Evaluation of the susceptibility to flow liquefaction of an iron ore tailings using the state parameter and Yield Stress Ratio approach

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ABSTRACT

Flow liquefaction is observed in saturated or nearly saturated geomaterials, showing a strain-softening response during undrained shear, primarily in very loose sands and silts, as well as in very sensitive clays. The Cone Penetration Test (CPTu) has been gaining popularity among the geotechnical community to evaluate the state of a soil profile due to its good repeatability, detailed information on the soil stratigraphy and extensive detailed scientific studies to guide the application of the test results. Susceptibility to flow liquefaction is typically evaluated by estimating the in-situ state parameter (ψ) since it directly correlates to the soil behaviour at large strain, as shown by Jefferies & Been (2016). Mayne & Sharp (2019) suggested using the yield-stress-ratio ($YSR = \sigma_p'/\sigma_{v0}'$) to estimate the soil state using a threshold of $YSR \approx 3$. Currently, in the Brazilian Mining Industry, it is common to find the application of different approaches to evaluating the susceptibility of mining tailings to flow liquefaction (e.g., Plewes et al., 1992; Olson, 2001; Shuttle & Cunning, 2008; Robertson, 2016). This paper presents an evaluation of the susceptibility of an iron ore tailings (IOTs) to flow liquefaction using the following recently published approaches: i) Mayne and Sharp (2019), using the yield stress ratio (YSR) approach; ii) Smith et al. (2021), using a generalised CPTu state parameter inversion method based on the NorSand Widget; and iii) Robertson (2022), with the updated *Kc*. The main results show a good convergence between the three methodologies used.

Keywords: flow liquefaction; iron ore tailings (IOTs); CPTu.

1. Introduction

Casagrande (1936) evaluated the behaviour of sands in loose and dense state through direct shear tests in drained conditions and verified that loose sands tend to contract while dense sands tend to dilate when sheared.

At large strains (above 10%), Casagrande (1975) observed that the materials presented the same void ratio, which the author defined as the critical state void ratio (e_c), considering the performance of drained tests with 100 kPa normal stress of three states conditions: i) loose sand; ii) dense sand; iii) sample with critical void ratio.

Taylor (1948) observed that the critical void ratio is affected by the mean effective stress (p'), becoming smaller with confining stress increase. The author evaluated the soil behaviour during the plastic phase and observed that the relationship between the final void ratio and the logarithm of the applied stress can be given as a straight line parallel and slightly inferior to the normal compression line (NCL), being classified as the ultimate condition of the material and named as the critical state line (CSL).

Castro (1969) undertook a series of stress-controlled triaxial tests in uniform, clean quartz sand (Banding Sand) to reproduce field loading conditions, which Casagrande (1936) surmised were stress controlled. The tests on loose samples resulted in liquefaction failures since they show brittle strain-softening under undrained shear, leading to a well-defined *steady state* condition.

Poulos (1981) formalized the definition of the *steady state* as “the state in which the mass is continuously deforming at constant volume, constant normal effective stress, constant shear stress and constant velocity”. Been at al. (1991) examined in detail the difference between the critical and the steady-state line and concluded that, for practical purposes, equivalence could be assumed.

The CSL on the $q - p'$ and $e - p'$ planes can be represented by Eq. (1) and Eq. (2), respectively.

$$q = Mp' \quad (1)$$

$$e_c = \Gamma - \lambda ln(p_c') \quad (2)$$

Where q is the deviator stress, M is the critical friction ratio, Γ is the void ratio on the CSL at 1 kPa, λ is the slope of the CSL, and p_c' is the critical mean effective stress.

Robertson (2010) explains that the liquefaction phenomenon is associated with abrupt strength losses of the soil due to its metastable structure. Moreover, Robertson (2017) shows that most failures due to liquefaction occur in young, low plastic or non-plastic, loose granular soils without cementation that show brittle behaviour with significant strength loss for low strain rates during undrained shear.

