Cone penetration test data interpretation for layered tailings storage impoundments with perched phreatic surfaces

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ABSTRACT

Cone penetration testing (CPT) is a widely used technique for soil characterisation. The introduction of the Global Industry Standard for Tailings Management (GISTM) in 2019 increased the necessity for understanding tailings properties (particularly shear strength), requiring better risk management and monitoring of tailings storage facilities (TSFs) to achieve zero fatalities. The complexity of tailings materials, influenced by ore processing, slurry deposition and water management techniques, causes greatly different geotechnical characteristics compared to other soils, requiring specialised monitoring equipment and in-depth investigation. One main aspect in interpretation of tailings properties is understanding the pore water pressure (PWP) within the deposited tailings layers which has often been overlooked in engineering studies. For example, in cases of interbedded layers of silt and sand, the downward drainage due to underdrain systems and lateral flow due to differences in lateral and vertical hydraulic conductivities can lead to the formation of distinct 'perched' phreatic surfaces in-between layers due to dissimilarity in material permeabilities. The use of commonly assumed linear PWP profiles in such cases can result in misinterpretations, hence, overestimation of tailings' effective strengths. In these instances, instruments like vibrating wire piezometers (VWPs) have been widely used. However, VWPs provide limited data for understanding the true PWP profile. CPT with PWP measurements (CPTu) has therefore become a practicable technique for identifying internal PWP and distinguishing deposited layers. As such, a detailed approach to CPT data interpretation is required for interlayered tailings facilities with perched phreatic surfaces. This paper presents how CPTu techniques, combined with graphical interpretation, and VWP data can be used to understand complex internal profiles of interlayered TSFs.

Keywords: interlayered TSFs; CPT interpretations; PWP; VWP; shear strengths; geotechnical characterisations

1. Introduction

Cone penetration testing (CPT) has become a prime tool for geotechnical understanding of underlying strata and strength properties of soils. Introduced in the 1930s as a manual apparatus to primarily measure the push resistance, modern rigs have expanded to accommodate a variety of sensors, sampling, and in-situ testing capabilities (Robertson and Cabal 2015). One of the most commonly used sensors is the piezometer adjacent to a filter behind the cone tip which measures pore water pressure (PWP). Such 'piezocones' were developed in the 1970s and have come to be known as CPTu probes and are very common in site investigations of mine waste (tailings) storage facilities where PWP governs much of the strength of fine clay or silt-like material.

In common applications where the stratigraphy is comprised of materials with similar permeabilities, and the depth of interest is relatively shallow, the mechanism by which PWP affects soil layers can be simplified to a linear profile, where the PWP increases at a constant rate with depth. The most common engineering approach dictates that the phreatic surface is located at the onset of PWP measured by the CPT (Johns and Murray 2018). However, mechanisms like capillary rise of several metres can lead to misinterpretations of the phreatic surface in CPT (Johns and Murray 2018, Holt and Kovacs 1981). More complex PWP regimes can arise in cases where materials with dissimilar permeabilities interface, as granular layers such as sand may exhibit free-draining properties with less capillary rise than cohesive materials like silt which retain PWP. When interbedded in layers, a very complex phreatic profile can form depending on the level of saturation, which might result in several 'perched' phreatic surfaces acting on different layers of the stratigraphy. Tailings storage facilities (TSFs) can often possess such strata due to many reasons, whereby the fines formed during ore processing operations, which are then discharged into an impoundment facility are covered by a layer of coarse-grained material like sand to assist with drainage of excess PWP and promote consolidation. Over many years of operation this can result in several layers of sand and fines interbedding within the impoundment, and lack of construction records, ongoing changes in geometry due to consolidation and settlement, and limited site investigation data often makes the deciphering of the internal TSF profile challenging. CPTu is the main technique employed in TSF site investigations to determine tailings strata and phreatic profiles, while in-situ dissipation tests and instruments such as vibrating wire piezometers (VWP) can be used to supplement or correct PWP interpreted from the CPTu.

One of the fundamental parameters used to identify the strength of soil is effective stress (σ'), which is defined in Eq. (1) and is physically described as the load transmitted between soil particles. As such, increased PWP (u) decreases σ' as water pressure is isotropic and as such repels external pressures on the system.

$$\sigma' = \sigma - u \quad (1)$$

However, as tailings generally exist in saturated or partially saturated conditions, a more important understanding of soil strength is acquired via the undrained strength ratio (USR), which is described in Eq. (2), where S_u is the undrained shear strength of the soil interpreted from CPT probing data.

$$USR = \frac{S_u}{\sigma'} \quad (2)$$

There are two ways by which the USR is commonly mischaracterised in practice. Recent studies have shown that the common practice of applying Eq. (2) to any undrained material may result in over-estimation of the USR in contractive layers of cohesionless soil (Sadrekarimi 2014). For such material layers, Eq. (3), which employs a simple shear failure mode to estimate the USR was used.

$$USR = 0.189 + 0.008q_{c1} \pm 0.025 \quad (3)$$

For $q_{c1} < 8$ MPa, where q_{c1} is the standard cone tip resistance.

