Assessing Undrained Shear Strength in the Antucoya Mine Waste Dump

Ricardo Moffat, *Civil Engineer, Universidad Adofo Ibáñez, Santiago, Chile; email: ricardo.moffat@uai.cl*
Boris Duarte, *Mining Engineer, Minera Antucoya, Antofagasta, Chile; email: bduarte@antucoya.cl*

ABSTRACT: *A plan to increase the current height of a mine waste dump necessitates implementing a comprehensive testing program. This program includes in-situ tests such as SCPT, DMT, and dissipation tests, as well as laboratory tests such as soil classification and undrained triaxial tests. Additionally, monitoring instruments such as piezometers and TDRs are installed to assess the soil structure and its characteristics. This paper estimates the soil's undrained strength based on in-situ testing and laboratory tests. A comparative analysis of the values of undrained strength obtained by different methods is performed. The effects of these values on a total stress-based stability analysis are evaluated to determine their influence on the waste dump's safety factor. The study utilizes in-situ pore water pressure data from dissipation tests and on-site piezometers. This information is crucial in calculating the effective stress and the ratio S_u/σ'_v. A static analysis uses different combinations of S_u/σ'_v values and pore water pressure to assess the slope stability. The study reveals that the resulting safety factor remains consistent when the same water pressure distribution is utilized to determine S_u/σ'_v and the numerical model.*

KEYWORDS: undrained shear strength; in-situ testing; CPT; DMT; slope stability

SITE LOCATION: Geographic Database

INTRODUCTION

The Antucoya copper mine, situated in Antofagasta, Chile, is a mining project that obtains minerals using an open pit operation. This material is then processed using the leap leaching method. Subsequently, the waste material is deposited in waste dump stockpiles using mechanical belts, which transport and deposit the material in layers on the field. The stability analysis of these stockpiles is paramount for this project and similar endeavors, as it holds significant economic implications for the mining business. The material observed in these stockpiles generally exhibits a fines content ranging from 25 to 40% and is classified as sandy soil (SM-SC to SC). This material is crushed, and acid agglomerate is also used before heap leaching. It is noteworthy that the large-size particles (sands) float in the fine particle matrix; therefore, this fine matrix appears to dominate its behavior.

The geotechnical design of the stockpiles for this project requires characterizing the shear strength of the deposited soil. This geotechnical characterization is usually performed using laboratory and in-situ material testing. A substantial geotechnical campaign was carried out for this project to provide strength parameters for the geometrical stockpile design, primarily maximum height and slope angle. Stefanow and Dudzinki (2021) comprehensively summarize over 20 measurement methods for determining soil shear strength. Obtaining an accurate value of shear strength requires careful consideration of various factors, including the scale effect, strain rate effect, soil anisotropy, and stress paths. These factors are crucial in ensuring the adequacy and reliability of the deduced shear strength values.

Undrained shear strength is a fundamental parameter in geotechnical engineering that is used to design and analyze a wide range of earth structures, such as earthworks on soft clays to clayey sands, as can be found in Wei et al. (2023) and Asfaw et al. (2023). This shear strength frequently determines the maximum allowable height and slope of stockpiles; in soft fine soils or coarser soils with a matrix of fines, the undrained loading condition is the least favorable. The undrained shear strength (S_u) is not a single soil parameter, and its value depends on the testing method, stress path during testing, rate of loading, and other factors (see Mayne, 2016). Depending on the deformation of the soil, the undrained strength can be considered as an undisturbed (or peak) or residual value, the latter being when the soil experiences enough deformation to pass the peak strength and decrease to a steady value. Both are relevant for geotechnical design problems such as pile design, slope stability,

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bearing capacity, etc. In geotechnical practice, many methods are used to measure this value, from correlations, for example, using the plasticity index. One of the best-known correlations is that proposed by Skempton (1957), which relates the undrained shear strength of normally consolidated soils to the effective stress and the Atterberg limits by the following equation:

$$\frac{S_u}{\sigma'} = 0.11 + 0.0037(\mathrm{PI}) \qquad (1)$$

where S_u is the undrained shear strength, σ' is the effective stress, and PI is the plasticity index.

In geotechnical practice, various methods are employed to measure this parameter. These methods range from correlations, such as the one previously illustrated involving the plasticity index, to laboratory and in-situ tests. Direct shear and triaxial tests are the most common laboratory tests employed. Conversely, several in-situ testing techniques are utilized to accurately depict the soil's behavior in its natural or in-situ state. These include the cone penetrometer test (CPT), dilatometer test (DMT), vane shear test, and other methods. These in-situ tests enable the direct measurement of the undrained shear strength of the soil within its natural environment without disrupting the soil's structure or properties. A comprehensive description of CPT and DMT in-situ testing methods can be found in detail in Robertson and Cabal (2015) and Totani et al. (2001).

There are few projects where information can be obtained from different tests to derive the undrained shear strength (S_u) in soft soils. However, the Bothkennar site is a homogeneous UK national testing site on high plasticity clays. Undisturbed soil samples (piston and block) have been obtained and tested using triaxial equipment. CPT, DMT, pressuremeter tests, and vane tests have also been carried out at various depths on the site. Figure 1 summarizes some of the S_u values Powell (2001) and Hight et al. (1992) obtained on this site using different tests. Generally, excellent agreement is observed between values from compression triaxial tests on block samples and peak values from vane shear tests, CPT (using N_{kt}=10), and DMT tests. Directly comparing DMT and CPT tests from Powell (2001), it is observed that values from DMT are a little lower than S_u values derived from CPT tests (using N_{kt}=10). Equations used to calculate S_u are shown later in this work. Similarly, Wesley (2010) reveals data from CPT soundings, field vane tests, triaxial compression tests, and laboratory vane tests on clays of volcanic origin on the Hamilton site in New Zealand (see Figure 2). As depicted in this figure, an excellent agreement between the results from CPT testing and other tests is observed on this site.

Macek et al. (2019) also compare CPT and DMT tests for soft tailings. They find that S_u values determined using an approximation given by Marchetti (1980) are lower than those deduced using the CPT tip resistance and equation suggested by Robertson and Cabal (2015). A Field Vane Test (FVT) compares better with S_u values derived from CPT than DMT. The undrained shear strength of remolded samples correlates well with residual strength measured with FVT.

Stark et al. (2009) present another case study in which the undrained shear strength is measured using CPT soundings on Craney Island near Norfolk, Virginia, U.S. The stability of the west perimeter dike on this dredge material island is evaluated using an undrained strength stability analysis. The effect of the increase of undrained strength due to accelerated consolidation is considered in the stability analysis.