Recent tailings dams' failures in Brazil, namely Brumadinho B-I Dam (2019) and Fundão Dam (2015), highlighted the importance of assessing the susceptibility to flow liquefaction, especially for structures constructed with hydraulically deposited sand-like materials. Many field and laboratory procedures can be used to evaluate the flow liquefaction susceptibility, such as i) grain-size distribution curves, ii) atterberg limits, iii) CPTu tests, iv) vane shear test, and v) triaxial compression test.

Due to its good repeatability and detailed soil stratigraphy information, this paper evaluates the susceptibility to flow liquefaction of an iron ore tailings (IOTs) using the CPTu test. Additionally, it presents the characterization of the IOTs deposited in a Brazilian tailings dam.

2. Susceptibility do Flow Liquefaction

In the Brazilian Mining Industry, it is usual to find studies assessing the flow liquefaction with methodologies proposed by Plewes et al. (1992), Olson (2001), Shuttle & Cunning (2008), and Robertson (2016) (dos Santos Junior et al., 2022a, 2022b; Faria et al., 2023). In this context, the following recently published approaches were used in this study:

- The yield stress ratio (YSR) approach (Mayne and Sharp 2019).
- The generalised CPTu state parameter inversion method based on the NorSand Widget (Smith et al., 2021).
- The updated *Kc* to evaluate the in-situ state parameter (Robertson, 2022).

The methodology of Plewes et al. (1992) is also presented since it is complementary to the one proposed by Smith et al. (2021).

Notably, these methodologies are initial screening tools to evaluate the phenomenon, and more is needed to determine whether the evaluated tailings present brittle strain-softening behaviour under undrained shearing. As Robertson (2017) described, "(…) not all contractive soils are strain-softening, and not all soils that are strain-softening have high brittleness".

2.1. State Parameter Definition

A possible reference to be used to define the state of sandy materials is the distance between the current void ratio of the material (e) and the critical state void ratio (Been & Jefferies, 1985). Therefore, this distance, or state path, indicates a direct representation of the tendency for volumetric variation of the soil under shear, which defines the state parameter (ψ), according to Eq. (3).

$$\psi = e - e_c \tag{3}$$

Been & Jefferies (1985) defined that using the state parameter to evaluate the material behaviour is fundamental since assessing the expected behaviour based on parameters such as void ratio and relative density does not provide the necessary rigour. Jefferies & Been (2016) showed that samples with the same relative density showed widely different stress paths depending on the stress levels. In contrast, samples at the same ψ but different densities and stress levels showed similar behaviour.

Experimental studies performed by Jefferies & Been (2016) suggested that materials with $\psi > -0.05$ tend to contract (state looser than the critical state), whereas $\psi < -0.05$ indicates a dilative behaviour (state denser than the critical state).

2.2. Plewes et al. (1992)

Plewes et al. (1992) suggested a relationship between the slope of the critical state line (λ_{10}) and the normalized friction ratio (F or Fr), as indicated by Eq. (4) and Eq. (5).

$$\lambda_{10} = \frac{F_r}{10} \tag{4}$$

$$F_r = \frac{f_s}{(q_t - \sigma_{v0})} \times 100\% \tag{5}$$

Where f_s is the sleeve friction resistance, q_t is the corrected cone tip resistance; and σ_{v0} is the total vertical stress.

Once λ_{10} is defined, the state parameter can be calculated using the equation proposed by Shuttle & Cunning (2007), as shown in Eq. (6).

$$Q_p\left(1 - B_q\right) + 1 = k' e^{(-m'\psi)} \tag{6}$$

Where Q_p is the cone tip resistance normalized by the mean effective stress (p_0') and B_q is the porewater pressure ratio, defined as the Eq. (7) and Eq. (8), respectively.

$$Q_p = \frac{(q_t - p_0)}{p_0'} \tag{7}$$

$$B_q = \frac{(u_2 - u_0)}{(q_t - \sigma_{v0})} \tag{8}$$

Where u_2 is the porewater pressure measured behind the cone and u_0 is the in-situ porewater pressure.

