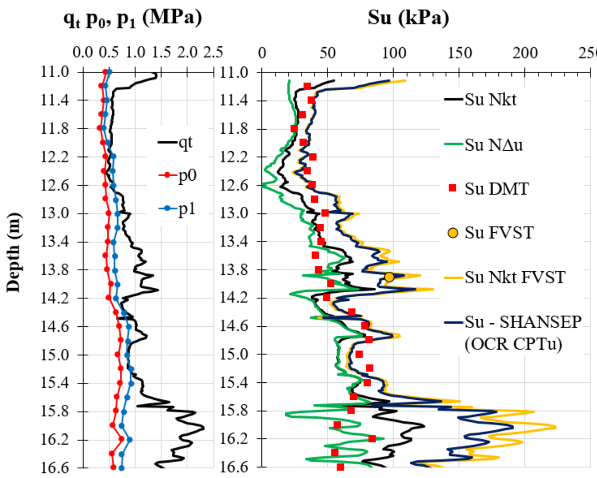

Figure 16. Undrained shear strength comparison between field and laboratory tests.

Using the data of the FVST into the Eq. (15) and the correspondent data (point at the same depth) of cone resistance (q_t) and the total vertical stress (σ_{v0}) the N_{kt} result was equal to 9. This result is close to the lower boundary values considering the common range between 10 to 18 according to Roberston and Cabal (2022).

Considering the N_{kt} equal to 9, the Su was calculated over the depth considering the FVST, as shown in the yellow line in Fig. (16). As can be noted, the highest Su values were those obtained by the Field Vane Shear Test (FVST), which is a similar result of the Bothkennar soft clay presented in Fig. (1).

Considering the SHANSEP model calibrated with the triaxial test (see Fig. (15)) and the OCR obtained by the SCPTu through Eq. (12) (see Fig. (11)) the yield undrained shear strength was calculated over the depth as shown in the blue line of Fig. (16). These results are the second highest boundary of values and just behind those obtained by the FVST.

The methodology presented by Mayne and Peuchen (2018), which consider the CAUC shear mode to assess the N_{kt} (black line in Fig. (16)), results in S_u values lower than the CIUC shear mode. These results also reflect the effect of the type of the confining pressure applied even when the same shear mode is used (triaxial in both cases, but isotropic and anisotropic confining pressures are applied).

Considering the DMT data, the S_u obtained by Eq. (19) was calculated to each measured point, as shown in Fig. (16). The DMT results shows an agreement with the $N_{\Delta u}$ in the middle and at the end of the alluvium with organics (13.0 m to 14.2 m and 15.4 m to 16.6 m).

Finally, the methodology based on the $N_{\Delta u}$ results in a lower boundary value comparing to all the methodologies, probably due to the low rate of porewater pressure generation.

7. Conclusions

This paper presents a case study focusing on the evaluation of the behaviour, stress history, and the yield undrained shear strength of an organic alluvium through laboratory and field tests. Also, geotechnical characterization was determined by laboratory tests (grain-size distribution, Atterberg Limits, specific gravity, natural water content and organic percentage).

Based the SCPTu and the DMT, the soil behaviour and the stress history were evaluated and the organic alluvium was classified as clay-like material with contractive shear response. This classification was seen at all the field tests. In addition, the laboratory data indicate that clay size fraction is the major composition and that the soil has high plasticity. Therefore, the field data show an agreement with the laboratory characterization, indicating that the grain size and the plasticity of the fine fraction governs the compressibility of the organic alluvium in the in-situ condition.

The stress history was assessed by the field test and indicated an OCR between 1 and 2 (evaluated by the SCPTu and the DMT). In complement, the oedometer test led to a similar conclusion when a normally consolidated condition was observed comparing the pre-consolidation stress and the in-situ effective vertical stress.

Considering the SCPTu data, the undrained region was defined by the index $I_{Q\text{-}Bq}$ proposed by Mayne et al. (2023) and indicated that all the organic alluvium behaviour meets an undrained condition, since the values are lower than 4 ($I_{Q\text{-}Bq} < 4$).

The results of the undrained shear strength show the importance of comparing different methodologies based on the SCPTu, DMT and with direct measurement of undrained shear strength through CIUC tests and the FVST to define the best methodology to evaluate the shear strength parameter. Also, this paper showed how to evaluate the influence of the stress history in the CIUC interpretation and the effects of the undrained shear strength evaluation.

Finally, it is important to highlight that the conclusions obtained in this paper are specific to the organic alluvium evaluated and the authors do not recommend a direct replication of the presented results.

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References

American Society for Testing and Materials – ASTM. 2019. D2216-19: Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass

American Society for Testing and Materials – ASTM. 2009. D6913-04: Standard Test Methods for Particle-Size Distribution (Gradation) of Soils Using Sieve Analysis

American Society for Testing and Materials – ASTM. 2011. D2435-04. Standard Test Methods for One-Dimensional Consolidation Properties of Soils Using Incremental Loading.