Additionally, seemingly conservative phreatic profile interpretations can lead to mischaracterisations of the USR and overestimate the undrained strength ratio of the soil as increased PWP decreases σ' and therefore increases USR. Underestimating the phreatic profile would not model stability accurately due to decreased PWP in an undrained material. As such, simplistic approaches to stability analysis are not reliable in cases of layered TSFs with perched phreatic surfaces, and a robust analysis is needed to properly understand the complex phreatic pressure regimes and reliably interpret the USR. This paper demonstrates, through a case study, how such analysis can be implemented in practice with the graphical inference of CPTu data accompanied by dissipation tests and VWP data.

2. Facility Selection

The interlayered TSF is located in Australia, and was constructed using compacted process sand by-product with multiple upstream raises. The critical cross-section of the TSF was identified using a 2-dimensional limit-equilibrium stability analysis, and is the focus of this case study. CPT data from a recent site investigation on this cross-section were processed to estimate geotechnical strength parameters, and VWPs installed during prior site investigations along the cross-section were used to supplement the inference of the PWP profile from the CPT. The TSF, cross-section, data and instruments were all anonymised for the benefit of the facility operator. A preliminary cross-sectional stratigraphic profile was conceptualised based on past construction records, which show basic geometrical features like embankment slopes, crest geometries and elevations, with the aim to test their accuracy and refine it using the CPT data.

Table 1 and Figure 1 present the locations and depths of the CPTs conducted along the selected transect. These were strategically placed to target locations expected to be critical to the stability of the facility, and sufficiently spaced apart to provide broad understanding of potential variations in stratigraphy. The target depth was selected to provide sufficient range for dissipation tests to study the PWP profile, while also maintaining sufficient distance from the basal clay liner and underdrainage system to retain the integrity of the TSF.

Table 1. CPTs along Transect of Interest

ID	Surface Elevation (RL)	Termination Depth (m)
CPT1	40.8	13.4
CPT2	57.0	25.3
CPT3	63.6	32.8
CPT4	36.7	2.6
CPT5	65.1	36.9

3. Case Study Scenarios

The following scenarios were analysed to understand the impact of PWP interpretation on strength parameter derivation and ultimately the Factor of Safety (FoS):

1. Linear PWP profile, starting at the phreatic surface and extending to the depth of the TSF.
2. Non-linear PWP profile, with perched phreatic surfaces, each affecting distinct layers of embankment/tailings.
3. Linear PWP profile in the stability model as per Scenario 1, with strength parameters derived using Scenario 2.

4. Instrumentation

VWPs previously installed along the studied transect are summarised in Table 2, and not all corresponding to the locations of the CPTs included in Table 1 as compared in Figure 2. The PWP readings presented are maxima (excluding erratic readings) over the twelve month period preceding the CPT investigation.

Table 2. VWPs along Transect of Interest

ID	Install Elevation (RL)	Reading (mH_2O)
VWP1	34.0	-0.7 (dry)
VWP2	47.0	-0.5 (dry)
VWP3	40.7	1.9
VWP4	27.4	3.1
VWP5	52.5	-0.4 (dry)
VWP6	50.0	-0.1 (dry)
VWP7	45.9	-0.6 (dry)
VWP8	27.8	1.7

VWPs and in-situ dissipation tests were used to approximate the phreatic surfaces and distinguish between distinct phreatic surfaces. Three phreatic surfaces are clearly distinguishable in Figure 2 as shown.