Plewes et al. (1992) use Eq. (9) and Eq. (10) to estimate the effective inversion coefficients k' and m', which are a function of the slope of the CSL.

$$\frac{k'}{M} = 3 + \frac{0{,}85}{\lambda_{10}} \tag{9}$$

$$m' = 11.9 - 13.3(\lambda_{10}) \tag{10}$$

2.3. Mayne & Sharp (2019)

Mayne et al. (2023) indicate that the pre-consolidation stress can be presented in dimensionless terms using a normalized form called the overconsolidation ratio (OCR), or equivalent yield stress ratio (YSR), which is defined by Eq. (11). According to them, the term YSR is becoming more prevalent because the traditional OCR is associated to mechanic unloading effects, while other geologic and environmental changes can also cause an apparent preconsolidation (i.e., desiccation, compaction, cyclic loading, repeated wetting-drying and other effects).

$$OCR = \frac{\sigma_p'}{\sigma_{v0}'} = YSR \tag{11}$$

In this context, Mayne et al. (2009) proposed that the yield stress (σ'_p) of soils can be evaluated from cone net resistance (q_{net}), whereas Mayne (2017) indicates that the exponent m' is a function of CPT material index (I_c), according to Eq. (12) and Eq. (13), respectively.

$$\sigma'_p = 0.33(q_{net})^{m'}(\frac{\sigma_{atm}}{100})^{1-m'} \quad (12)$$

$$m' = 1 - \frac{0.28}{1+\left(\frac{I_c}{2.65}\right)^{25}} \quad (13)$$

Where σ_{atm} is the atmospheric pressure ($\sigma_{atm} = 100\ kPa$).

As the threshold defined by the state parameter (ψ = -0.05) delimits the contractive-dilative behaviour (Jefferies & Been, 2016), alternatively, a critical threshold is defined in terms of yield stress ratio. Mayne (2017) defines that the YSR_{CSL} becomes the boundary separating contractive and dilative soil behaviour, according to Eq. (14) and Eq. (15).

$$YSR_{CSL} = \left(\frac{2}{\cos\phi'}\right)^{\frac{1}{\Lambda}} \quad (14)$$

$$\Lambda = 1 - \frac{k}{\lambda} = 1 - \frac{C_s}{C_c} \quad (15)$$

Where ϕ' is the critical friction angle, k is the recompression/expansion line slope, and λ is the CSL slope on the $e - p'$ plane. Given the absence of oedometer tests for the present paper, λ_{10} was defined according to Plewes et al. (1992) and converted to the natural logarithm base. Additionally, the correlation $k = 0.1\lambda$ was assumed.

Considering the purposes of calculating the yield stress ratio at the critical state, Eq. (16) and Eq. (17) proposed by Mayne (2007) were used to estimate ϕ'.

When $I_c < 2.6$:

$$\phi' = 17.6° + 11.0° \log(Q_{tn}) \quad (16)$$

When $I_c > 2.6$:

$$\phi' = 29.5° B_q^{\ 0.121}\left[0.256 + 0.336 B_q + \log(Q_{tn})\right] \quad (17)$$

Where Q_{tn} in the normalized tip resistance.

Mayne & Sharp (2019) indicated that considering a range of friction angles of sands between 30° ≤ ϕ' ≤ 45º and a representative value $\Lambda = 0.8$, the corresponding range of YSR_{CSL} is 2 < YSR_{CSL} < 4, with a typical value $YSR_{CSL} \approx 3$.

2.4. Smith et al. (2021)

Smith et al. (2021) presented a generalised method to estimate ψ from either drained or undrained CPTu test, based upon CPTu calibrations modelled using the Widget for a wide range of NorSand calibrated soils presented in the literature.

Considering the CSSM in its formulation, the NorSand model was developed during the 1980s and 1990s, based on the experience with the construction of structures on loose sands. The analysis of the occurrence of static liquefaction during the construction of these structures contributed to the development of the model, initially proposed by Jefferies (1993) and later updated by Jefferies & Been (2016).

As the names suggest, the intent of the model is to simulate the behaviour of loose and dense sands, in both drained and undrained conditions. In this aspect, the NorSand model was the first model to consider the state parameter within the CCSM approach.

Using the NorSand model and the cavity expansion modelling, Smith et al. (2021) proposed a correction for the k' and m' values proposed by Plewes et al. (1992).