This paper focuses on determining undrained shear strength values from the correlation of CPT results, DMT tests, and consolidated undrained triaxial tests performed on soil specimens from the field and reconstituted in the laboratory at the average density observed on the field. A comparison of the obtained values of S_u is performed. The effect of pore water distribution is evaluated on the deduced value of S_u/σ'_v. In total, 59 CPT soundings, 2 DMT tests, and 15 consolidated undrained triaxial tests are summarized and evaluated.

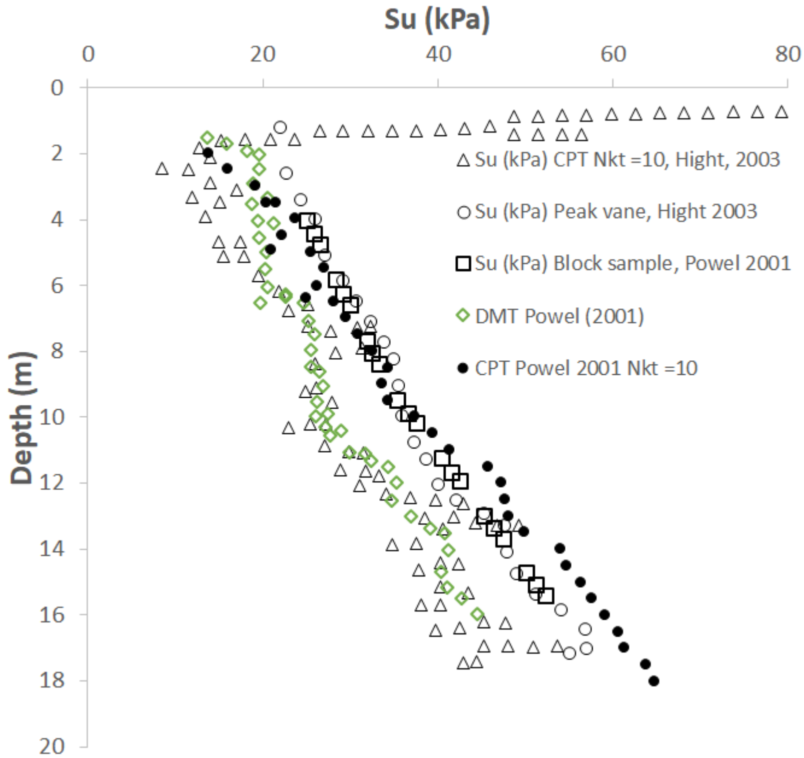

Figure 1. Undrained strength (S_u) from the Bothkennar site, UK. Data from Powel (2011) and Hight et al. (1992).

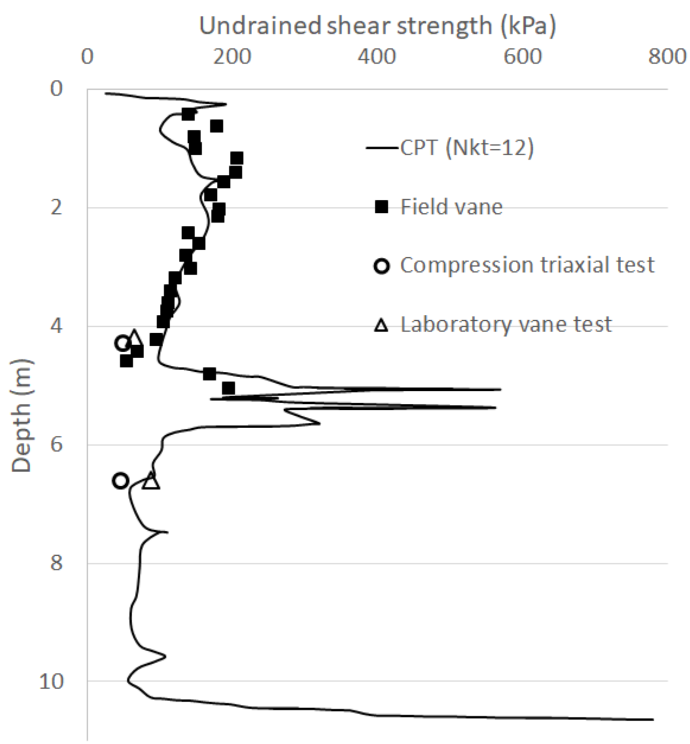

Figure 2. Undrained shear strength from in-situ and laboratory tests at the Hamilton site. Data from Wesley (2010).

IN-SITU TESTING

59 CPTs were performed in the field at different locations of the stockpile. Two DMTs were performed a few meters from two CPT soundings (CPT46 and CPT21).

The CPTs were conducted following the guidelines in ASTM D5778-20. Cones with base areas of 10 cm² and 15 cm² were used, and the penetration rate was maintained at 2 cm/sec, employing a 22-ton truck for the testing (refer to Figure 3). These tests allow for soil penetration up to the base of the stockpile, with data being recorded at 2 cm depth intervals. Refer to ASTM D5778-20 and Robertson and Cabal (2015) for detailed information on the CPT equipment and testing procedures.

The critical measurements obtained from the CPT cones include the tip cone resistance (q_c), sleeve friction (f_s), and the penetration pore pressure behind the cone tip (u_2). Corrections are applied to account for the unequal end area effect, as Campanella et al. (1982) mentioned, a correction factor (a) of 0.75 was considered for the specific cones used in this study.It is worth noting that this correction is particularly crucial when dealing with soft soils, as mentioned in Robertson (2009).

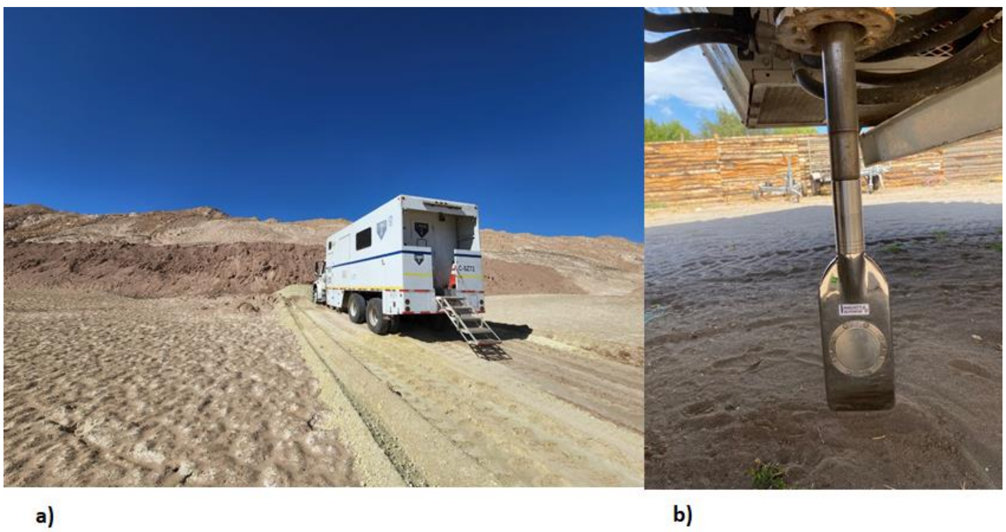

Figure 3. In-situ testing: a) CPT truck, b) DMT probe.