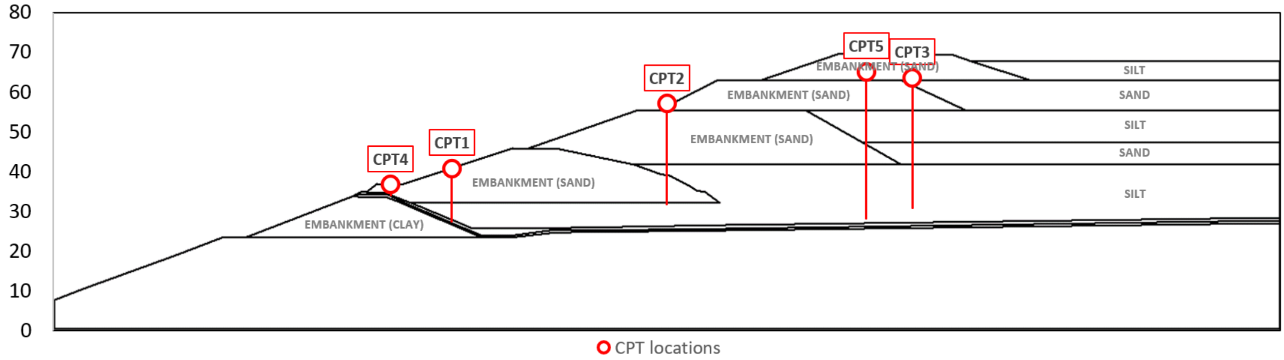

Figure 1. CPT Locations and Target Depths on Conceptualised Cross-Section

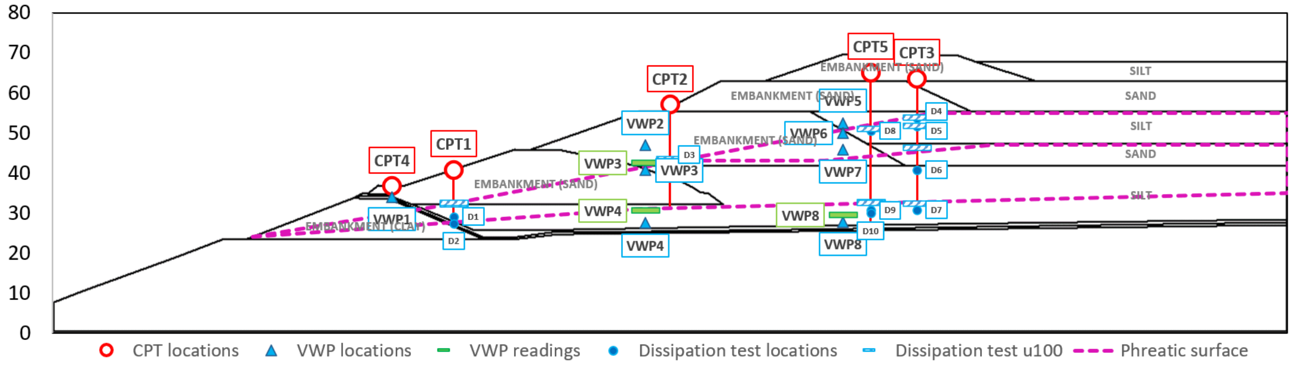

Figure 2. VWP and Dissipation Test Measurements Superimposed on Conceptualised Cross-Section with CPT Locations with Interpreted Phreatic Surfaces

5. Dissipation Tests

Obtaining the PWP in the soil at discrete depths is critical in developing an understanding of the PWP profile to aid strength parameter interpretation and stability analysis. During CPT probing, the PWP of the sheared soil, u2, consists of equilibrium and excess PWP. The equilibrium PWP needs to be measured to obtain the typical soil conditions when not undertaking CPT. Dissipation testing is used to measure the equilibrium PWP by halting CPT at specific depths and measure PWP over time as the excess PWP dissipates. Once all the excess PWP has dissipated, the equilibrium PWP, u100, is achieved. While this can take a few minutes in free-draining sand layers, equilibrium can take many hours to achieve in silts and may require overnight testing.

6. CPT Interpretation

CPT data was interpreted using a non-linear PWP profile. As such, dissipation tests and VWP data were used to obtain the PWP at discrete depths along the CPT sounding, and thus provide confidence in the interpretation of the PWP profile. Figure 3 shows the PWP profile developed for one of the CPTs, clearly differentiating between an assumed linear behaviour (green line), and the interpreted PWP (red line) based on interpreted points from engineering understanding of the stratigraphy and PWP supported by dissipation test results.

The linear PWP profile was used in Scenarios 1 and 3 (for the model), and the non-linear PWP profile was used for Scenarios 2 and 3 (for strength parameter derivation).

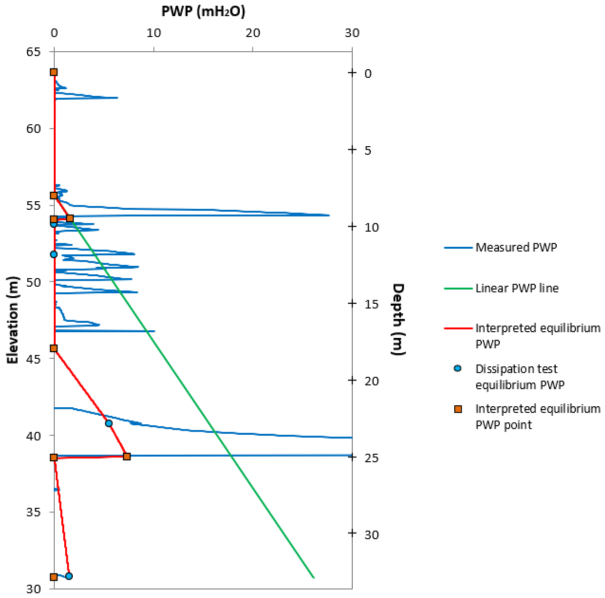

Figure 3. PWP Profile Inferred for CPT3.

