Characterization and evaluation of the undrained shear strength of a bauxite mine tailings

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ABSTRACT: The correct determination of the geotechnical properties of mine tailings is of fundamental importance in order to accurately determine the safety of a tailings dam. Commonly, the determination of the undrained shear strength will greatly rely on field investigations, such as CPTu and the Field Vane Shear Test, or laboratory test, like the undrained triaxial compression test. The objective of this paper is to compare different methodologies to evaluate the undrained shear strength of a bauxite mine tailings using field assessment (CPTu and Vane Shear Test) and laboratory tests (i.e., Isotropically Consolidated Undrained Triaxial Compression Test - CIUC). Based on CPTu data different methodologies (empirical and analytical) were evaluated in order to determine the undrained shear strength using the bearing capacity factors N_{kt}, N_{ke} and $N_{\Delta u}$. The results obtained were compared with the Vane Shear Test and the CIUC. Furthermore, the site-specific calibration of the bearing factors (N_{kt}, N_{ke} and $N_{\Delta u}$) to the triaxial compression mode was performed.

1 INTRODUCTION

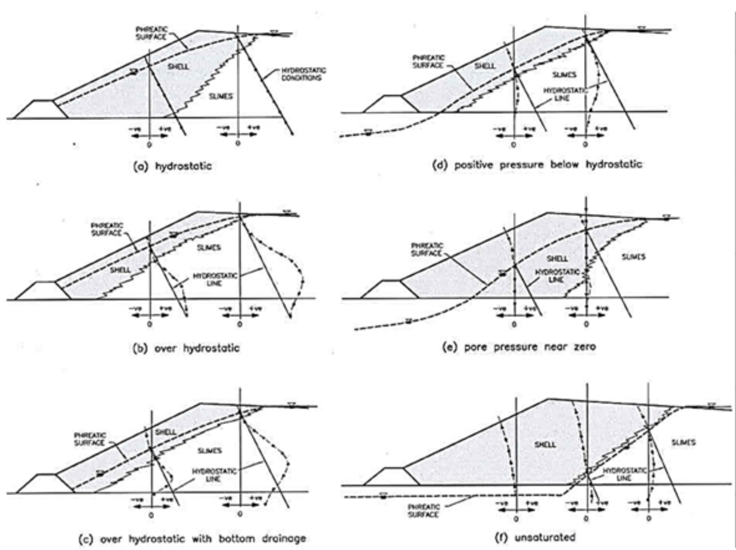
The correct determination of the strength parameters of a soil is highly important in the context of geotechnical engineering since these are crucial aspects for evaluating the stability of a geotechnical structures. If the geotechnical parameters, such as the undrained strength or stiffness, are not correctly determined, there will be a risk of over-dimensioning a geotechnical structure or, which is even of greater concern, one can dimension a structure that will not present enough resilience against the expected load that the structure will face throughout its life cycle. The importance of the physical and chemical characterization of the tailings is also highlighted in the Global Industry Standard on Tailings Management – GISTM (GISTM, 2020).

Case histories of recent failures of tailings storage facilities, like Brumadinho and Mount Polley, demonstrate the importance of a proper understanding of the mechanical behavior of the soils to evaluate the possibility of undrained shear failure (especially where brittle behavior could be expected) and to correctly determine the strength parameters.

To characterize the tailings' geotechnical behavior, one can use different methodologies including field and laboratory tests. Among the field tests commercially available, the Vane Shear Test and the Cone Penetration Test (CPTu) are the ones most often used to determine important properties such as the shear strength and for understanding the in situ of pore pressures profile. As describe by Brown & Gillani (2016), most of the in-situ tests provide only an indirect estimation of shear strength parameters, by using correlations that are predominately developed for natural soils (sedimentary or residual). Since tailings are a by-product of mining with certain unique characteristics, such as geochemistry, angularity, and compressibility, the applicability of the correlations must be evaluated for site specific conditions. Also, the authors, recommend, when possible, to collect high-quality undisturbed samples to perform laboratory tests, such as the isotopically consolidated triaxial test (CIDC/CIUC) or direct simple shear (DSS) to directly measure the shear strength or stiffness parameters.

The CPTu tests are internationally recognized as one of the most important geotechnical in situ tests (Schnaid and Odebrecht, 2012). The test consists of a 60º cone penetrometer pushing equipment and a data acquisition system. The standard test uses a cone with a cross-sectional area of 10 cm² and a 150 cm² friction sleeve located above the cone. The cone penetration is usually carried out with a speed of 2.0±0.5cm/s, with readings being recorded every 1cm to 5cm. This field assessment usually provides three main 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 and iii) the penetration porewater pressure, commonly measured behind the cone tip (u_2).