DMT (Dilatometer) testing was conducted utilizing the same truck depicted in Figure 3, following the procedures outlined in ASTM D6635-01. Measurements were taken at 20 cm intervals in depth for both DMT tests.

In this study, the dilatometer used is depicted in Figure 3b. It is a flat steel blade with a thin circular membrane attached to its surface. During the testing process, the operator inflates the membrane and records two primary readings: the A pressure,also known as lift-off pressure; and the B pressure, required to displace the center of the membrane by 1.1 mm against the tested soil. A supplementary reading, C, the closing pressure, can be obtained by gradually deflating the membrane after reaching the B reading.

Two values, ΔA and ΔB, are determined by calibrating the membrane's stiffness. These values are crucial in converting the obtained data into p_0 and p_1, which are practical parameters used to deduce various geotechnical properties for design purposes (Mayne, 2016).

TRIAXIAL TEST RESULTS

15 consolidated undrained triaxial tests were performed on reconstituted soil specimens using soil from the stockpile. These tests were conducted on reconstituted saturated soil samples, at 50, 100, and 150 kPa confining stresses. The density of the specimens closely matches the field conditions during the testing phase. The testing procedure follows the guidelines outlined in ASTM D4767-04, employing a strain deformation rate of 0.13% per minute. The reconstituted soil specimens have an approximate diameter of 10 cm and a height of 20 cm. Plasticity index values vary typically between 7 and 13.

Figure 4 displays an example of results obtained from consolidated undrained triaxial tests. The tests provide values of S_u (undrained strength) for each initial effective stress applied. It should be noted that the total unit weight of the soil is considered to be 17 kN/m³, void ratio of approximately 0.65, moist content between 8 to 12%, and the pore water pressure on the stockpile is considered, as first approximation, constant at 0 kPa. As a result, the corresponding representative depths for the confining pressures of 50, 100, and 200 kPa are approximately 3 m, 6 m, and 12 m, respectively. Later, it will be shown that this assumption is valid for depths 3 m and 6 m, but there is a higher water pressure at 12 m. Therefore, the actual representative depth for 200 kPa could be higher than 12 m. These depth values will be later compared with the undrained strength values deduced from CPT soundings and DMT tests.

Figure 5 summarizes deduced values of undrained strength (S_u) obtained from triaxial tests on reconstituted specimens compacted at an average density as observed on the field. Applied confining pressure was converted to equivalent depth according to deduced stress. The soil was taken from different stockpile locations, which may explain the value difference, especially in soil C5-C6. Considering all data except C5-C6, the undrained strength can be regarded, on average, as:

$$S_u = 0.45 \cdot \sigma'_0 \tag{2}$$

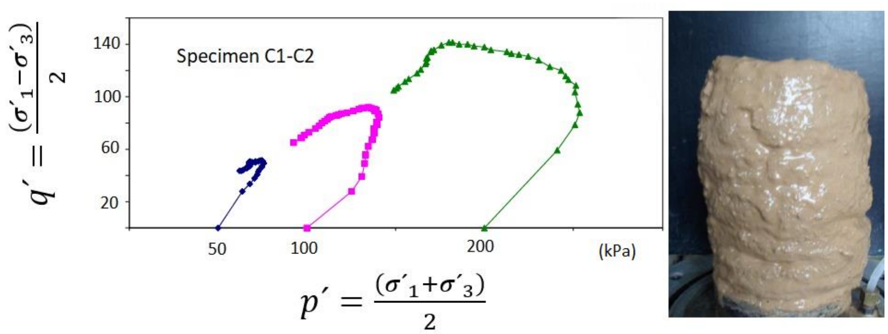

Figure 4. Example of triaxial test results from reconstituted specimens (e = 0.65 at reconstitution).

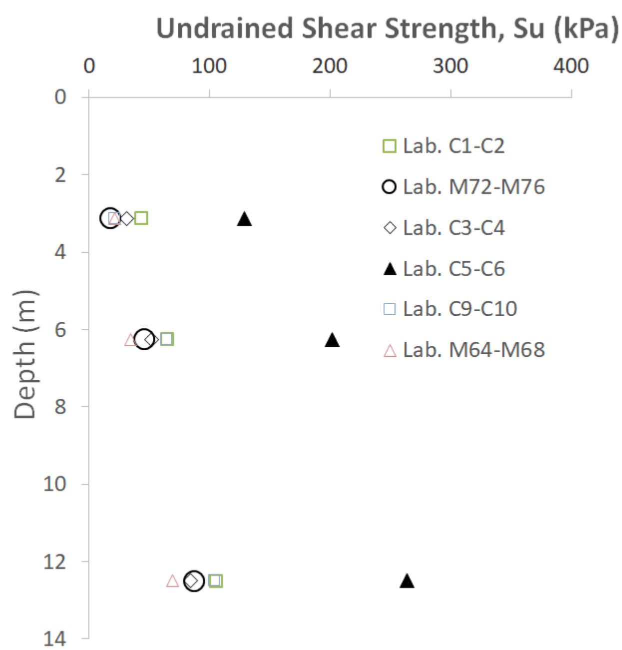

Figure 5. Deduced values of S_u from consolidated undrained triaxial tests.

CPT TEST RESULTS

CPT sounding reached depths between 5.7 m and 37.1 m until natural foundation soils were reached. Data was collected every 2 cm and analyzed to deduce the undrained strength of the soil. Readings from the CPT include cone resistance (q_t), sleeve friction (f_s), and excess pore water pressure during penetration (u_2). Dissipation tests were also carried out, and shear wave velocity (V_S) was obtained from a downhole test performed on CPTs.

