Selection of soil shear strength parameters based on integrated *in situ* tests, lab tests and numerical calibration approach

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ABSTRACT

Site investigations were conducted to obtain soil parameters for stability and deformation assessment for an upstream tailings storage facility. Three material types that control the tailings dam stability were identified during the site investigations, comprising coarse-grained silty/sandy tailings, fine-grained silty/clayey tailings and residual soil. Advanced *in situ* tests, including Seismic Cone Penetrometer Tests (SCPT) and Vane Shear Tests (VST) were conducted. A comprehensive laboratory testing program was undertaken, which included direct simple shear (DSS) tests and triaxial tests to assess the Critical State Line (CSL).

An integrated approach was adopted to determine soil parameters, takings advantage of both *in situ* and laboratory test results. The peak undrained shear strengths ratios (s_u/σ'_v) of the fine-grained silty/clayey tailings were determined from the DSS test results. The CPT cone factor (N_{kt}) was calibrated using *in situ* s_u/σ'_v strengths of the materials in relatively thin layers. Subsequently, the *in situ* strengths of all other layers were calculated using the N_{kt} factor. Due to partial drainage in the coarse-grained tailings, its peak and residual undrained shear strengths were assessed using triaxial and non-standard VST test results.

The yield stress of the residual soil was determined using laboratory tests, Self-boring Pressuremeter (PMT) and FLAC™ numerical modelling results. The undrained shear strength of the residual soil was based on the calculated yield stress, PMT undrained shear strength and DSS test results.

INTRODUCTION

A comprehensive site investigation was carried out to characterise the undrained shear strengths for deposited tailings and foundation soil, due to the relatively high phreatic surface in an upstream tailings storage facility. Three types of materials that control the stability of the facility were identified during the site investigations, comprising:

- coarse-grained silty/sandy tailings
- fine-grained silty/clayey tailings
- foundation residual soil.

To carry out detailed stability assessments, the tailings and residual soil shear strengths need to be characterised. An integrated approach was selected, using *in situ* tests, laboratory tests and numerical modelling to select the soil strength parameters.

The site investigations comprised drilling and sampling to obtain high-quality undisturbed samples, laboratory testing, and the following *in situ* tests:

- Seismic Cone Penetration Tests (SCPTu), and non-standard Vane Shear Test (VST) in coarse-grained tailings.
- SCPTu and standard VST in fine-grained tailings.
- Pressuremeter (PMT) and standard VST in residual soil.

The methodologies to select the undrained shear strength parameters are described in this paper.

INTEGRATED APPROACH

Mayne (2005) recommended a hybrid of empirical, analytical, experimental, and/or numerical methods to interpret ground behaviour due to the inherent benefits and limitations of the various approaches. Table 1 summaries the benefits and limitations of *in situ* and laboratory test. It is evident that they are, in most cases, complementary. Consequently, the reliability of each method is greatly enhanced when used in conjunction with the other methods.

TABLE 1

Advantaged and disadvantages of *in situ* and laboratory tests.

	***In situ* tests**	**Laboratory tests**
Pros	• Continuous Profile (CPT). • Tests are carried out in place, in the natural environment, without sampling disturbance. • Testing is usually fast and repeatable. • A large soil volume can be tested.	• Samples are reconsolidated for testing at the proposed stress level. • Tests are performed in a controlled strain-rate, drainage and temperature environment. • Soil behaviour under cyclic loading conditions can be assessed.
Cons	• Sample cannot be tested at an increased stress level. • Some engineering properties are not measured directly and are based on correlations. • Some loading conditions cannot be tested eg cyclic loading.	• Samples may not represent the *in situ* condition. • A small soil volume is tested.

In addition to *in situ* and laboratory tests, numerical modelling was undertaken to calibrate soil parameters. One of the primary benefits of numerical modelling is the ability to incorporate advanced soil constitutive models, enabling the simulation of soil behaviour under complex loading conditions. Shape Acceleration Arrays (SAAs) were installed in the tailings storage facility, enabling the comparison of numerical results and SAA monitoring results to calibrate the soil parameters.

TYPICAL DAM SECTION

The typical tailing dam section is shown in Figure 1, illustrating that the coarse tailings is underlain by a layer of fine-grained tailings and residual soil at depth. The relevant undrained shear strength parameters were determined based on the potential failure mode, as follows:

- Triaxial compression for the coarse-grained tailings.
- Direct simple shear for the fine-grained tailings and residual soil.