Once a PWP profile for the depth of the CPT is established, the soil behaviour parameters of I_c and $Q_{tn_{cs}}$ could be calculated (Robertson 2009) as per Eqs. (4) and (5), where I_c is the soil behaviour type index, $Q_{tn_{cs}}$ is the normalised cone tip resistance for clean sands, Q_{tn} is the normalised cone tip resistance, F_r is the normalised friction ratio, and K_c is a correction factor dependent on soil plasticity and fines content (Robertson and Cabal

2015). This is an iterative process which could require the redefining of the PWP profile based on I_c as needed.

$$I_c = \sqrt{(3.47 - \log(Q_{tn}))^2 + (\log(F_r) + 1.22)^2} \quad (4)$$

$$Q_{tn_{cs}} = K_c Q_{tn} \quad (5)$$

I_c was primarily used to distinguish between silt and sand layers and in locating the perched phreatic surfaces as it only pertains to soil behaviour type and does not indicate the mechanical behaviour of soils. $Q_{tn_{cs}}$ was used to differentiate between contractive and dilative sand regions. Zones with an $I_c > 2.6$ were designated as silt, and sands with a $Q_{tn_{cs}} < 70$ were defined as contractive (Fourie, et al. 2022). The state of silt layers was determined using the state parameter (Ψ) based on Plewes (Plewes, et al. 1992), with contractive layers identified as those with $\Psi > -0.05$.

Although contractive material can be more accurately modelled using numerical techniques such as those employed by FLAC and Plaxis software, our approach adopts common practice, involving limit equilibrium 2D modelling which is sufficient to meet ANCOLD guidelines (ANCOLD 2019) for preliminary studies.

The I_c was used to distinguish silt and sand layers, and an identifying line (red line) at the location of the CPT was plotted to indicate silt regions at depth in Figure 4. Sand regions are shown by $Q_{tn_{cs}}$ (green line), which is then used to distinguish between contractive and dilative sands with the aid of a threshold (dashed black line) at 70. All silt layers were contractive due to $\Psi > -0.05$. The interpreted PWP profile (blue line) was also plotted (in reverse) alongside the silt identifying line and $Q_{tn_{cs}}$. Magenta regions were added to highlight changes from the expected stratigraphy which was shown in Figure 1 following overlay of CPT data.

Through overlaying CPT information into the section, an additional underlying contractive sand layer was confirmed around RL 35 m, a contractive sand layer was also identified in the embankment of the first raise, and the geometry of the second raise obtained from best estimates, was corrected.

The determination of a contractive sand layer at an elevation near the expected critical slip surface raised concerns regarding liquefaction of the tailings at this layer. To determine the impact of this layer, the lower phreatic surface elevation shown in Figure 2 was raised to span the top of the layer past where data has been obtained as shown by piezometric line 3 in Figure 5.

Once all embankment and tailings layers were distinguished and defined, the strength parameter of USR was calculated for each contractive layer and modelled using Stress History and Normalized Soil Engineering Properties (SHANSEP) (Ladd and Foote, 1974). For silt layers, USR was obtained as a direct interpretation of the CPT data, whereas USR for contractive sands was estimated using the simple shear failure mode method (Sadrekarimi 2014).

Dilative sand layers were assumed to be completely drained as the CPT data did not indicate evidence of saturation in the sands. As such, the effective cohesion $c'$and effective friction angle ϕ' were used to model the strength of dilative sand layers using the Mohr Coulomb model.

Figures 6 and 7 present the combined USR and peak friction angle data for all locations along the transect for the linear and non-linear scenarios respectively through use of CPT information only, with the distinct layers clearly marked and their associated strength parameters.