In conjunction with the CPTu test, it is also common to perform pore pressure dissipation tests, to determine the in-situ equilibrium pore pressure (u_0). The dissipation test consists of a pause in penetration, followed by the measurement of the pore pressure with time. Using the equilibrium pore pressure relative to its depth is possible to evaluate the in-situ pore pressure regime, allowing the correctly characterize the stress state, which governs the soil's strength and deformability. As described by Martin (1999) in Figure 1, the in-situ pore pressure can be categorized into 6 different regimes.



Case	Comments
(a) Hydrostatic	Essentially horizontal seepage flow
(b) Over hydrostatic	Under-consolidated condition
(c) Over hydrostatic with bottom drainage	Under-consolidated condition with double drainage conditions
(d) Positive pressure below hydrostatic	Under-consolidated with vertical seepage or fully consolidated with horizontal and vertical seepage components
(e) Pore pressure near zero	Fully consolidated condition with downward seepage due the bottom drainage
(f) Unsatured	Unsaturated conditions (arid climates)

Figure 1. In situ pore pressure (Martin, 1999).

The undrained shear strength (S_u) can be defined as the soil resistance in a saturated or nearly saturated condition, which is mobilized under a fast loading without allowing time for the soil to change its volume (Lunne et al., 1997). The S_u can be calculated by the CPTu using three independent equations, that rely on the bearing capacity factors: i) for net tip resistance, N_{kt} (Equation 1), ii) for excess porewater pressure, $N_{\Delta u}$ (Equation 2) and iii) for effective cone resistance N_{ke} (Equation 3).

$$S_u = \frac{q_t - \sigma_{v0}}{N_{kt}} \quad (1)$$

q_t – Corrected cone resistance (Equation 4);
σ_{v0} – Total Vertical stress.

$$S_u = \frac{u_2 - u_0}{N_{\Delta u}} \quad (2)$$

u_2 – Penetration porewater pressure (behind the cone tip);
u_0 – Equilibrium porewater pressure obtained from the dissipation test;

$$S_u = \frac{q_t - u_2}{N_{ke}} \quad (3)$$

Different methodologies can be found in the literature to determine the bearing capacity factors, such as Battaglio et al. (1981), Karlsrud et al. (2005), Mayne (2016), Mayne and Peuchen (2018), Agaiby and Mayne (2018), and others. As shown by Herza et al. (2017), the change in the bearing capacity factor, represented by the N_{kt} in Figure 2, as well as the unit weight, will have relevant changes in the factor of safety of the structure.

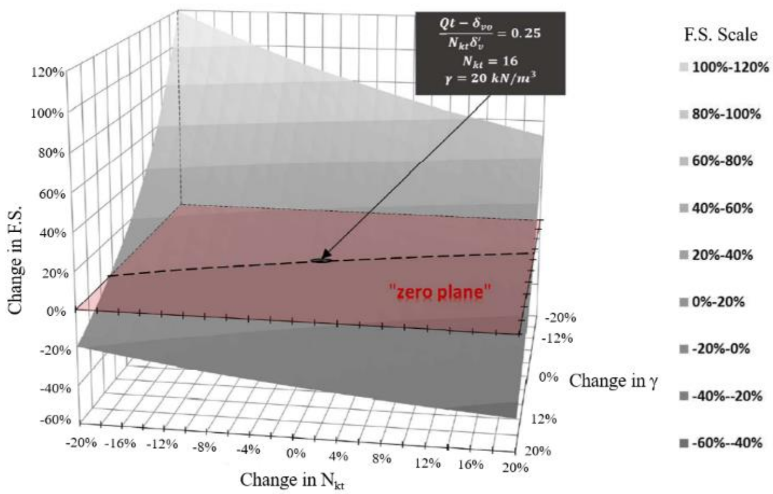


Figure 2. F.S. variations due the Nkt and the unit weight (γ) (Herza et al., 2017).

The Vane Shear Test is the equipment used to determine the undrained shear strength in clay deposits (Schnaid and Odebrecht, 2012). To evaluate the S_u, the Vane Shear Test consists of the rotation of a set of cruciform rectangular blades pushed to pre-defined depths, which can be performed with the blade driven directly into the ground (test type A) or with previous drilling (test type B). The blade's rotation must be controlled, requiring 6±0.6º/min to mobilize an undrained behavior in the tested clay, avoiding the dissipation of the excess porewater pressure generated during the shear as described by the Brazilian standard NBR 10905 (ABNT, 1989).