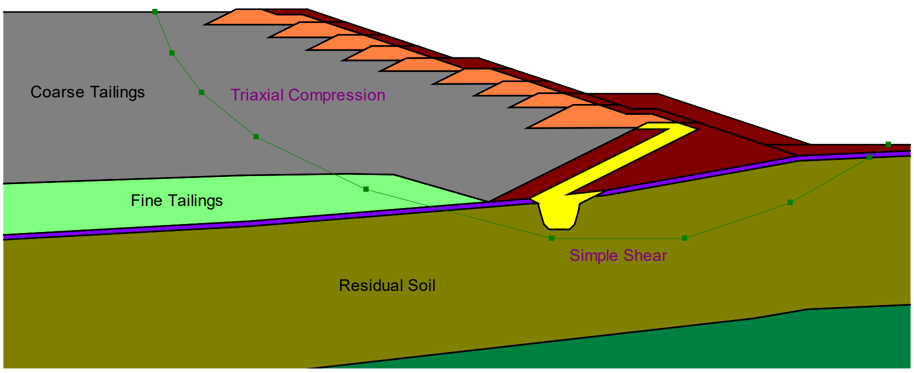

FIG 1 – Typical cross-section of tailings storge facility.

Based on the CPT results and the soil behaviour types by Robertson (1990, 2009, 2016), the following materials classification were made:

- Coarse tailings can be classified as 'Sand – Clean sand to silty sand', and 'Sand mixture – Silty sand to sandy silt'. Negative dynamic pore pressures were often recorded in dilative tailing layers and positive pore pressures in contractive tailings layers.
- Fine tailings, if encountered, can be classified as 'Silt mixtures – clay silt to silty clay'. Positive dynamic pressures were recorded in the tailings due to the undrained contractive condition.
- Residual soil, if encountered, can be classified as 'Silt mixtures – clay silt to silty clay'.

Figure 2 shows typical CPT results and material types from the CPT interpretation.

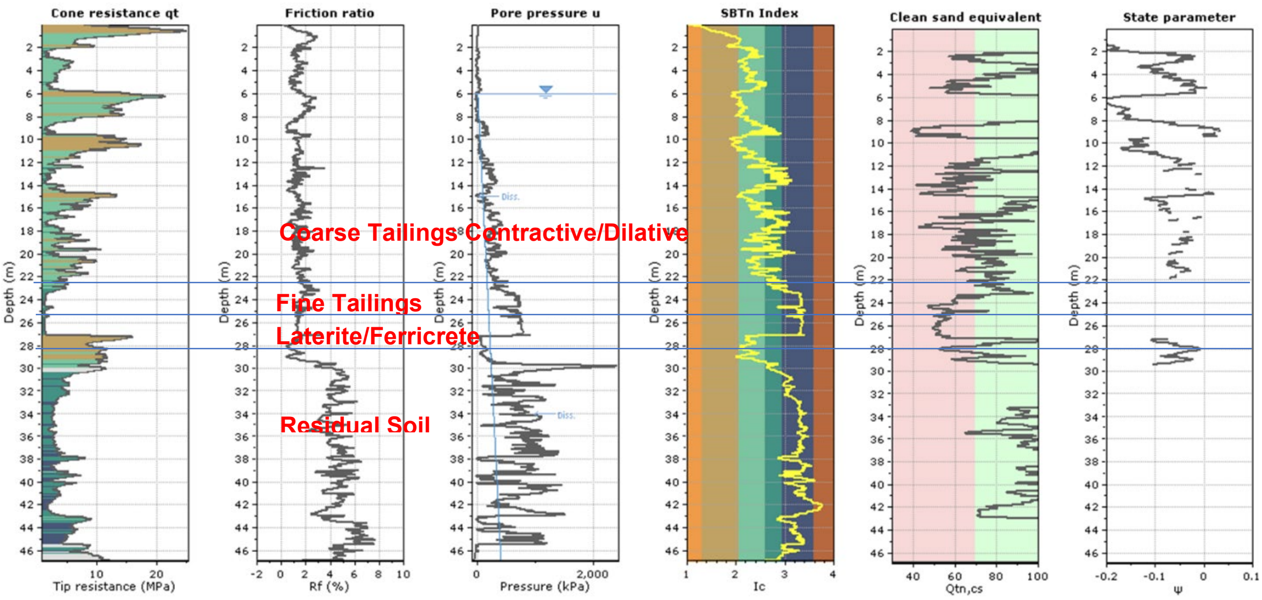

FIG 2 – Typical CPT results.