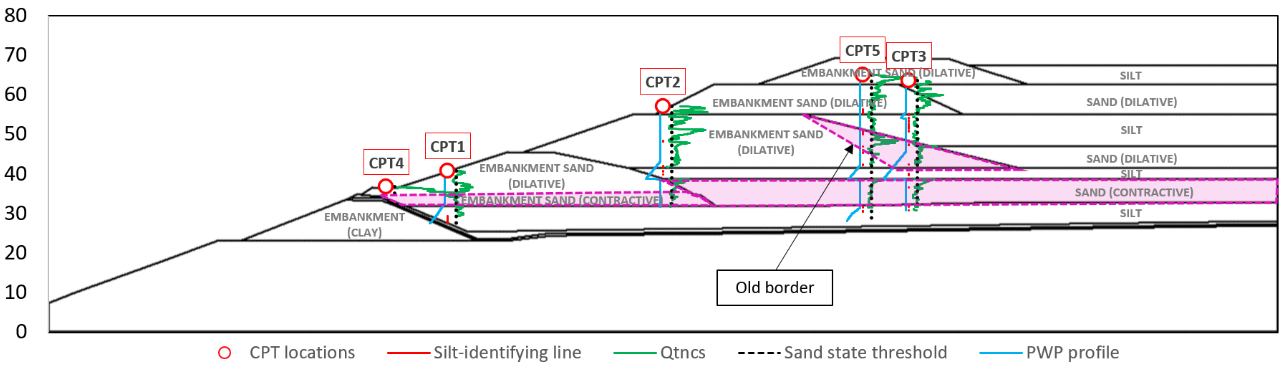

Figure 4. CPT Locations and Data Overlaid on Modified Cross-Section

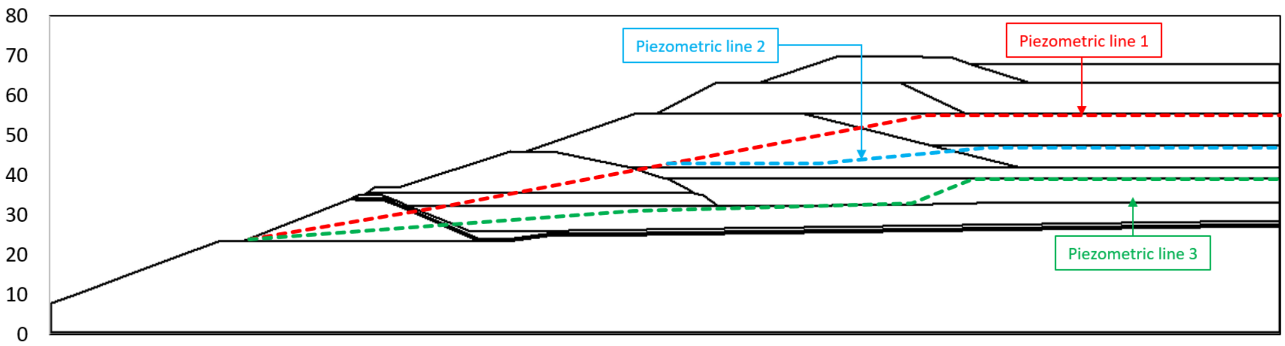

Figure 5. Piezometric Lines used for the Stability Model

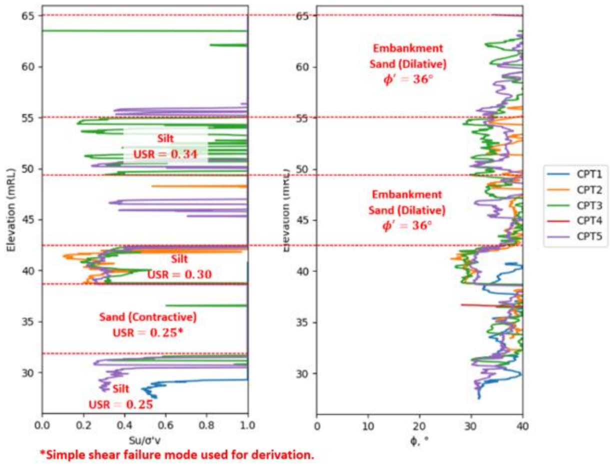

Figure 6. Linear Scenario Combined Plots of Peak Silt USR (left) and Peak Friction Angle (right) from CPT Data

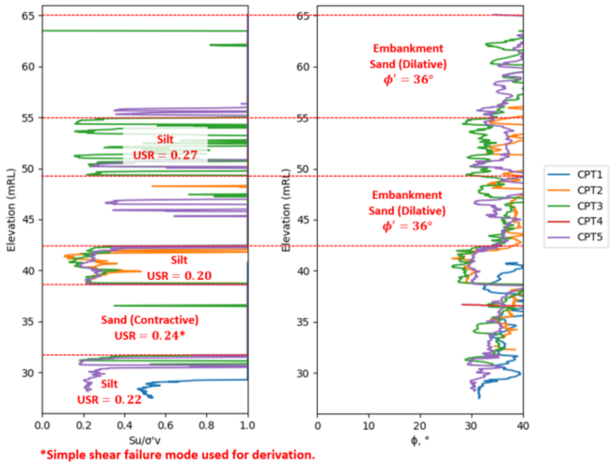

Figure 7. Non-Linear with Perched Phreatic Surfaces Scenario Combined Plots of Peak Silt USR (left) and Peak Friction Angle (right) from CPT Data

7. Limit Equilibrium Stability Analysis

Given the geometry model, strength parameter, and PWP profile have been established through available information, limit state-equilibrium stability analysis was undertaken on the revised geometry in Figure 4 using GeoStudio 2023.1.2. The following modelling parameters were employed:

- Morgenstern-Price analysis type.
- Optimised entry-exit slip surface.
- Unit weight of water, $\gamma_w = 10.3$ kN/m^3.

The piezometric lines used in the models were then extrapolated from dissipation test and VWP data as explained in Sections 4 and 5.

The limit-state equilibrium stability analysis results are summarised in Table 3. The results alongside material strength parameters used for each scenario are presented in the Appendix.