Adopting the CPTu inversion parameter framework, Smith et al. (2021) indicated that some screening level methods suffer from stress level bias, which can result in overestimation of ψ with increasing depth. In this context, according to the authors, such bias can be removed through normalization by the elastic soil rigidity index $I_{r,e}$, as well as the use of drainage condition-specific functions, as indicated by Eq. (18) to Eq. (25).

Drained penetration:

$$\frac{k'}{M_{tc}} = a + b\ ln(\frac{1}{\lambda_{10}}) \quad (18)$$

$$a = 13.58 - 0.52 \ln(I_{r,e}) \quad (19)$$

$$b = 0.87 \ln(I_{r,e}) - 0.19 \quad (20)$$

$$m' = 3.83(\lambda_{10})^{-0.31} \quad (21)$$

Undrained penetration:

$$\frac{k'}{M_{tc}} = c + \frac{d}{\lambda_{10}} \quad (22)$$

$$c = 7.36 - 4.61 G_0 \quad (23)$$

$$d = 0.06 + 0.02 G_0 \quad (24)$$

$$m' = \frac{\ln(10)}{\lambda_{10}} \quad (25)$$

Where M_{tc} was assumed in the present paper as the mean value of 1.45, as indicated by Jefferies & Been (2016), given the absence of tests to define the CSL.

It is noteworthy mention that the proposed method does not provide inversion equations for partially drained CPTu. Smith et al. (2021) suggest using drained inversion equations if $|B_q| < 0.02$ and $|\Delta u| < 20$ kPa. For the undrained inversion equations, the authors highlight that it is essential to consider the dissipation test, and t_{50} must be higher than 60 s. If t_{50} < 60 s, an alternative screening level method must be used (e.g. Plewes et al., 1992). Additionally, the method does not apply to soils with atypical soil fabric or bonding.

In terms of recorded CPTu data, the proposed method relies upon the accuracy of the $F_r - \lambda$ correlation proposed by Plewes et al. (1992). Therefore, it also depends upon accurate measurement of f_s. McConnel & Wassenaar (2022) indicate that the F_r values can be affected by the precision of the CPTu to measure the sleeve friction in very soft geomaterials, and special attention is required when evaluating data from CPTu performed in hydraulically disposed tailings (e.g. iron ore tailings, IOTs).

2.5. Robertson (2022)

To estimate ψ from CPTu tests, different methods are available. In sand-like soils, with $I_c < 2.6$, Robertson (2010) suggested the normalized cone resistance (Q_{tn})

can be associated with the state parameter using a clean sand equivalent normalized cone resistance ($Q_{tn,cs}$), according to Eq. (26).

$$Q_{tn,cs} = Q_{tn} K_c \tag{26}$$

Where K_c is a correction factor to account for changing behaviour with increasing fines content. In this context, Roberston (2010) suggested a link between $Q_{tn,cs}$ and ψ for sand-like soils, as indicated by Eq. (27).

$$\psi = 0.56 - 0.133 \, log \, (Q_{tn,cs}) \tag{27}$$

To account for partial drainage, Robertson (2022) indicated an update in the equation to calculate K_c suggested by Robertson & Wride (1998), as shown in simplified version by Eq. (28) for $I_c \leq 3.0$.

$$K_c \approx 15 - \frac{14}{1+\left(\frac{I_c}{2.95}\right)^{11}} \tag{28}$$

Finally, Robertson (2022) indicates that the modified K_c relationship should not be extended beyond $I_c = 3.0$, where undrained penetration occurs.

3. Material Characterization

3.1. Geotechnical Characterization

The geotechnical characterization of the iron ore tailings was composed of i) the grain size distribution curve, ii) the water content, iii) the specific gravity of soils solids (G_s), iv) the liquid limit (LL), and v) the plastic limit (PL). The samples collected to perform the tests and the SCPTu analysed are indicated in Fig. 1 in yellow and green, respectively.

Figure 1. Location of the SCPTu and samples collected.

The grain size distribution curves (ASTM D422-63) obtained from disturbed samples collected in depth near the SCPTu are presented in Fig. 2. Between 0.0 - 17.0 m deep, the average tailings composition is 17% sand, 75.9% silt and 7.1% clay. Between 17.0 – 18.0 m deep the composition is 4.9% sand, 42.1% silt and 53.0% clay, showing a distinctive composition.