The following equation is used to calculate the undrained shear strength:

$$S_u = \frac{q_t - \sigma_v}{N_{kt}} \tag{3}$$

Robertson and Cabal (2015) mention that N_{kt} varies from 10 to 18, but an average of 14 has been commonly used in practice. It is recommended that the value of N_{kt} be adjusted locally with additional laboratory or field testing. Figure 7 shows the undisturbed and remolded shear strength values derived from the average CPT sounding data shown in Figure 6. Figure 7 also shows the undrained shear strength value from a single in-situ vane shear. If this value were considered representative, a value of N_{kt} = 10 would better represent the undisturbed value of the material. However, as only one value of undisturbed shear strength is deduced from the shear vane test, a conservative value of N_{Kt} = 14 is used to derive the undisturbed strength. However, values of S_u deduced from these CPTs would be higher if values N_{kt} = 10 or 12 were used, as in Figures 1 and 2, respectively.

Remolded undrained shear strength, $S_{urem,}$ is frequently assumed to be equal to the sleeve friction stress, fs. The fs value can be very low in sensitive soils and should be used carefully. The CPT soundings performed and the average value obtained for tip, sleeve resistance, and pore pressure u_2 are shown in a representative zone where soil samples were collected from the stockpile (see Figure 6).

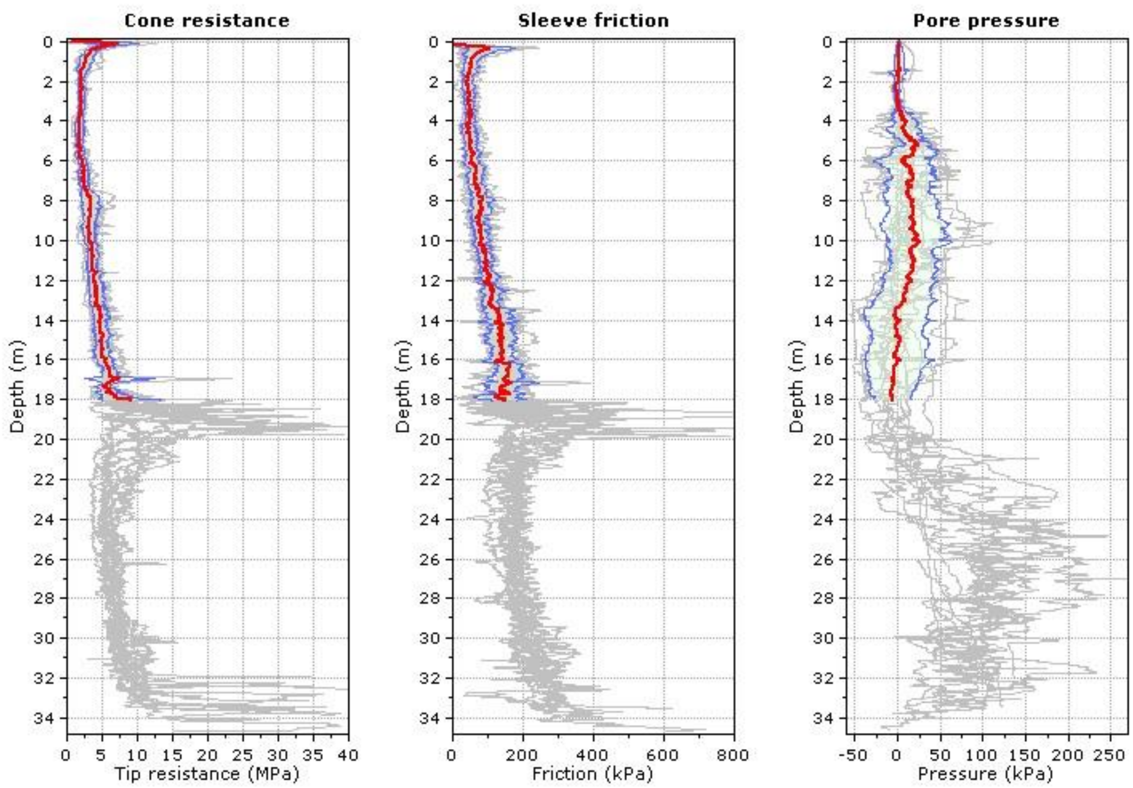

Figure 6. Average values of CPT soundings.

Using the average values of CPT data (in Figure 6) up to a depth of approximately 18 m, the undrained strength (S_u) and remolded undrained strength ($S_{u\text{-}rem}$) are obtained and plotted in Figure 7, where they are compared with the undrained strength from laboratory tests. There is a significant agreement between the S_u value obtained from triaxial tests on reconstituted specimens and the remolded strength derived from the CPTs. This agreement is expected since the reconstituted specimens are remolded entirely. However, it is essential to acknowledge that additional strength may be exhibited in undisturbed S_u values due to aging, chemical cementation, and other effects. Figure 7 also shows that the tested material's sensitivity is between 2 and 3. This range is similar to most clays (Wesley, 2010).