Table 3. FoS for Analysed Scenarios

Scenario	Description	FoS
1	Linear PWP profile modelled and used for strength parameter derivation.	1.26
2	Non-linear PWP profile (perched phreatic surfaces) modelled and used for strength parameter derivation.	1.22
3	Linear PWP profile modelled, and non-linear PWP profile used for strength parameter derivation.	1.17

The results presented in Table 3 demonstrate the following:

- An assumed linear PWP profile leads to an over-estimation of undrained strength ratio parameter, resulting in a non-conservative stability analysis (Scenario 1).
- More conservative soil strength parameters are obtained using a more realistic non-linear PWP profile with perched phreatic surfaces (Scenario 2).
- The most conservative model employs the use of both PWP profiles, with the non-linear profile used to derive the strength parameters, and the linear profile to model stability.
- While innately correlated, both the piezometric profiles used for the stability model and the PWP profile used for deriving the USR independently influence TSF stability. As such, careful consideration of the PWP on distinct material layers is required when undertaking stability analyses on layered TSFs. Scenario 3 provides an approach which benefits from the consideration of both linear and non-linear understanding of the PWP.

The results in this case study indicate that the TSF performs below the Australian industry practice guideline (ANCOLD 2019) with FoS < 1.5 for an undrained loading stability analysis. However, the importance of evaluating PWP profiles is clearly shown through the varying impact on the FoS, which could, in some cases, differentiate between compliance and non-compliance with local guidelines and government regulation. Subsequent analyses may uncover non-compliant TSFs, which would then need remediation works to be undertaken to meet such requirements.

8. Limitations

The scope presented in this study was limited to a single transect in the TSF focusing on primarily traditional stability analysis methods. Due to the prevalence of more complex methods employed on such projects, the following qualifications are made for this publication:

- A single transect was studied, limited only to investigate the effects of PWP profile interpretation on stability. Other factors such as embankment geometry were excluded from this study.

- While ‘perched’ phreatic surfaces were selected as a sufficiently representative PWP profile for layered TSFs with varying material permeabilities, it is recommended that the governing PWP gradient be obtained using a calibrated seepage analysis.
- This study focused only on undrained loading and did not consider the more critical post-seismic or post-liquefaction scenarios.
- Strength parameters utilised were based solely on empirical CPT data derivations, which were not confirmed nor calibrated with laboratory tests. Laboratory testing could be used to reinforce or correct the interpreted CPT data, subject to the quality of samples and handling.
- A simple 2D limit equilibrium model was used to determine the facility’s stability. There is an improvement opportunity by conducting quality laboratory testing and using numerical techniques if the facility owner is concerned about stability.
- The TSF foundations were assumed to be dry and high strength, thereby excluding failure through the foundation as a credible failure mode.
- The impact of the interface shear strength of the liner was excluded in the assessment of FoS.

9. Conclusions

Traditional stability analysis methods in which stability is highly dependent on PWP assumptions require detailed assessment of the PWP profile. In cases of tailings, interlayered TSF stratigraphy can result in complex PWP profiles particularly when different layers of disparate permeabilities exist. In such cases, assumption of simple, single linear PWP profile, instead of ‘perched’ phreatic surfaces can lead to over-estimation of embankment stability.

Comprehensive understanding of the PWP can be obtained using CPT and reinforced with in-situ testing and instrumentation. This case study demonstrated that both linear and non-linear PWP profiles can be used in the stability analysis to obtain the critical factor of safety. However, care is needed to ensure the undrained strength ratios derived from the CPT data are not over-estimated from considerations of conservative linear PWP profile.