The atterberg limits were also evaluated according to ABNT NBR 6459 and ABNT NBR 7180. The samples collected at 0 – 17.0 m were non-plastic, and the sample at 17.0 – 18.0 m presented a Liquid Limit of 24% and a Plastic Limit of 16% (Plasticity Index of 8%), as shown in Fig. 3.

According to ABNT NBR 6457, the tailings show an average natural moisture content of 14.5%, varying from 7.0% at the surface to 18.5% at 17.0 m. Additionally, the average G_s is 3.7.

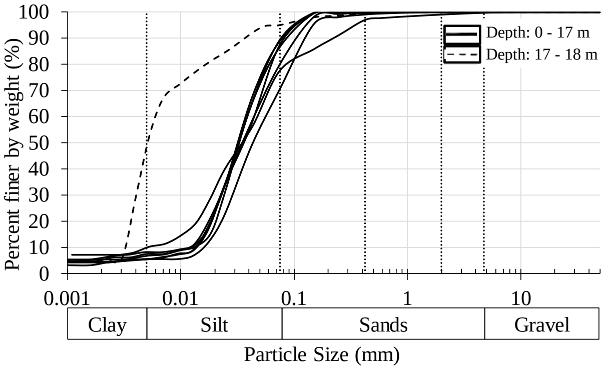

Figure 2. Grain size distribution curves (ASTM D422).

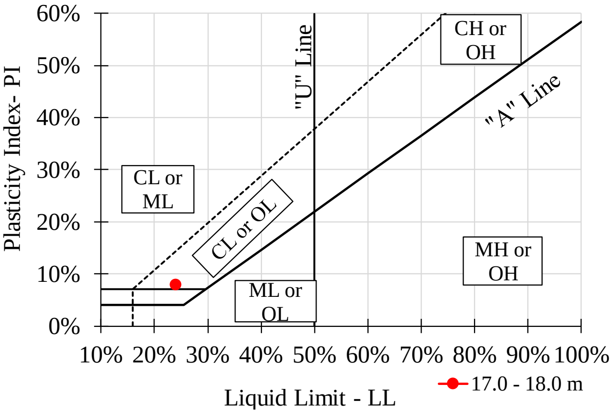

Figure 3. Casagrande Plasticity Chart.

Using the grain size distribution curves and the atterberg limits the samples were classified according to the Unified Soil Classification System (USCS – ASTM D2487), as show in Table 1.

Table 1. Classification of the tailings following USCS.

Depth (m)	USCS Classification
0.0 – 1.0	Silt (ML)
2.0 – 3.0	Silt (ML)
3.0 – 4.0	Silt (ML)
6.0 – 7.0	Silt (ML)
7.5 – 8.5	Silt with sand (ML)
9.0 – 10.0	Silt with sand (ML)
16.0 – 17.0	Silt with sand (ML)
17.0 – 18.0	Lean clay (CL)

3.2. SCPTu Test

An SCPTu was performed in the iron ore tailings with a total depth of 18.70 m. Fig. 4 shows the basic SCPTu parameters, whereas Fig. 5 shows the normalized parameters. Based on the SCPTu results, five regions were defined in depth with clear q_t, f_s, and u_2 variation patterns. The dissipation tests showed a hydrostatic equilibrium porewater pressure (u_0), while the shear wave velocity (V_s) increased from a depth of 6.0 m. The value of $V_s = 341 \text{ m/s}$ at 3.0 m can be explained by the compaction resulting from the construction of a cover layer during the de-characterization process of the dam.

The normalized parameters confirmed the initial classification. Special attention is given to B_q, which showed that the material has a predominant drained behaviour during cone penetration since it does not generate considerable excess porewater pressure. B_q did not reach 0.30 as suggested by Schnaid (2009) for undrained behaviour.