GEOTECHNICAL CHARACTERISATION

The characterisation of each material type is presented below.

Fine-grained tailings

The fines contents of the fine-grained tailings are typically 60 per cent to 90 per cent. The material exhibits typical clay-like soil behaviour.

The undrained shear strengths of the fine-grained tailings can be estimated using the total cone resistance (q_t) and a cone factor (N_{kt}). While a cone factor of 14 is typically adopted, literature indicates values ranging from 8 to 16.

The fine-grained tailings undrained shear strength was determined as follows:

- Determine the peak undrained shear strength ratio of the material under triaxial compression or direct simple shear using laboratory test results.
- Use the strength of the thin *in situ* material layers, which should be normally consolidated, to calibrate the N_{kt} factor.
- Use the calibrated N_{kt} factor to determine the shear strength of materials at other locations, including relatively thick layers.

Figure 2 shows the interpreted soil shear strength ratios:

- The s_u/σ'_v ratio under triaxial compression is approximately 0.31.
- The s_u/σ'_v ratio under direct simple shear is approximately 0.25.

Based on a s_u/σ'_v ratio of 0.25 and the thin fine-grained materials with a thickness of less than 5 m, a N_{kt} factor of 12 was assessed for material under simple shear. Figure 3 shows the interpretation of the undrained shear strength with a N_{kt} factor of 12 in comparison with laboratory test results.

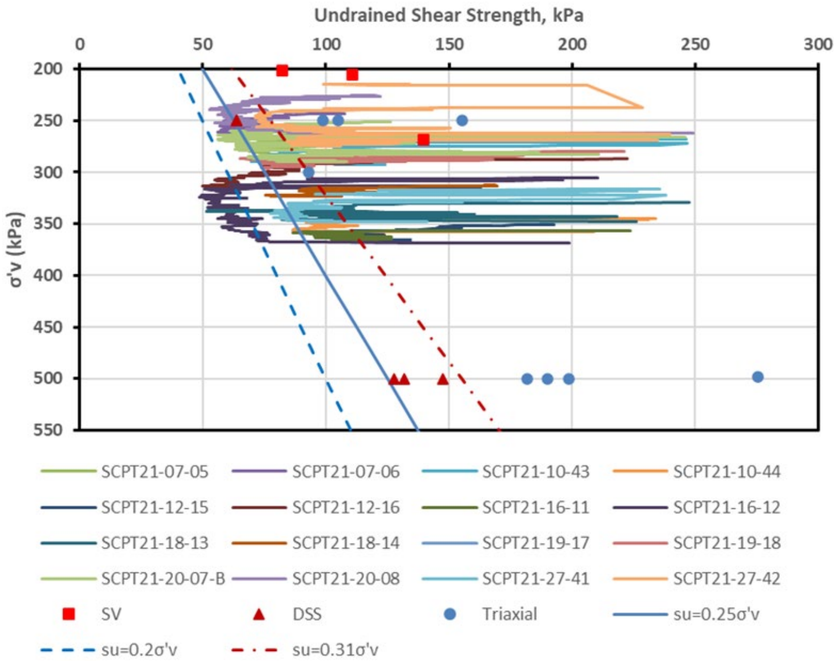

FIG 3 – Peak undrained shear strength of fine-grained tailings.

By combining *in situ* and laboratory tests it is possible to estimate the strength of the materials more accurately. Furthermore, this method can be used to identify materials that are still under consolidation, as shown in Figure 3 for materials with a lower undrained shear strength ratio of ~0.2.

Coarse-grained tailings

Undrained shear strength

The fines content for coarse tailings ranges from 40 per cent to 80 per cent, with low plasticity. The 50 per cent CPT dissipation time (t_{50}) is usually less than 10 seconds, suggesting partial drainage (Robertson, 2012; DeJong and Randolph, 2012). The vane shear test was therefore conducted at a non-standard rate of 240°/min to ensure undrained behaviour (Reid, 2016).