Finally, in complement of the field assessment, it is understood as best practice to perform laboratory tests, such as the Isotropically Consolidated Undrained Triaxial Compression Test (CIUC), which is standardized by the ASTM D4767:11 (2020). The CIUC test is used to determine the shear strength and stiffness of the soil by axial compression of a soil sample and can be divided into two parts: (i) consolidation phase, usually performed at different stresses that are of interest to the project and; (ii) the shear phase which drives the soil to failure by applying axial loading.

2 METHODOLOGY

To evaluate the bauxite tailings' undrained shear strength (S_u), laboratory and field tests were performed. The field assessment was conducted using the CPTu with dissipation test and the Vane Shear Test. To complement the in-situ characterization, Isotropically Consolidated Undrained Triaxial Compression Tests (CIUC) were performed and considered as the most appropriate mode of shear failure for this material. Also, samples were collected in depth near the CPTu and Vane Shear Test to determine the Solids Content (S.C.) and the unit weight.

The CPTu test was performed following the criteria defined by ASTM D5778 (2020). Following the recommended practice, the porewater pressure measurement was recorded behind the cone, in the u_2 location. The soil behavior-type index was determined using the methodology proposed by Been and Jefferies (1992) and the tailings behavior discussed.

The undrained shear strength determined by the CPTu test was calculated based on the bearing capacity factor, using Equations 1 to 3 presented by Lunne et al. (1997). Such bearing capacity factors were determined by different authors as detailed in the next items. Also, the bauxite tailings' consolidation was evaluated quantitatively using a hybrid formulation of spherical cavity expansion and critical state soil mechanics framework (SCE-CSSM) presented by Agaiby and Mayne (2018) and qualitatively by the methodology proposed by Martin (1999).

The Vane Shear Test was performed following the Brazilian standard NBR 10905 (ABNT, 1989) measuring the yield shear strength and the remolded shear strength (shear strength under large deformations). Using both yield and remolded, it will be calculated the soil sensitivity (S_t), as defined by the ratio of the yield shear strength to the remolded shear strength. Also, the Vane Shear Test was performed next to the CPTu test, making it possible to compare results.

To compare the results of the field assessment, three samples of the bauxite tailings were collected and the CIUC tests were performed, using the confined pressures of 50kPa, 100kPa, and 200kPa. Using the laboratory test, the normally consolidated shear strength ratio based on the maximum deviatoric stress and the slope of the critical state line in the p' x q space (M_{tc}) was calculated.

2.1 *Cone Penetration Test with Pore pressure Measurement*

Using the CPTu data, the corrected cone resistance (q_t) values and the normalized porewater pressure parameter (B_q) were determined by using the Equations 4 and 5 respectively.

$$q_t = q_c + u_2(1 - a) \tag{4}$$

a – Cone area ratio, considered to be equal to 0.80.

$$B_q = \frac{u_2 - u_0}{q_t - \sigma_{v0}} \tag{5}$$

2.1.1 *Battaglio et al. (1981)*

Based on an extensive database, Battaglio et al. (1981) found a relationship between the normalized porewater pressure parameter and the bearing factor for excess porewater pressure as shown by Equation 6.

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

2.1.2 *Karlsrud et al. (2005)*

Using the B_q values and the soil sensitivity, Karlsrud et al. (2005) develop a methodology to calculate the N_{ke} values, as detailed in Equation 7, valid for sensitivity lower than 15 ($S_t < 15$), and $N_{ke} > 2.0$.

$$N_{ke} = 11.5 - 9.05B_q \tag{7}$$

2.1.3 *Mayne (2016)*

Based on the Spherical Cavity Expansion (SCE), Mayne (2016) develop Equation 8 (valid for $Bq \neq 1.0$) Equation 9 to determine $N_{\Delta u}$ and N_{ke}.

$$N_{\Delta u} = \frac{3.90}{\left({}^{1}/_{B_q}\right) - 1} \tag{8}$$

$$N_{ke} = {}^{2}/_{M_{tc}} + 3.90 \tag{9}$$

M_{tc} – slope of the critical state line in the p' x q space;

2.1.4 *Mayne and Peuchen (2018)*

Based on a database of 407 high-quality triaxial compression tests (CAUC), for a total of 62 different clays categorized into five groups based on their varying degrees of stress-history (ranging from fissured to sensitive clays), as well as for different test conditions (onshore and offshore), the researchers Mayne and Peuchen (2018) developed a relationship between B_q and the bearing factor for net tip resistance N_{kt}, expressed by the Equation 10.

$$N_{kt} = 10.5 - 4.6 \,.\, (B_q + 0.1) \tag{10}$$

2.1.5 *Agaiby and Mayne (2018)*

Agaiby and Mayne (2018) developed analytical equations to determine the overconsolidation ratio (OCR) and N_{kt} using SCE-CSSM. The methodology suggested by the authors proposes that the OCR (Equations 12 to 14) and the N_{kt} (Equation 15) are expressed as a function of the soil rigidity index (I_R), which can be determined using Equation 11.