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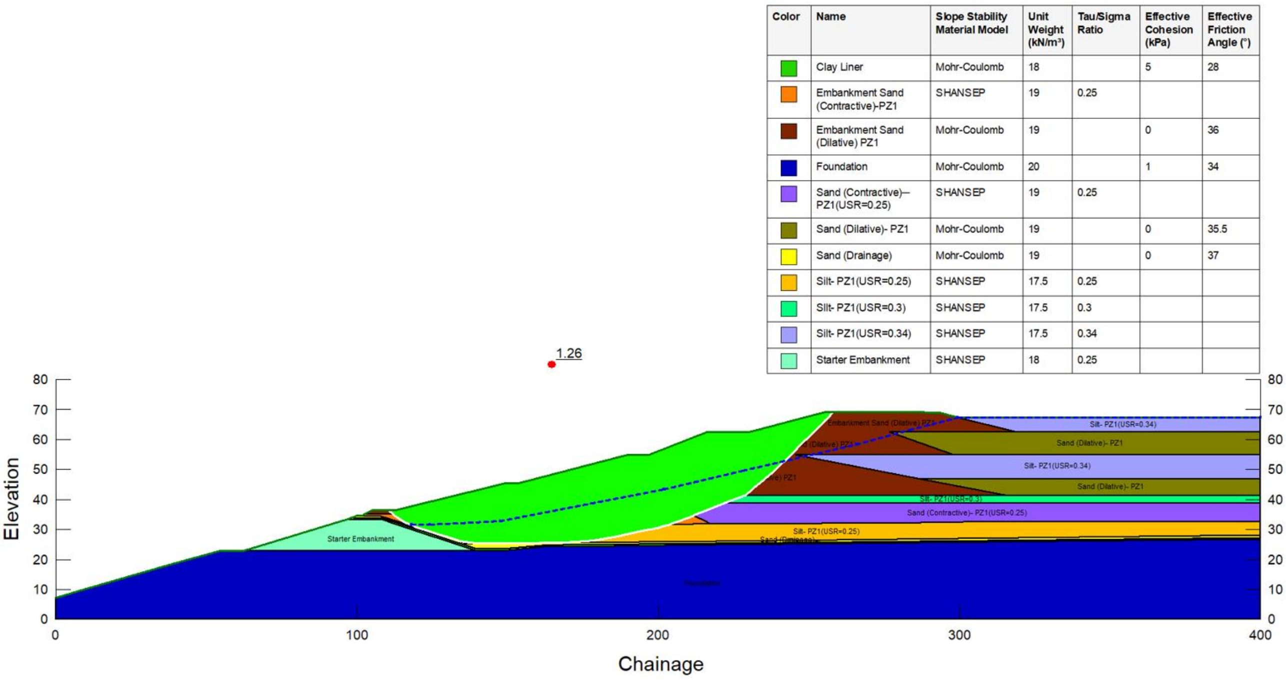
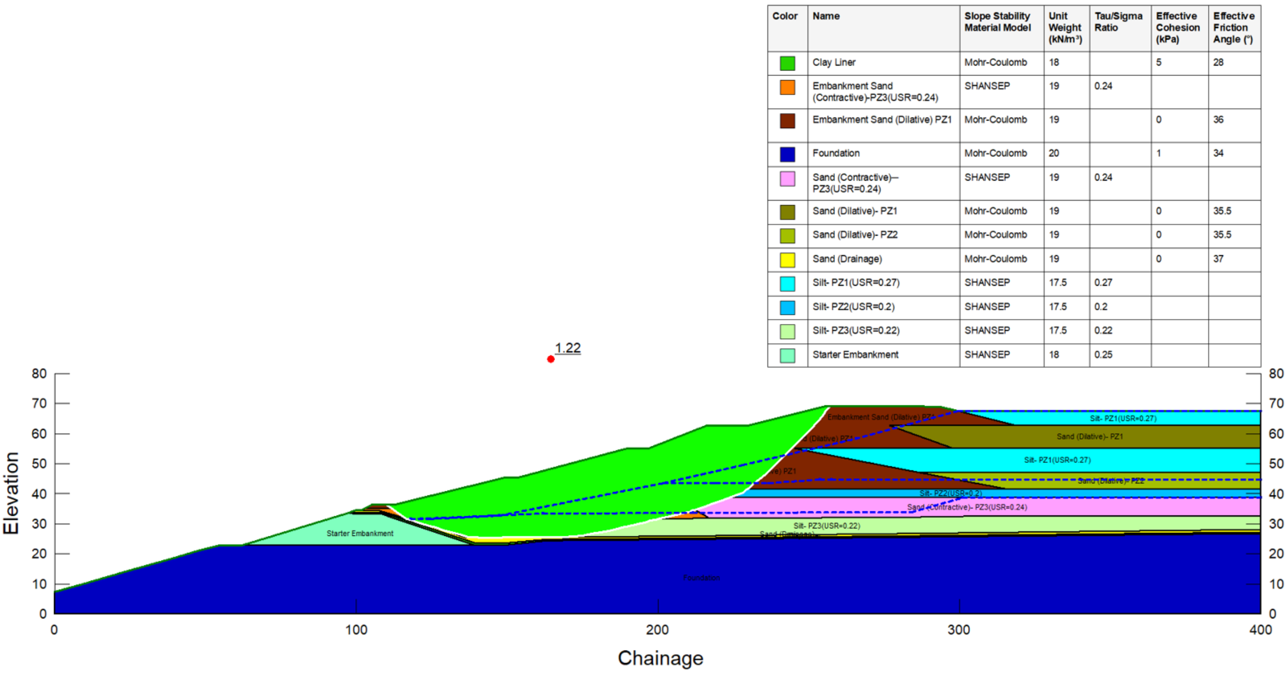
11. Appendix – Stability Analysis Results



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Clay Liner	Mohr-Coulomb	18		5	28
	Embankment Sand (Contractive)-PZ1	SHANSEP	19	0.25		
	Embankment Sand (Dilative) PZ1	Mohr-Coulomb	19		0	36
	Foundation	Mohr-Coulomb	20		1	34
	Sand (Contractive)–PZ1(USR=0.25)	SHANSEP	19	0.25		
	Sand (Dilative)- PZ1	Mohr-Coulomb	19		0	35.5
	Sand (Drainage)	Mohr-Coulomb	19		0	37
	Silt- PZ1(USR=0.25)	SHANSEP	17.5	0.25		
	Silt- PZ1(USR=0.3)	SHANSEP	17.5	0.3		
	Silt- PZ1(USR=0.34)	SHANSEP	17.5	0.34		
	Starter Embankment	SHANSEP	18	0.25		

Figure 8. Scenario 1 (Linear only) Stability Analysis Result.