The peak undrained shear strength was proposed as a function of the state parameters based on the CSL triaxial test results and 240°/min vane shear test results. Figure 4 shows the relationship. Some of the VST tests undertaken during the early project stages were carried out at the standard shear rate of 12°/min. Due to partial drainage, these tests indicated higher undrained shear strengths.

Figure 5 shows the residual undrained shear strength, based on vane shear tests results and CSL triaxial test results. The relationship correlates well to the Jefferies and Been (2016)'s best practice line.

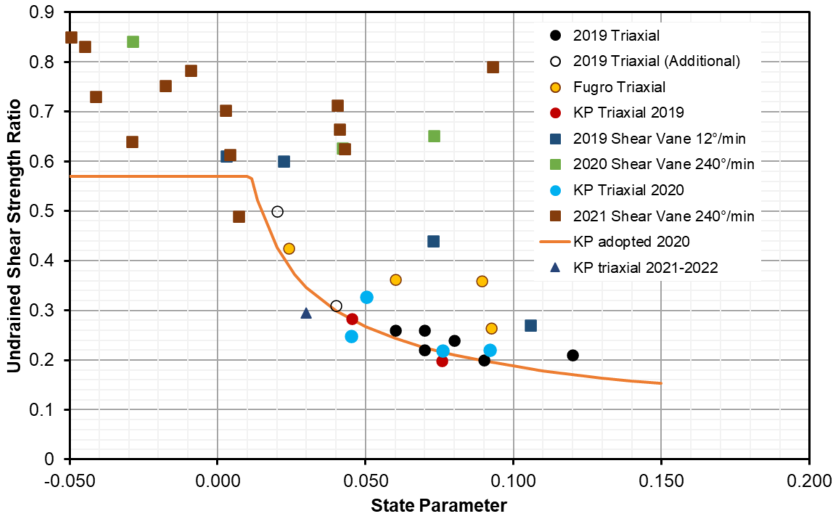

FIG 4 – Peak undrained shear strength of coarse-grained tailings.

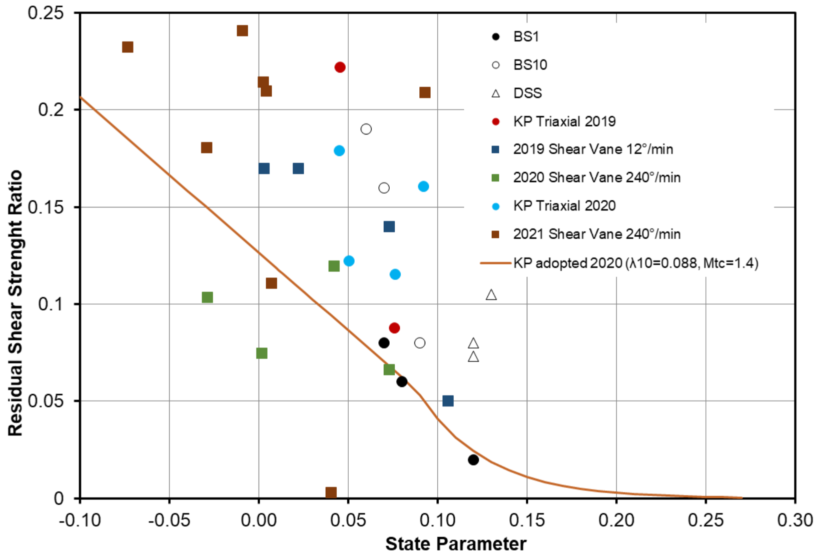

FIG 5 – Residual undrained shear strength of coarse-grained tailings.

State parameter

The state parameter is an important parameter that should be determined accurately for the coarse-grained tailings. The interpretations of the state parameters were based on the following methods: Robertson and Cabal (2015) and Plewes, Davies and Jefferies (1992).

The CPT 'widget' (Shuttle, 2019), which is a finite element cavity expansion program, further improved CPT data interpretation.