$$I_R = \exp\left[\frac{1.5 + 2.925 \cdot M_{tc} \cdot a_q}{M_{tc} \cdot (1-a_q)}\right] \tag{11}$$

I_R – Rigidity Index;

a_q – Slope of the chart made by u_2 - σ_{vo} (y-axis) versus q_t - σ_{vo} (x-axis).

$$OCR = 2\left[\frac{\left({}^{2}/_{M_{tc}}\right) \cdot (q_t-\sigma_{vo})/\sigma'_{vo}}{{}^{4}/_{3} \cdot (\ln I_R + 1) + \frac{\pi}{2} + 1}\right]^{\frac{1}{\Lambda}} \tag{12}$$

$$OCR = 2\left[\frac{1}{1{,}95\ .\ M_{tc} + 1}\frac{(q_t-u_2)}{\sigma'_{vo}}\right]^{\frac{1}{\Lambda}} \tag{13}$$

$$OCR = 2\left[\frac{({}^{u_2-u_0}/_{\sigma'_{vo}})-1}{{}^{2}/_{3} \cdot M_{tc} \cdot \ln(I_R) - 1}\right]^{\frac{1}{\Lambda}} \tag{14}$$

σ'_{v0} – Vertical effective stress;

$$N_{kt} = {}^{4}/_{3} \cdot [\ln(I_R) + 1] + {}^{\pi}/_{2} + 1 \tag{15}$$

The Λ parameter is the plastic volumetric strain potential (1-C_s/C_c). Herein it was adopted the value of 0.80, as recommended by Agaiby and Mayne (2018).

2.1.6 *Been and Jefferies (1992)*

Using a critical state framework, Been and Jefferies (1992) developed a soil behavior classification index (I_C). The I_C is calculated by Equation 16, using the normalized cone tip resistance (Q), Equation 17, the normalized sleeve friction (F), Equation 18 and the normalized porewater pressure parameter (Equation 5). The I_C range is detailed in Table 1.

$$I_c = \sqrt{\left[\left(3 - \log(Q(1 - B_q) + 1\right)^2 + (1.5 + 1.3\log F)^2\right]} \tag{16}$$

$$Q = \frac{q_t-\sigma_{v0}}{\sigma'_{v0}} \tag{17}$$

$$F = \frac{f_s}{q_t-\sigma_{v0}} \tag{18}$$

Table 1. Relationship between Soil Behavior-Type descriptions and I_C - Been and Jefferies (1992).

Zone	CPTu Index I_C	Soil Behavior Classification
6	$I_C < 1.80$	Sands – clean sand dan gravel to silty sand
5	$1.80 < I_C < 2.40$	Sand mixtures – silty sand to sand silty
4	$2.40 < I_C < 2.76$	Silt mixtures – clayey silt to silty clay
3	$2.76 < I_C < 3.22$	Clays
2	$3.22 < I_C$	Organic soils

2.2 *Vane Shear Test*

To calculate the undrained shear strength, using the Vane Shear Test, it was used the Equation 19 suggested in the Brazilian Standard NBR 10905 (ABNT, 1989). Also, the sensitivity can be calculated using the Equation 20, which express the relationship between the yield shear resistance and the remolded shear resistance, providing an idea of the soil brittleness.

$$S_u = 0.86\left(\frac{T}{\pi\ .D^3}\right) \tag{19}$$

T – Maximum torque measured by the Vane Shear Test, in yield or remolded conditions;

D – Vane diameter (used $6{,}5 \times 10^{-3}$m as provided by the company that performed the test).

$$S_t = \frac{Su_{yield}}{Su_{remolded}} \tag{20}$$

2.3 *Undrained Consolidated Triaxial Compression Test*

The CIUC tests were performed in undisturbed samples to determine the normally consolidated undrained shear strength ratio using the criteria of the maximum deviatoric stress. Using the results of the triaxial tests the slope of the critical state line in the p' x q space (M_{tc}) was also determined.

3 RESULTS

3.1 *CIUC test*

As shown in Figure 3, the 3 samples generated a high shear-induced excess porewater pressure. Also, it is noted that the bauxite tailings do not lose its resistance during shear showing a very ductile and clay-like behavior, in accordance with what is expected of very plastic tailings. Using the laboratory data, the M_{tc} value was found to be ≅ 1.72 and the normally consolidated undrained shear strength ratio was equal to ≅ 0.32.