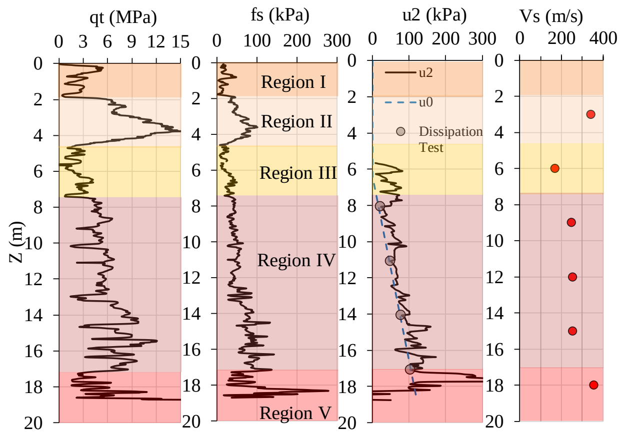

Figure 4. Basic SCPTu parameters.

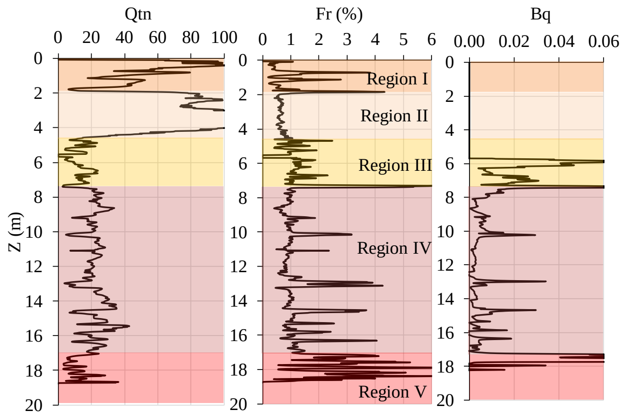

Figure 5. Normalized SCPTu parameters.

Analysing Fig. 6, based on the soil behaviour type indexes and the fines content (FC), as well as the clay-size fraction, the five regions can be described as:

- Region I (0 – 1.8 m): composed predominantly of transitional behaviour, with $2.05 < I_{c,R\&W} < 2.6$ and $I_B > 22$. The clay-size friction was less than 10% and the silty fraction was around 81.9%.
- Region II (1.8 – 4.6 m): presented a sand-like tailings, with the evidence predominance of points plotted for $I_{c,R\&W} < 2.05$ and $I_B > 32$. On average, the silty fraction was 79.5%, whereas the sandy fraction was 12.9%.
- Region III (4.6 – 7.3 m): with points predominantly plotted for $2.6 < I_{c,R\&W} < 2.95$ and $22 < I_B < 32$, this region presented a behaviour of silt mixtures (clayey silt to silty clay). The clay-size friction was less than 10% and the silty fraction was 83.7%. It is noteworthy mention that for this region the disturbed sample was collected near to Regio IV, which presented less fines content.
- Region IV (7.3 – 17.2 m): considering the majority classification of $2.05 < I_{c,R\&W} < 2.6$ and $I_B > 32$, this region presented sand-like tailings. On average, the silty fraction was 68.9% and the sandy fraction was 23.9%.
- Region V (17.2 – 18.7 m): composed predominantly of clay-like behaviour, with $I_{c,R\&W} > 2.95$ and $I_B < 22$ in many points. The clay-size friction was 53% and the silty fraction was 42.1%, confirming the classification indicated by the SCPTu. This region presented the highest clay-size fraction in the profile.

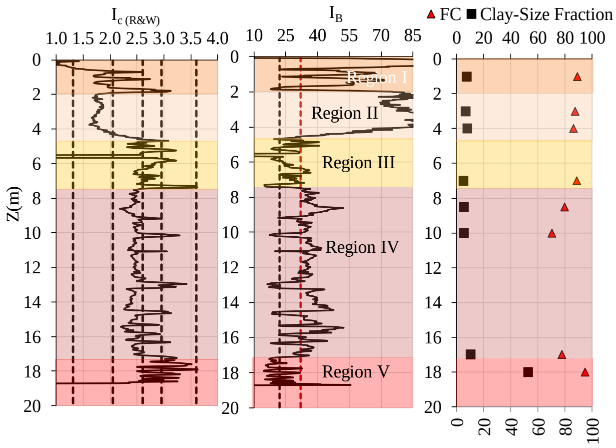

Figure 6. Soil behaviour type suggested by Robertson and Wride (1998), modified soil behaviour type index (I_B) suggested by Robertson (2016), and grain size characteristics.