All methods yielded similar results as shown in Figure 6. Therefore, either of Robertson and Cabal (2015) or Plewes, Davies and Jefferies (1992) methods can be used for the state parameter interpretation. Robertson updated the Robertson and Cabal method in 2022 (Robertson, 2022). The updated state parameters using the Robertson (2022) method are typically ~0.02 smaller than those from the Robertson and Cabal (2015) method. However, the residual undrained shear strength would be similar if the following methods are adopted:

- Jefferies and Been (2016) residual strength method using either Plewes, Davies and Jefferies (1992) or Robertson and Cabal (2015) state parameters.
- Robertson (2022) residual strength with Robertson (2022) state parameters or $Q_{tn,cs}$.

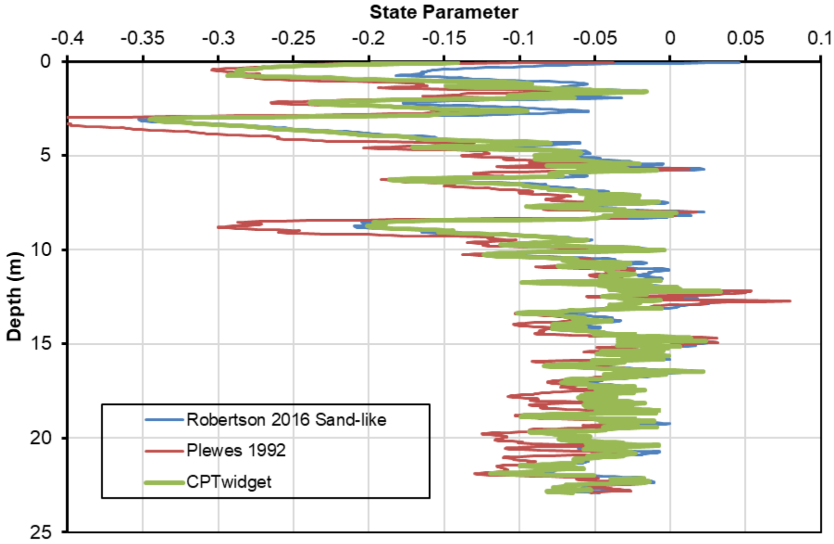

FIG 6 – State parameters by Robertson and Cabal (2015), Plewes, Davies and Jefferies (1992) and CPT 'widget' method.

Residual soil

Yield stress

All residual soils behave as if over-consolidated to some degree (Leroueil and Barbosa, 2000). Their compressibility is relatively low at low stress levels. Once a threshold has been exceeded, the compressibility increases. Traditionally, the threshold was termed as 'pre-consolidation stress' for transported soil. However, residual soils are formed as the result of weathering and decomposition of rocks and thus stress history is of no relevance (Wesley, 2010). A more general term of 'yield stress' is recommended for residual soils (Wesley, 2010; Mayne, 2017) as a measure of the strength of the interparticle, or inter-mineral crystal bonds remaining in the soil after weathering.

The over-consolidation behaviour of the residual soils was assessed based on oedometer, Constant Rate of Strain Cell (CRS) and self-boring PMT tests. In addition, FLAC™ modelling was undertaken to predict embankment deformations. A Plastic Hardening (PH) Model with memory of yield stress (equivalent pre-consolidation) was used to model the residual soil. A representative yield stress was determined through calibration of the FLAC™ model against inclinometer monitoring results through a trial-and-error approach. The stiffness of the residual soil was determined by the PMT tests, laboratory tests and Standard Penetration Tests (SPT). In addition, in order to accurately model the brittle coarse tailings behaviour, a NORSAND model was adopted for the coarse tailings. The calibration indicates a yield stress of between 500 kPa and 550 kPa.

Table 2 summarises the determined yield stress through the laboratory tests, *in situ* tests and numerical modelling. Figures 7 and 8 show the predicted embankment deformation and the calibration of the FLAC™ modelling against the inclinometer monitoring results.

TABLE 2

Yield stress of residual soil.

Methods	Yield stress (kPa)
Oedometer	290–1230
CRS	730–1080
Self-Boring PMT	340–660
Numerical Calibration	500–550

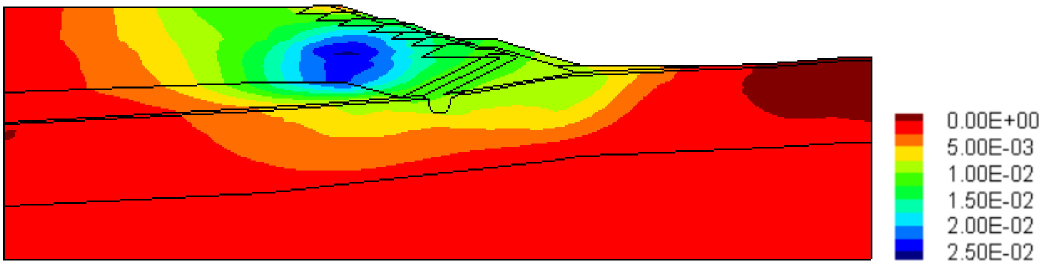

FIG 7 – Predicted FLAC horizontal displacement results (Unit: m).