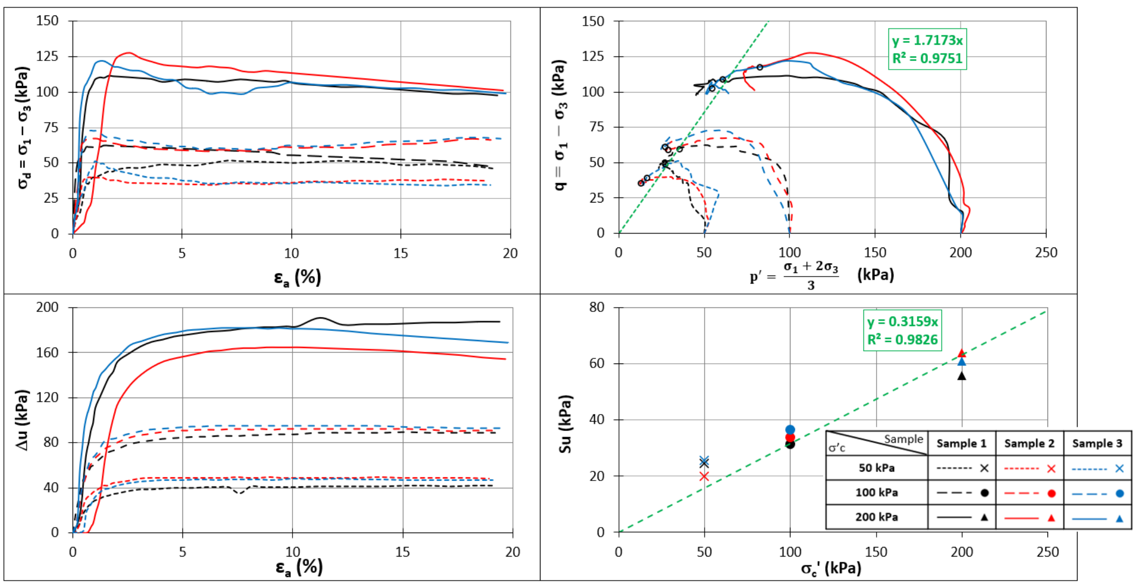


Figure 3. Summary of CIUC test.

3.2 *Field Assessment*

Using the dissipation test, the equilibrium porewater pressure was determined and interpolated over the CPTu test. Figure 4 shows the summary of the main CPTu parameters along with the porewater pressure profile for the tailings. In this figure, the penetration porewater pressure (u_2) is plotted along with the equilibrium pore pressure (u_0) profile and a condition of 100% hydrostatic for comparison. Also, in Figure 4, the results of solids content and unit weight are plotted with depth.

Analyzing Figure 4, three points can be highlighted: (i) the bauxite tailings analyzed generates high porewater pressures during penetration (which is common for saturated and loose clayey soils); ii) the correction of q_c to q_t is relevant, showing a difference around 100kPa (≈33%) at the end of the CPTu profile; and (iii) the seepage conditions measured by the dissipation test indicates an over hydrostatic with bottom drainage condition (case "c" suggested by Martin (1999) and detailed in Figure 1).

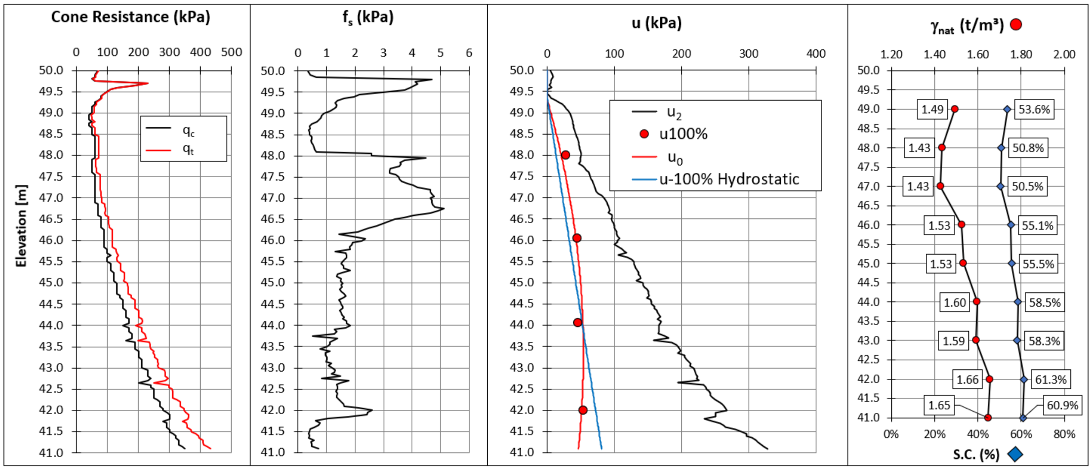


Figure 4. Dissipation test data, Solids Content and Unit Weight.