Fig. 7 shows the results of the four dissipation tests (DPP) performed in depth. As can be seen, all tests resulted in a t_{50} < *40 s*, which indicates a drained penetration condition, since the values did not reach the interval of t_{50} > *60 s* as suggested by DeJong & Randolph (2012) for undrained behaviour. It is noteworthy mention that the dissipation tests had a total time of approximately 1.000 s and the values up to 100 s indicated in Fig. 7 represent only a part of the test to illustrate the correct range defined for the t_{50} values.

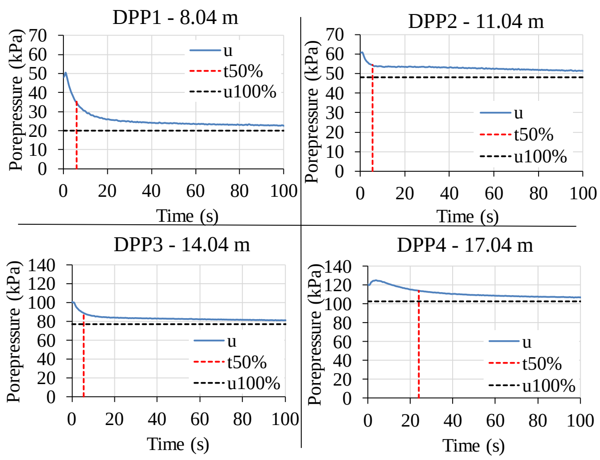

Figure 7. Dissipation tests performed in depth.

3.3. Microstructure Evaluation

The evaluation of microstructure in the iron ore tailings was performed using the shear wave velocity from the SCPTu (performed at elevation 3.0 m, 6.0 m, 9.0 m, 12.0 m, 15.0 m and 18.0 m.) and the methodology suggested by Robertson (2016).

According to Fig. 8, the results obtained showed that the tailings classify as an ideal material (no microstructure) for most of the evaluations, with only two elevations in the structure region (3.0 m and 18.0 m).

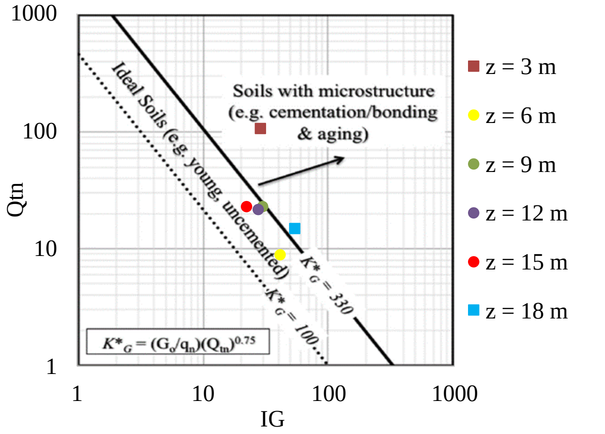

Figure 8. Microstructure evaluation (Roberston, 2016).

4. Results - Susceptibility to Flow Liquefaction

The evaluation of the susceptibility to flow liquefaction of the tailings using the screening level methods of Mayne & Sharp (2019), Smith et al. (2021) and Robertson (2022) are shown in Fig. 9.

The method proposed by Plewes et al. (1992) was applied to complement the evaluation of Smith et al. (2021) since the only region with predominantly $|B_q| < 0.02$ to apply the drained inversion equations was the Region IV (Fig. 5). The other regions presented higher values than 0.02, but $t_{50} < 60\ s$ (Fig. 7), which does not allow the application of the undrained inversion equations.

Analysing Fig. 9 and considering the three methodologies, Regions I and II presented a predominantly dilative behaviour in the partially saturated tailings. This fact can be related to compaction resulting from the construction of a cover layer during the de-characterization process of the dam.