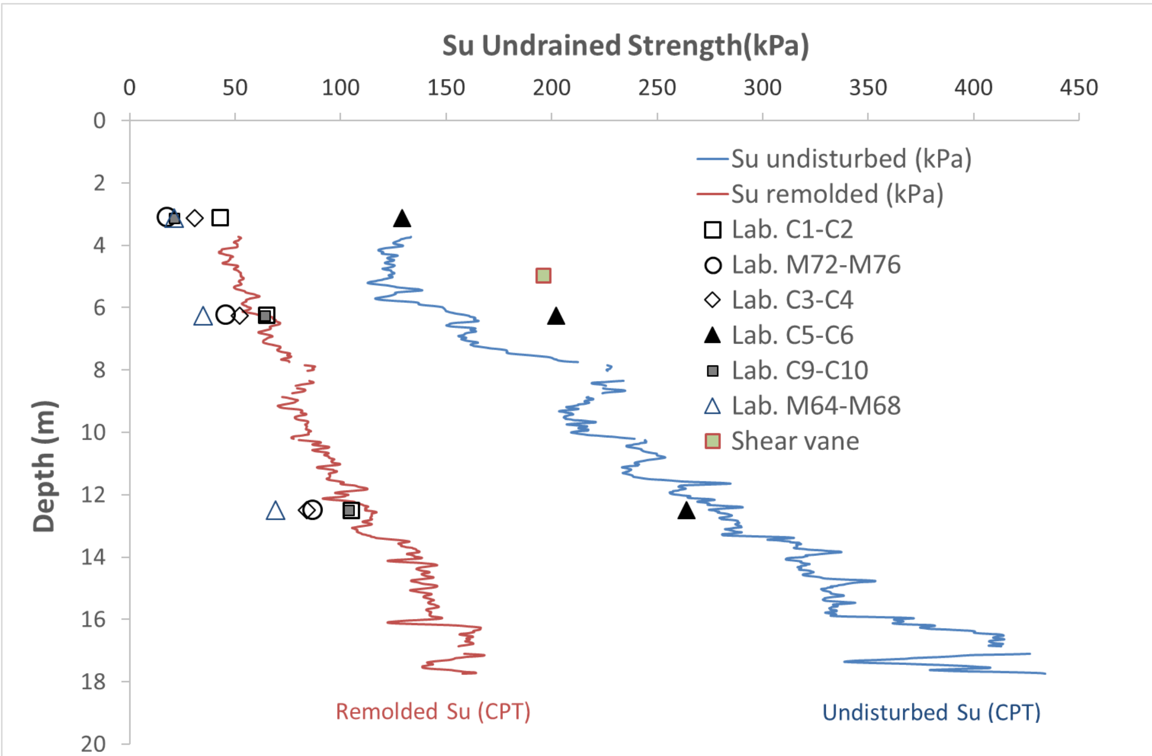

Figure 7. Comparison between S_u values obtained from CPT and laboratory triaxial tests.

DMT TEST RESULTS

Two DMT tests were performed alongside the two CPT soundings (CPT43 and CPT21). The distance between the CPTs and corresponding DMTs was close to 5 m to ensure similar material conditions and minimize the influence of the first test (CPT) on the second test (DMT). A and B pressure readings were taken at each 0.2 m depth intervals. A reading was corrected to p_0 (contact pressure) and B to p_1 (expansion pressure). Using these readings, the following DMT indices are calculated: soil material index $I_D = (p_1\text{-}p_0)/(p_0\text{-}u_0)$; dilatometer modulus $E_D = 34.7(p_1\text{-}p_0)$; and horizontal stress index: $K_D = (p_0\text{-}u_0)/\sigma'_{v0}$. Details on the interpretation of this test are provided by Marchetti (2015).

Equations for determining the undrained shear strength (S_u) have been proposed by different authors. Marchetti (1980) proposes a simple empirical relationship to predict the undrained shear strength of cohesive soils as follows:

$$\frac{S_u}{\sigma'_{v0}} = 0.22(0.5K_D)^{1.25} \quad (4)$$

This equation considers an estimation of the over-consolidation ratio (OCR) obtained from the value of K_D. Several authors show that this equation, in its original form, generally underpredicts the undrained strength. Mlynarek et al. (2018) show the results from eight investigation sites in Poland. They propose a general form to obtain S_u by taking the plasticity index of the material into account and using a modified K_{D1} parameter as follows:

$$K_{D1} = \frac{(P1 - \sigma_{v0})}{\sigma'_{v0}} \quad (5)$$

Mlynarek et al. (2018) find that for a group of soils with PI between 6 to 13% and cemented (Group 2), the equation for S_u is:

$$S_u = 45.86 - 5.31PI + 14.42(K_{D1}\sigma'_P)^{0.31} \quad (6)$$

Where σ'_p represents the pre-consolidation stress expressed in kPa and can be calculated based on field or laboratory tests. The material in this waste dump has a plasticity range between PI = 6 and 13%, and the CPT results show some cementation, so we use this equation (Eq. 5) to deduce the value of S_u from DMT tests.

By considering the identical values of effective stress at different depths when using equations based on cone penetration testing (CPT) and flat dilatometer testing (DMT) and utilizing the over-consolidation ratio (OCR) ratio determined from CPT tests, the undrained shear strength (S_u) values are deduced and compared. The comparison is made between the deduced undisturbed S_u values from DMT and those obtained from CPT tests at CPT21 and CPT43, along with the adjacent DMT test. Figure 8 illustrates the results of the undisturbed S_u values derived from the CPT21 data and the S_u values obtained from the DMT test using the equations proposed by Marchetti (1980) and Mlynarek et al. (2018).

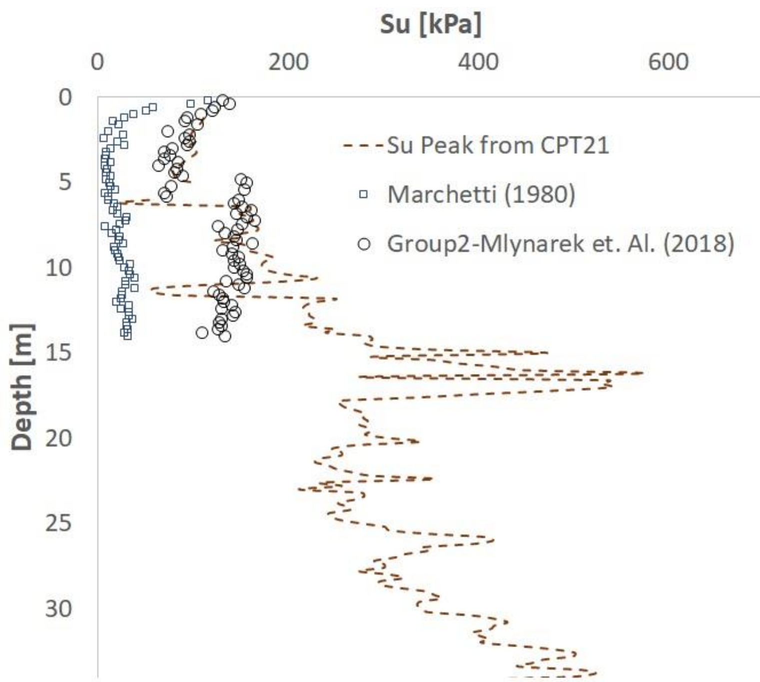

Figure 8. Comparison between S_u values obtained from CPT and DMT tests at CPT21.

Figure 8 shows that the deduced values of S_u obtained using Mlynarek et al. (2018) closely align with those acquired from the CPT tests. However, when employing Marchetti (1980), the deduced S_u values are consistently 6.5 times lower, on average, compared to those derived using Mlynarek et al. (2018). This trend was similarly observed in the in-situ testing results reported by Mlynarek et al. (2018).

The critical distinction between the S_u values deduced by Mlynarek et al. (2018) and Marchetti (1980) lies in the parameters used. Mlynarek et al. (2018) utilize p_1, a function of the B value obtained in DMT testing. On the other hand, Marchetti (1980) employs p_0, derived from the A and B readings from the DMT. In the DMT, reading A corresponds to the external pressure required to move the DMT membrane to a free-air position, while pressure B corresponds to a position that further mobilizes the strength of the soil.