Using the unit weight and porewater pressure profile, the total and effective vertical stresses were calculated which allows for the calculation of B_q and I_c, as shown in Figure 5.

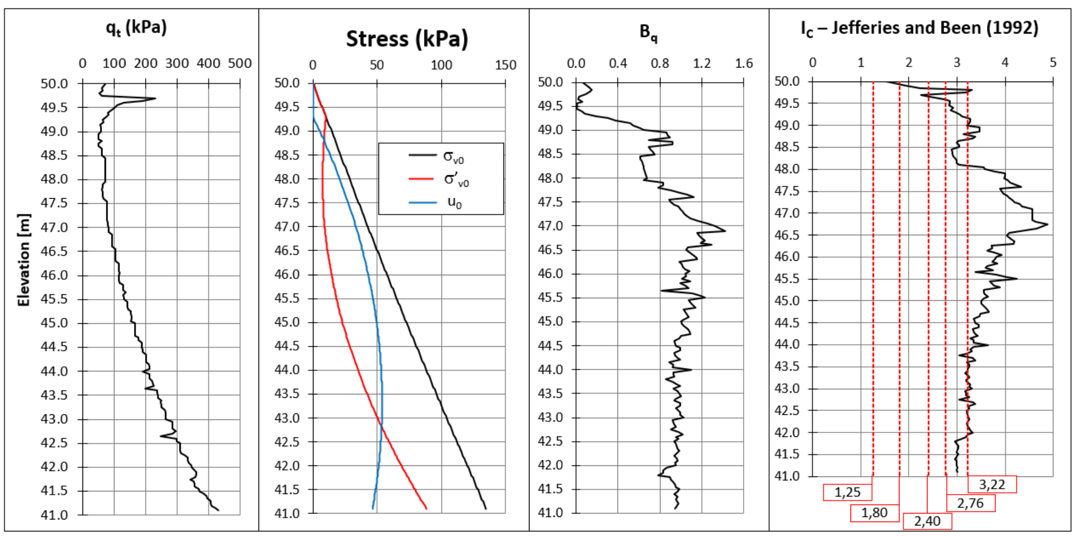


Figure 5. CPTu classification and stress state.

As observed in, the bauxite tailings show a behavior of clayey soil ($2.76<I_C<3.22$) and organic soil ($I_C>3.22$) by using the soil behavior-type classification proposed by Been and Jefferies (1992). The I_C index classifies the soil based on its behavior and not on the composition of the material (grain-size distribution and plasticity). The organic soil classifications highlight the fact that the tailings is highly compressible, contractive and saturated, which is also possible to notice by the low values of cone tip resistance (q_t<1.0MPa) and high porewater pressure generated during the test (indicated by the values of B_q).

Using Equation 11, the I_R obtained was 270.7 by using a calculated a_q of 0.65. Using these parameters, and the M_{tc} value of 1.72 (Figure 3), the OCR was calculated (Equations 12 to 14) as proposed by Agaiby and Mayne (2018) and is shown in Figure 6.

The bauxite tailings are predominantly classified as under consolidated (OCR<1) as shown in Figure 6. Also, this result is supported by qualitative analysis proposed by Martin (1999) based on the porewater pressure profile (case "c" suggested by Martin (1999) detailed in Figure 1).

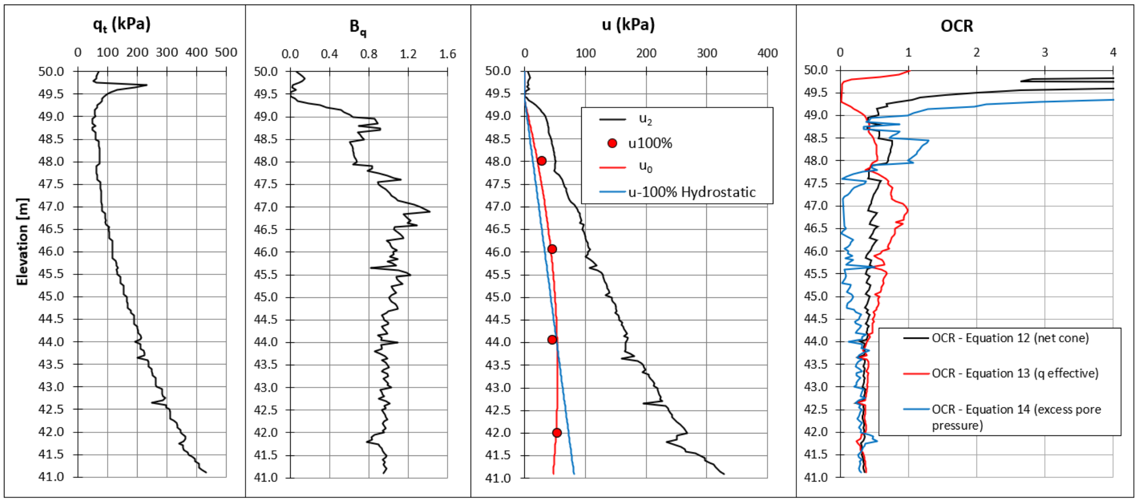


Figure 6. CPTu profile including an OCR evaluation.