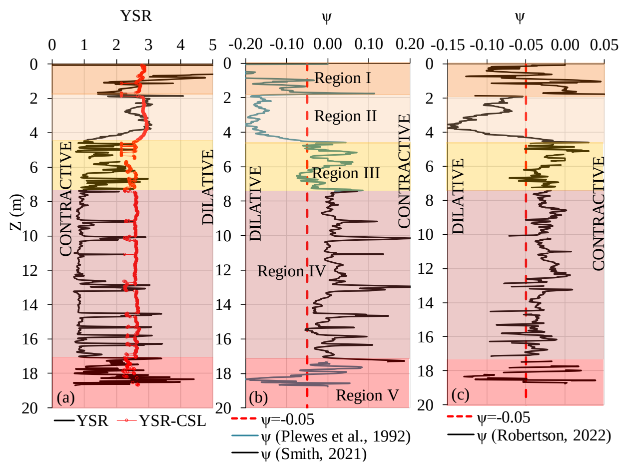

Figure 9. Evaluation of flow susceptibility based on (a) Mayne & Sharp (2019), (b) Smith (2021) and Plewes et al. (1992), and (c) Robertson (2022).

Regions III and IV presented a predominantly contractive behaviour according to all methodologies. The yield stress ratio (YSR) used for Mayne & Sharp (2019) showed many layers where YSR < 1, which is indicative of under-consolidated material. Considering the state parameter evaluation, according to Robertson (2022) there are more interbedded tailings with thin layers of dilative behaviour than the methodologies proposed by Plewes et al. (1992) and Smith et al. (2021).

Region V is composed of highly interbedded iron ore tailings, with layers of contractive and dilative behaviour according to all methodologies. These findings show the general convergence between the methods for the five regions.

5. Conclusion

This paper presented a case study of the susceptibility of iron ore tailings (IOTs) to flow liquefaction according to the screening level methods of i) Mayne & Sharp (2019), ii) Smith et al. (2021) and Plewes et al. (1992), and iii) Robertson (2022).

The geotechnical characterization of the IOTs was composed of i) the grain size distribution curve, ii) the water content, iii) the specific gravity of soils solids (G_s), iv) the liquid limit (LL), and v) the plastic limit (PL). After the unified soil classification (ASTM D2487), the IOTs were predominantly classified as non-plastic silts (group symbol ML).

An SCPTu with four dissipation tests was performed inside the TSF to evaluate the susceptibility of the tailings to flow liquefaction. Five distinct regions were identified in depth based on the behaviour characteristics and the geotechnical characterization. A predominant drained behaviour during cone penetration was observed, and the results from the shear wave velocity indicated that, in general, the IOTs were classified as ideal materials (no microstructure).

The evaluation of the susceptibility to flow liquefaction indicated good convergence between the three methodologies used. Regions I and II presented a predominantly dilative behaviour in the partially saturated tailings. Regions III and IV showed a predominantly contractive behaviour (more prone to be susceptible to liquefaction), with the methodology proposed by Robertson (2022) indicating more interbedded tailings with thin layers of dilative behaviour than the methodologies proposed by Plewes et al. (1992) and Smith et al. (2021). Finally, in Region V, highly interbedded iron ore tailings with layers of contractive and dilative behaviour were identified.

Considering the use of drainage condition-specific functions, the difficulty in applying the methodology proposed by Smith et al. (2021) was observed since the only region with predominantly $|B_q| < 0.02$ to use the drained inversion equations was Region IV and the other regions did not present $t_{50} > 60\ s$ to apply the undrained inversion equations. Based on this, it was necessary to use a complementary methodology (e.g. Plewes et al., 1992).

It is noteworthy to mention that the methodologies used in this paper are initial screening tools to evaluate the susceptibility to flow liquefaction. They only help to identify if the soil is contractive or dilative, and they alone are not enough to determine whether the tailings under evaluation present strain-softening and brittle behaviour in undrained shearing. Finally, for this study, triaxial and oedometer tests were not available to better characterize the iron ore tailings, which demonstrates the reality of conceptual projects in Brazil, in which basic characterization and CPTus tests are usually performed to evaluate the tailings behaviour.

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