Based on these observations, it is reasonable to conclude that using p_1, as done by Mlynarek et al. (2018), is more suitable for deducing the undrained shear strength. This is because p_1 represents a value where the soil strength is mobilized, whereas p_0 may not fully account for the mobilization of strength.

Similarly, Figure 9 displays the results from testing at the location of CPT43. It is evident in both places (CPTs 43 and 21) that utilizing the original correlation of Marchetti is excessively conservative. The deduced values do not even reach the remolded strength depicted in Figure 7, obtained from CPT or tested on undrained triaxial compression tests on reconstituted soil specimens. Conversely, the equation proposed by Mlynarek et al. (2018) yields results that closely align with those obtained using tip resistance to get undisturbed undrained strength on CPT soundings.

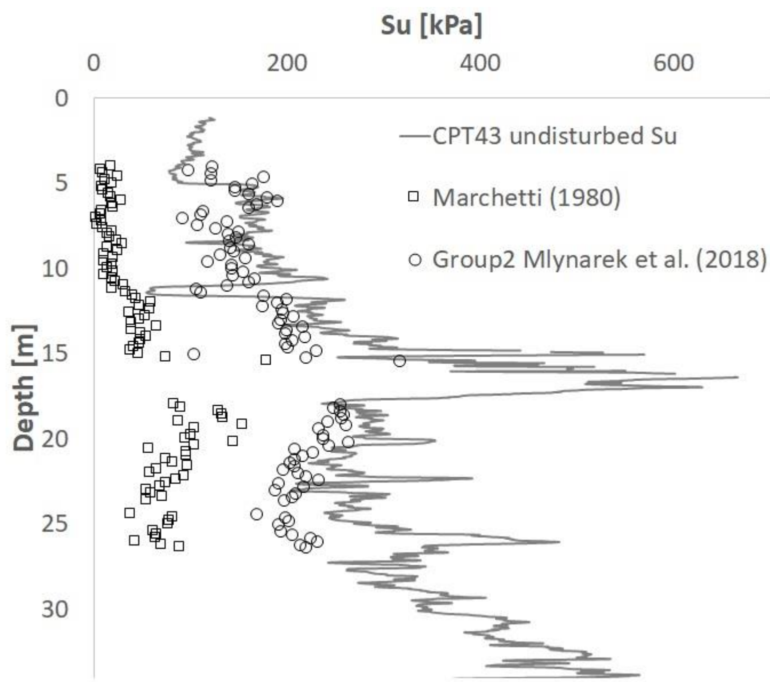

Figure 9. Comparison between S_u values obtained from CPT and DMNT tests at CPT43.

INFLUENCE OF PORE WATER PRESSURE ESTIMATION

It is widely recognized that the undrained strength of soils is directly influenced by the material properties and the effective stress of the soil at the time when S_u is evaluated. The effective stress is determined by the total stress, which is a function of the total unit weight of the soil and the pore water pressure present in the field during in-situ testing. Figure 10 illustrates the deduced pore pressure distribution when CPT and DMT are conducted at position CPT43. This is achieved by performing several dissipation tests on CPT soundings and installing vibrating wire piezometers at two depths in the exact location (CPT43). The data reveals a significant agreement between the pore water values measured through dissipation tests on CPT43 and those measured by the vibrating wire piezometers.

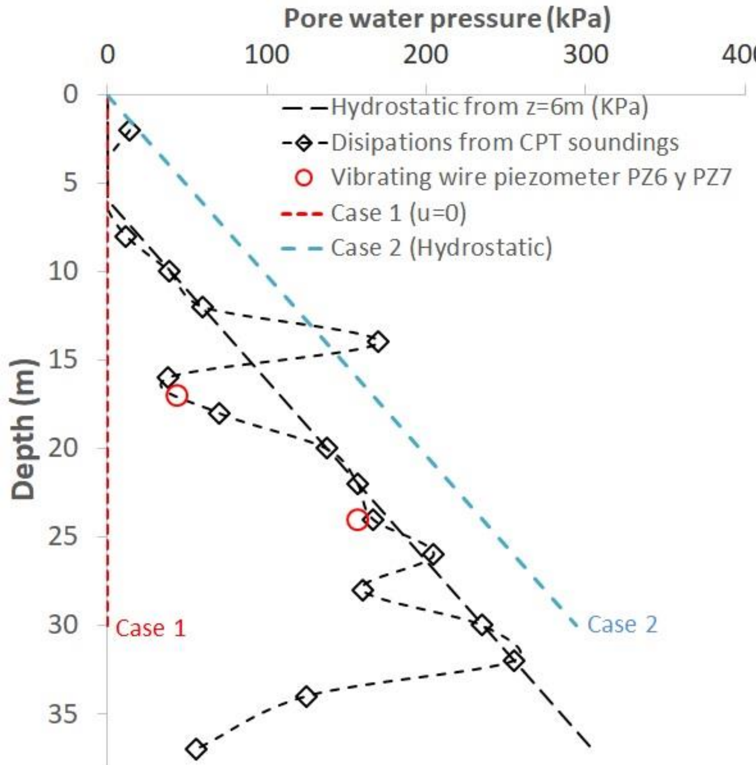

Figure 10. Measurements and analysis of the distribution of pore water pressure.

In Figure 10, the pore water pressure observations align with the anticipated pressure patterns that arise from the accumulation of saturated or nearly saturated material layers, leading to an increase in the height of the waste dump over time. At the top of the waste dump, faster dissipation of pore water pressure is observed due to the unrestricted escape of liquid to the surface, mainly due to evaporation.

Conversely, a drainage layer of native soil at an approximate depth of 35 m is evident, rapidly decreasing pore water pressure. This establishment of a double drainage condition within the waste dump indicates the existence of two drainage paths: one through the unrestricted escape at the top and another through the drainage layer at the bottom. These observations suggest the complex interplay of drainage mechanisms within the waste dump, with differential dissipation rates and a drainage layer contributing to pore water pressure patterns. Robertson et al. (2023) observe similar pore pressure profiles on tailings at the Candelaria mine.

As the soil consolidates, the pore water pressure will gradually decrease. To estimate the pore water pressure during testing, a reasonable approximation is to consider the hydrostatic pressure at a z = 6 m depth, based on CPT results, as indicated in Figure 10. The water pressure is lower at the bottom of the waste dump due to downward drainage seepage at that depth.

Figure 10 also includes two conceptual pore pressure distributions, Case 1 (u = 0) and Case 2 (hydrostatic pressure from the surface). These distributions will be utilized later to evaluate their conceptual significance about the deduced value of S_u/σ'_v (undrained shear strength over effective vertical stress) and the calculated safety factor for slope stability.