After the evaluation of the stress history, the next step is to determine the region of the profile that showed undrained behavior during the CPTu. As described by Schnaid (2009), regions of the investigation where $B_q<0.40$ is probably responding in a drained or partially drained manner (such as the initial portion of the investigation, between the elevations 50.0m and ≈49.2m on (Figure 5 or Figure 6) and should not be considered as undrained response.

Another way to evaluate the undrained behavior, as shown by Robertson and Cabal (2015), is to compare the results of the undrained shear strength calculated based on the bearing factors N_{kt} and $N_{\Delta u}$. Herein, this comparison was done using the proposed equations by Mayne and Peuchen (2018) and Battaglio et al. (1981), respectively. When the values of undrained shear strength obtained from Equations 1 and 2 are approximate, there is a greater probability that the cone drilling is occurring in an undrained manner. Using these criteria, it was determined that the calculation of the undrained shear strength should be performed below the elevation of 49.2m, as shown in Figure 7.

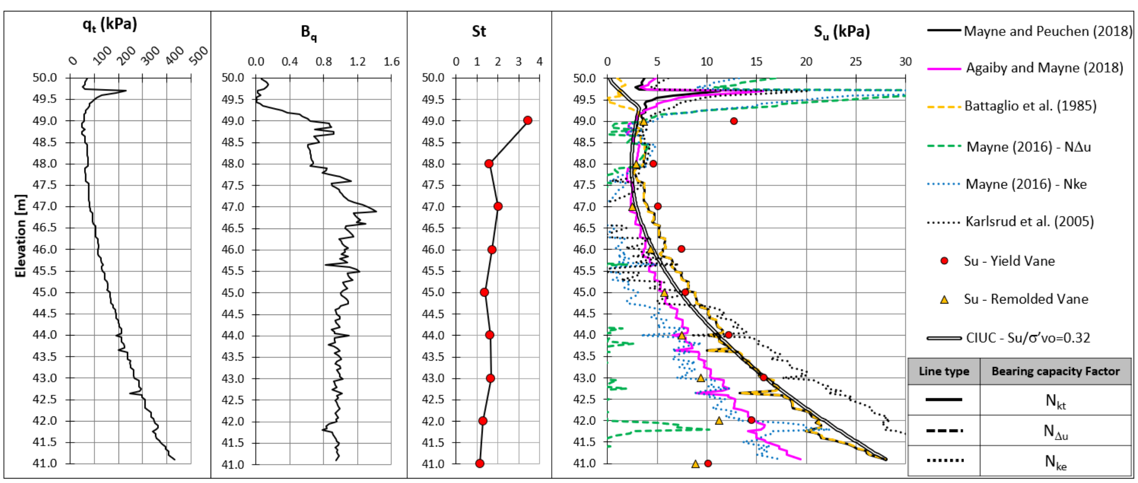


Figure 7. CPTu Undrained Shear Strength evaluation.

Based on the results from the methodologies studied herein the Mayne and Peuchen (2018), the Battaglio et al. (1981) formulations are the ones that more accurately calculated the undrained shear strength based on the compression mode, which is represented by the undrained strength profile calculated from the triaxial compression test.

The results from the Field Vane Test showed, at most of the profile, an upper bound value above that of the triaxial compression mode. Similar results have been reported in the literature, such as those from Bothkennar soft clay showed by Mayne (2016). Under the elevation 42.0m, the shear strength obtained from Vane Test Tests showed decreasing values, which was not observed on the CPTu.

The methodology proposed by Karlsrud et al. (2005), which uses the N_{ke} values, showed S_u values lower than expected from the triaxial compression mode above 45.0m and higher at end of the CPTu profile, in elevations below 45.0m. The methodology proposed by Mayne (2016) to evaluate N_{ke}, on the other hand, showed values lower than expected for the triaxial compression mode over all of the CPTu profile.