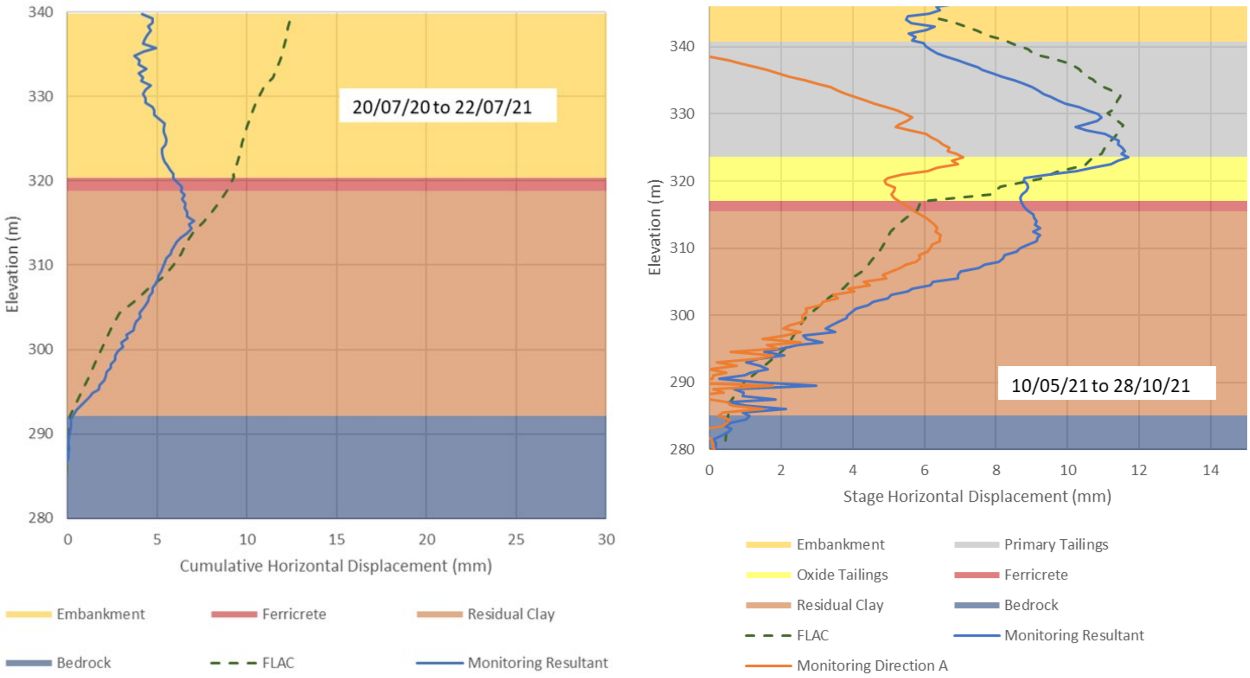

FIG 8 – Comparison of predicted deformations versus site monitoring data.

Undrained shear strength

Figure 9 illustrates all triaxial and DSS test results conducted on undisturbed residual soil samples. It shows a clear overall trend with the triaxial compression strength exceeding the DSS strength. The s_u/σ'_v ratio under direct simple shear is approximately 0.26. The factor of $s_{uDSS}/s_{uTriaxial}$ is approximately 0.7, which is consistent with Mayne (1985).