Color	Name	Slope Stability Material Model	Unit Weight (kN/m³)	Tau/Sigma Ratio	Effective Cohesion (kPa)	Effective Friction Angle (°)
	Clay Liner	Mohr-Coulomb	18		5	28
	Embankment Sand (Contractive)-PZ3(USR=0.24)	SHANSEP	19	0.24		
	Embankment Sand (Dilative) PZ1	Mohr-Coulomb	19		0	36
	Foundation	Mohr-Coulomb	20		1	34
	Sand (Contractive)–PZ3(USR=0.24)	SHANSEP	19	0.24		
	Sand (Dilative)- PZ1	Mohr-Coulomb	19		0	35.5
	Sand (Dilative)- PZ2	Mohr-Coulomb	19		0	35.5
	Sand (Drainage)	Mohr-Coulomb	19		0	37
	Silt- PZ1(USR=0.27)	SHANSEP	17.5	0.27		
	Silt- PZ2(USR=0.2)	SHANSEP	17.5	0.2		
	Silt- PZ3(USR=0.22)	SHANSEP	17.5	0.22		
	Starter Embankment	SHANSEP	18	0.25		

Figure 9. Scenario 2 (Non-Linear only) Stability Analysis Result.

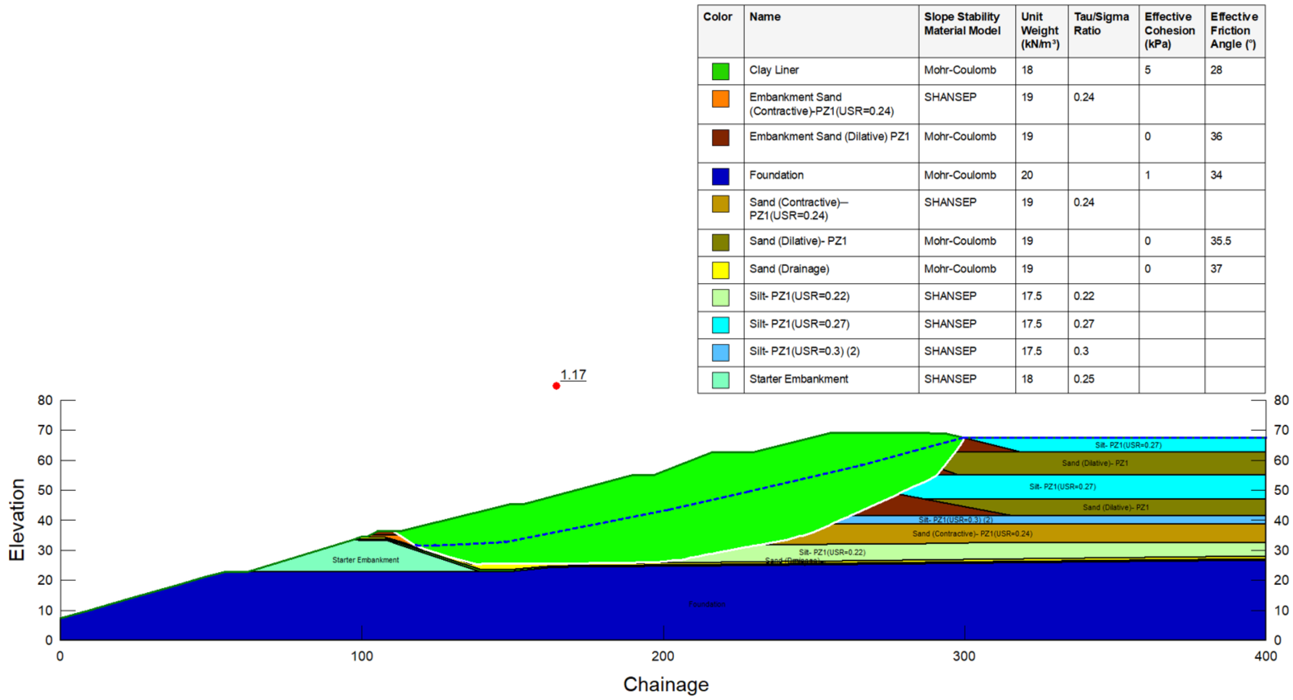


Figure 10. Scenario 3 (Linear PWP Profile, Non-Linear Strength Parameter Derivation) Stability Analysis Result.