American Society for Testing and Materials – ASTM. 2015. D2573-08: Standard Test Method for Field Vane Shear Test in Cohesive Soil.

American Society for Testing and Materials – ASTM. 2018. D4318-17e1: Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils.

American Society for Testing and Materials – ASTM. 2020. D2974-14. Standard Test Methods for Moisture, Ash, and Organic Matter of Peat and Other Organic Soils.

American Society for Testing and Materials – ASTM. 2020. D4767-11: Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils.

American Society for Testing and Materials – ASTM. 2020. D792-20: Standard Test Methods for Density and Specific Gravity (Relative Density) of Plastics by Displacement.

Battaglio, M., Jamiolkowski, M., Lancellotta, R., & Maniscalco, R. 1981. "Piezometer probe test in cohesive deposits. In Cone penetration testing and experience" (pp. 264-302). ASCE.

Bjerrum, L. 1973. "Problems of soil mechanics and construction of soft clays and structually unstable soils." Proc. 8th ICSMFE, Moscow, 3, 109-159.

Brown, B., & Gillani, I. 2016. "Common errors in the slope stability analyses of tailings dams." In APSSIM 2016: Proceedings of the First Asia Pacific Slope Stability in Mining Conference (pp. 545-556). Australian Centre for Geomechanics.

Chen, B. S., & Mayne, P. W. 1996. "Statistical relationships between piezocone measurements and stress history of clays." Canadian Geotechnical Journal, 33(3), 488-498.

International Organization for Standardization - ISO 22476-1. 2022: Geotechnical investigation and testing — Field testing — Part 1: Electrical cone and piezocone penetration test.

Janbu, N. 1985. "Discussion." Proc. ASCE Specialty Conf. on In-situ Measurement of Soil Properties, North Canadian University, Raleigh, NC, 2:150–152.

Ladd, C. C., & Foott, R. 1974. "New design procedure for stability of soft clays." Journal of the geotechnical engineering division, 100(7), 763-786.

Lade, P.V. 2016. "Triaxial testing of soils." John Wiley & Sons, Ltd, Chichester, U.K.402 p.

Lunne, T.; Robertson, P.K. & Powell, J.J.M. 1997. "Cone Penetration Testing in Geotechnical Practice." First Edition. Abingdon: E & FNSpon Press.

Mayne P. W. Ethan. C. Greig. J. Styler. M. Entezari. I. Sharp. J., 2023 "A CPT design parameter manual." ConeTec.

Mayne, P. W., Peuchen, J. 2018. "Evaluation of CPTu N_{kt} cone factor for undrained strength of clays." Cone Penetration Testing, 423-429.

Mayne, P.W. 2016. "Evaluating effective stress parameters and undrained shear strength of soft-firm clays from CPT and DMT." Australian Geomechanics Journal 51 (4): 27-55.

Merani, J. M., Hunt, C. E., Donahue, J. L., & Bray, J. D. 2016. CPT interpretation in highly organic soils and soft clay soils. In Geo-Chicago 2016 (pp. 412-421).

Pacheco Silva F. 1970. "Uma Nova Construção Gráfica para Determinação da Pressão de Pré-adensamento de uma Amostra de Solo." In: Congresso Brasileiro de Mecânica dos Solos 4, v2, Tomo I, p.225-232. Rio de Janeiro. Publicação IPT, nº 968.

Robertson, P. K. 2016. "Cone Penetration Test (CPT)-based soil behaviour type (SBTn) classification system—an update." Canadian geotechnical journal, 53(12), 1910-1927.

Robertson, P. K. Kelly O. Cabal. 2022. "Guide to Cone Penetration Testing." 7th Ed. Gregg Drilling LLC.

Schnaid, F. 2009. "In-situ testing in geomechanics: the main tests." CRC Press.

Schnaid, F., Odebrecht, E., Sosnoski, J., & Robertson, P. K. (2016). "Effects of test procedure on flat dilatometer test (DMT) results in intermediate soils." Canadian Geotechnical Journal, 53(8), 1270-1280.

Schneider, J.A., Randolph, M.F., Mayne, P.W., and Ramsey, N.R. 2008. "Analysis of factors influencing soil classification using normalized piezocone tip resistance and pore pressure parameters." Journal of Geotechnical and Geoenvironmental Engineering, ASCE, 134(11): 1569–1586. doi:10.1061/(ASCE)1090-0241(2008)134:11(1569).

Totani, G., Marchetti, S., Monaco, P., & Calabrese, M. 2001. "Use of the Flat Dilatometer Test (DMT) in geotechnical design." On In situ Measurement of Soil Properties, Bali, Indonesia.