By considering these different pore pressure scenarios, a comprehensive assessment can be made regarding their impact on the deduced geotechnical parameters and the overall stability of the slope.

In Equation 2, total stress distribution can be calculated by inferring the total unit weight of the material and then determining the value of S_u from CPT soundings. Therefore, obtaining S_u from CPT data does not require a direct determination of the pore water distribution of the material in depth. Similarly, Equation 5 shows a similar aspect for DMT testing, particularly in normally consolidated soils where the pre-consolidation stress (σ'_p) equals the effective vertical stress (σ'_v). In this case, the undrained strength of the material is obtained directly from dilatometer measurements without the necessity to calculate or assume pore water pressure distribution or effective stress of the material profile in depth. This does not mean that the actual undrained strength does not depend on the effective stress of the soil and, therefore, the pore water pressure distribution, but this is captured by the in-situ tests (CPT or DMT) directly based on the measurements at the time the in-situ test was performed. CPT and DMT data changes in time due to alterations in pore water pressure and effective stress.

However, the situation is different when considering the coefficient S_u/σ'_v. To calculate this ratio, it is necessary to know the pore water pressure distribution, which, in turn, allows for determining the effective stress at each depth. This strength ratio is commonly used in design applications, such as the SHANSEP method employed in slope stability software, as shown by Coffman et al. (2010). Therefore, to calculate the safety factor of a slope, we may directly use the undrained strength of the material in its complete profile or obtain a correct value of S_u/σ'_v (knowing the actual pore water distribution).

Based on the S_u values shown in Figure 7, we can approximate the change in shear strength with depth as $S_u/z = 9$ (considering remolded strength). To illustrate the effect of different assumed pressure distributions, two simple pore water pressure distributions are considered:

a) Case 1: u = 0 (no pore water pressure)
b) Case 2: Hydrostatic pressure assumed from z = 0 (surface)

Considering these different pressure distribution scenarios allows us to assess their impact on the calculated shear strength and the subsequent implications for slope stability analysis.

In the first case, the total stress and effective stress have the same values since u = 0. When considering a unit weight value of 17 kN/m^3, it can be obtained that the ratio S_u/σ' is constant and equal to 0.53 for remolded undrained strength. For the second case, it is also easy to prove that the effective stress will be equal to $\sigma'_v = (17-9.8)*z$. Therefore, $S_u/z = 9$ would derive a value of S_u/σ'_v equal to 1.25, also constant with depth. Consequently, an important change in the ratio of S_u/σ' is derived from field measurements, depending on the assumed pore pressure with depth. For example, values of S_u/σ' of 0.5 have been found in gold mine projects, as shown by Dillon and Wardlaw (2010). The value of $S_u/\sigma_v' = 1.25$ is high and is only used here as an example to show its influence in obtaining the safety factor of the waste dump. Moreover, it will be subsequently demonstrated that the chosen value of the S_u/σ'_v ratio does not impact the calculated factor of safety as long as the numerical model employed for slope stability analysis incorporates the same assumed or measured value of pore water pressure and, thus, the same effective stress as utilized in deducing S_u/σ'_v.

Figure 11 portrays a modeled waste dump with a height of 60 m and a slope angle of 4%. This illustration assumes a constant pore pressure of u = 0 and a Su/σ'v ratio of 0.53. The factor of safety for slope stability is calculated using slide2 software and the Morgenstern-Price method.

As mentioned earlier, two cases of S_u/σ' ratio were derived based on the assumed pore water pressure. These cases are as follows:

$\frac{S_u}{\sigma'_v} = 0.53$ First Case A (u=0)

$\frac{S_u}{\sigma'_v} = 1.25$ Second Case B (hydrostatic from surface)

Both cases are implemented on the stability analysis using the SHANSEP model as:

$$\tau = A + \sigma_v' S(OCR)^m \tag{7}$$

In the mentioned context, the exponent "m" typically ranges between 0.75 and 1. The overconsolidation ratio (OCR) measures how much the soil has been consolidated in the past. To adopt a conservative approach, an OCR value of 1.0 is considered (assuming no significant past consolidation), and the remolded strength is utilized. S is the normalized undrained shear strength at the normally consolidated state $S = (S_u/\sigma_v')_{NC}$. The equation has a parameter "A" that represents the minimum strength. Based on the values observed from the in-situ and laboratory data, it is considered zero (A = 0) in this case.

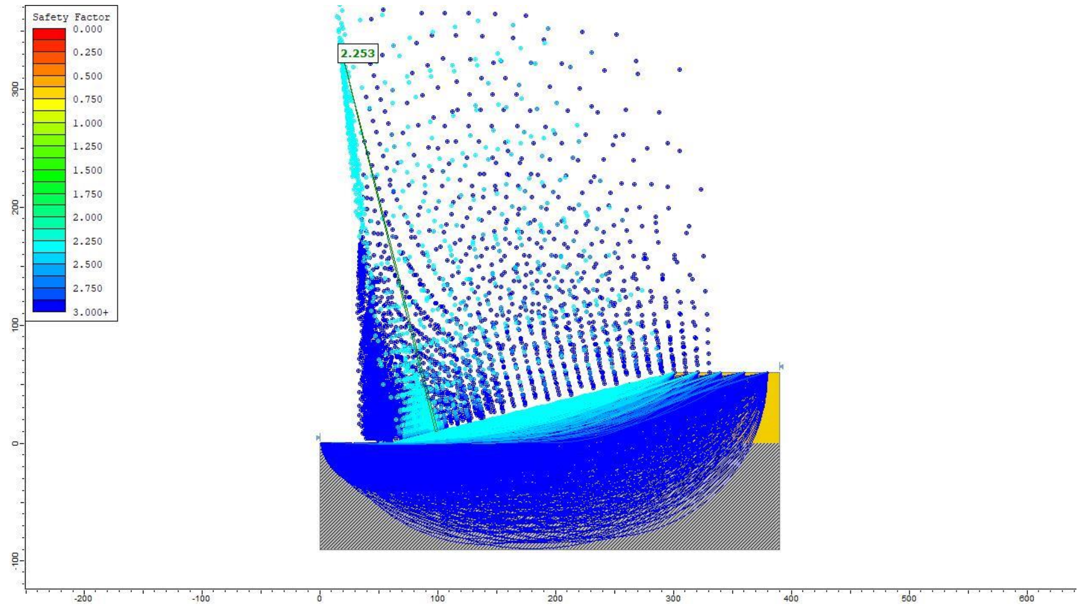

Figure 11. Slope stability Case 1 (u=0), and Case A, Su/σ´$_v$ =0.53.