The methodology proposed by Agaiby and Mayne (2018) yielded values of undrained shear strength that were very close to the remolded shear strength over the entire profile. In Figure 7 it is noted that Equation 8, proposed by Mayne (2016) for $N_{\Delta u}$ showed very inconsistent results of S_u values. These results happened due to the mathematical limitation imposed by the Equation 8 when $B_q \approx 1.0$.

Furthermore, the current work intended to determine the site-specific calibration of the bearing factors (N_{kt}, N_{ke} and $N_{\Delta u}$) to the triaxial compression mode. As can be seen in Figure 8, the bearing factors of N_{kt}=11, N_{ke}=3 and $N_{\Delta u}$=10. are reasonably good bearing factors to be used to estimate the S_u relative to the triaxial compression mode based on CPTu.

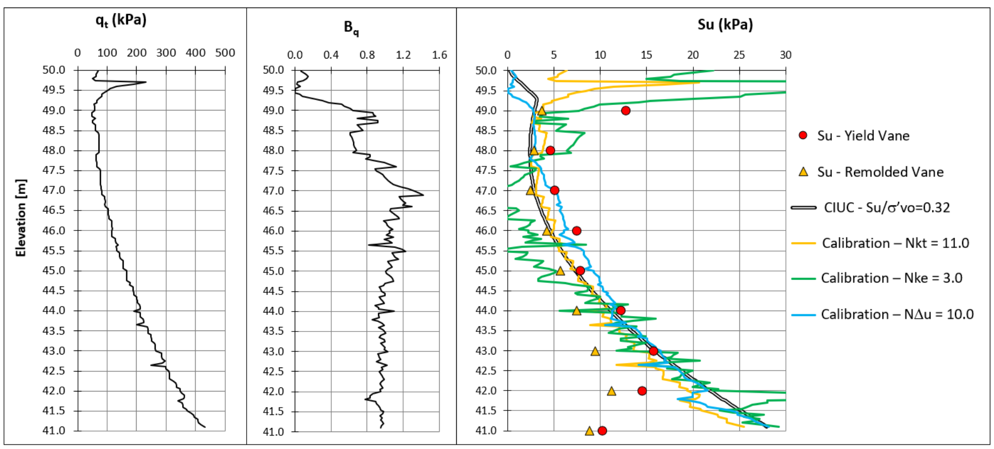


Figure 8. CPTu bearing factors N_{kt}, N_{ke} and $N_{\Delta u}$ for calibration.

4 CONCLUSION

The comparison of different methodologies using field assessment to calculate the undrained shear strength of a bauxite mine tailings was performed to determine which formulations are most appropriate to estimate the undrained shear strength representative of the compression failure mode. Furthermore, the soil behavior-type index using the equations proposed by Been and Jefferies (1992) and the OCR using the suggestion by Agaiby and Mayne (2018) were evaluated.

The undrained shear strength was calculated in the portions of the sounding where B_q>0.4 where the results calculated by Mayne and Peuchen (2018) and Battaglio et al. (1981) yielded similar values, as suggested by Schnaid (2009) and Robertson and Cabal (2015). For the tailings evaluated herein the B_q values indicate that it is very likely that the CPTu was fully undrained below the elevation of 49,0m, as can be seen by the high B_q values (Figure 7).

The undrained shear strength calculated by the bearing factor for net tip resistance (N_{kt}) using Mayne and Peuchen (2018), and the bearing factor for excess porewater pressure ($N_{\Delta u}$) suggested by Battaglio et al. (1981), were the ones to more accurately determine the expected results for the triaxial compression failure mode. The values of the undrained shear strength calculated by the Vane Shear Test yield an upper bound value above that of the triaxial compression failure mode. Similar findings have been reported in the literature such as the example of the Bothkennar soft clay shown by Mayne (2016).

The equation proposed by Karlsrud et al. (2005), based on N_{ke} values, yielded results above that of the triaxial compression mode especially at the end of the CPTu. All the other methods resulted in values below the expected for the triaxial compression mode. The methodology proposed by Agaiby and Mayne (2018) to calculate N_{kt} showed convergence to the remolded shear strength calculated by the Vane shear test, as shown in Figure 7, and the methodology proposed by Mayne (2016) to evaluate $N_{\Delta u}$ did not present reliable results due to its inherent mathematical restriction for $B_q \approx 1.0$ in the Equation 8.

Finally, the site-specific calibration of the bearing factors (N_{kt}, N_{ke} and N_{Δ}) to the triaxial compression mode was presented yielding values of N_{kt}=11, N_{ke}=3 and $N_{\Delta u}$=10.

It is important to highlight that the conclusions obtained in this work were specific to the material being evaluated and the authors do not recommend a direct replication of the results presented herein before a site-specific study of the behavior of the geomaterials involved.

5 ACKNOWLEDGMENTS

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