Table 1. The slope stability safety factor is derived from numerical analysis by considering different pore water pressure distributions when deriving S_u/σ'_v and during numerical modeling. Natural soil is shown below with higher strength.

S_u/σ'_v	Water pressure on slope model	F.S.
0,53	u=0	2,25
1,25	u=0	5,31
0,53	hydrostatic	0,93
1,25	hydrostatic	2,25

Considering the value of $S_u/\sigma'_v = 0.53$ obtained by assuming u = 0 (Case 1), the resulting safety factor is S.F. = 2.25. Similarly, when using $S_u/\sigma'_v = 1.25$ (corresponding to the hydrostatic pressure case, Case 2), the safety factor remains S.F. = 2.25 when applying hydrostatic pressure on the numerical model.

However, for the other two cases (using $S_u/\sigma'_v = 0.53$ with hydrostatic pressure and $S_u/\sigma'_v = 1.25$ with zero pore pressure), the chosen S_u/σ'_v ratio does not match the corresponding pore water pressure distribution employed in the slope stability analysis. These cases yield different factors of safety values, indicating a discrepancy between the assumed ratio and the pore water pressure conditions used in the study.

Valid cases arise when the pressure distribution used in the slope stability analysis matches the water pressure distribution employed in deducing S_u/σ'_v. The resulting safety factor remains the same for cases where consistent pressure distributions are utilized (S.F. = 2.25, first and last case in Table 1). This consistency arises from correctly calculating the undrained shear strength S_u, as shown in Figure 12, representing the values measured on the field. Consequently, as is common practice, the undrained strength of the soil is measured directly on the field using a vane shear device, CPT soundings, or DMT testing. The results of these measurements depend on the current state of the effective stress of the tested material. However, these measured values of S_u do not require the measurement of the current effective stress or pore water pressure. The stability of the slope can be deduced directly using these values of undrained strength. If numerical models such as SHANSEP are used based on the ratio between S_u/σ'_v, then the same pore water pressure distribution must be used to deduct the ratio S_u/σ'_v and the slope stability assessment. Therefore, to assess the waste pile's safety factor, knowing the correct distribution of pore water pressure and effective stress is not required. However, to assess the change or improvement of the safety factor in time, the correct value of S_u/σ_v' must be obtained. This requires the correct deduction of the actual pore water pressure and the determination of how fast pore water pressure would dissipate on the field.

According to the evaluation, the global safety factor of 2.25 would represent the actual state of the waste pile. A low value of 4% of the average existing slope could be increased. Still, consideration of shallow slip movement material must be attended to, as the material in the border of the waste pile is under low effective stress, therefore has low undrained strength, and has been seen to cause shallow failure surfaces.

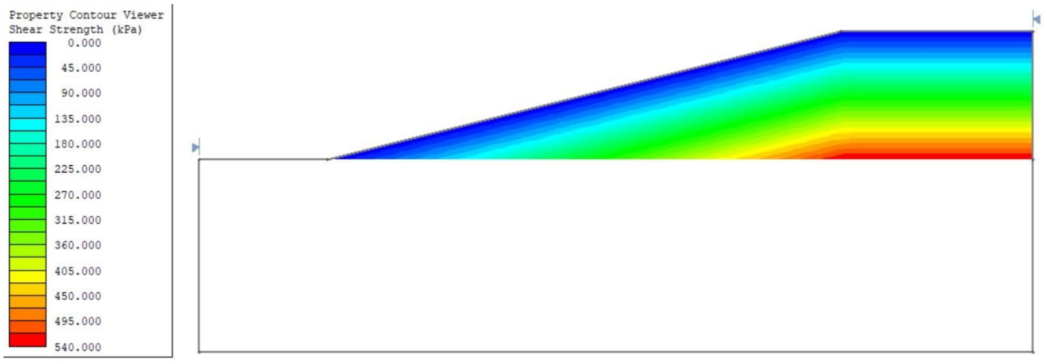

Figure 12. Deduced shear strength using SHANSEP.

As previously mentioned, the pore water pressure within the waste dump is expected to decrease gradually over time due to consolidation. In such a scenario, the effective stress within the dump increases. It is reasonable to anticipate that the undrained shear strength of the material will also increase over time. This increase can be measured through new in-situ testing or deduced by considering the consolidation process of the waste dump material over time. By monitoring the changes in pore water pressure and the corresponding effective stress, it becomes possible to assess the evolving undrained shear strength of the material.

CONCLUSION

Due to the mine's economic and safety implications, deducting undrained shear strength is crucial in various geotechnical design projects, especially in waste dump design optimization. These structures receive daily deposits of hundreds or thousands of tons of materials, and their design, which includes determining the maximum slope angle and height of the waste dump, heavily relies on undrained shear strength as a critical parameter.

It is well known that differences exist between undisturbed and remolded undrained strength, with a significant difference observed in sensitive soils. This disparity is considerable in construction and design considerations. In this study, differences between undisturbed and remolded undrained strength have been observed. Deduced sensitivity values vary between 1.5 and 3.5 in the tested material.

This study shows a strong agreement between the remolded shear strength derived from the peak strength value in undrained compression triaxial tests on reconstituted samples and the strength deduced from cone soundings using sleeve friction (fs). The undisturbed strength obtained from cone penetration testing (CPT) soundings is approximately 2 to 3 times the observed remolded strength, assuming a conservative value of $N_{kt} = 14$.

The original equation Marchetti (1980) proposed to deduce S_u from DMT tests appears overly conservative. Conversely, the equation proposed by Mlynarek et al. (2018), applicable to soils with a similar range of plasticity index, proves to be a good fit for deducing the undisturbed value of S_u from DMT data. Marchetti's proposed values are 6.5 times lower than those deduced using CPT data. Additional laboratory tests using undisturbed soil samples or field tests utilizing a vane shear device would be desirable to confirm the undisturbed strength on the field.

An essential finding of this study is the confirmation that utilizing the same pore water pressure for deducing the ratio S_u/σ'_v from cone penetration testing (CPT) soundings, as well as for the subsequent undrained slope stability analysis, leads to the same safety factor (in this case, 2.25), regardless of the pore pressure distribution considered. This highlights the consistency and reliability of employing the same pore water pressure distribution in both aspects of the analysis. Consequently, the information gathered from the field and laboratory investigations gives the designer ample data to carry out their task effectively.

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