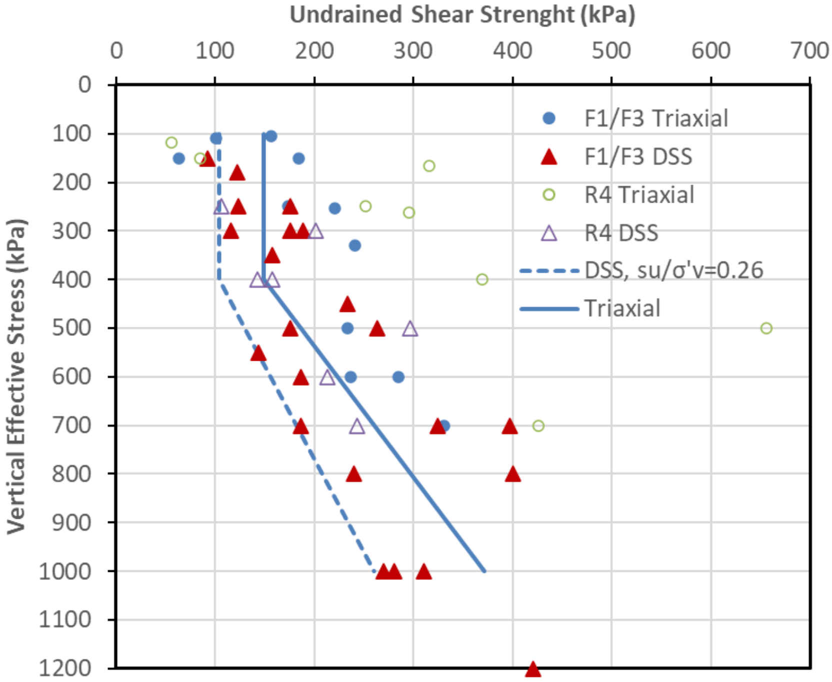

FIG 9 – Undrained shear strength of residual soil under triaxial compression and DSS condition.

Based on Unconsolidated Undrained (UU) triaxial test results conducted on samples collected at similar PMT depths, the interpretated PMT undrained shear strength is approximately the undrained shear strength under triaxial compression. The interpreted PMT undrained shear strength is therefore converted to s_{uDSS} by applying a factor of 0.7. The proposed undrained strength parameters under simple shear condition are presented in Figure 10 based on the converted PMT undrained shear strength and DSS test results.

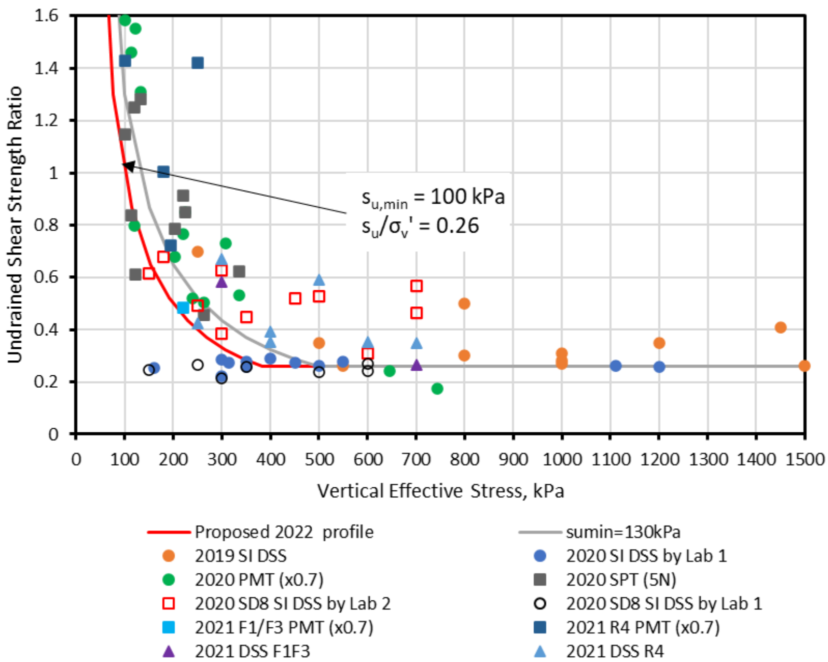

FIG 10 – Undrained shear strength of residual soil under triaxial compression and DSS condition.

Some of the early DSS tests were conducted independently in two different labs and they showed different results, with one indicating over consolidated behaviour and the other nearly normally consolidated behaviour. Upon comparison with the PMT tests and FLAC numerical modelling results, it was determined that the normally consolidated behaviour results may be erroneous, and they were excluded from the interpretations.

The following undrained shear strength was recommended for the stability assessments:

- Minimum undrained shear strength of 100 kPa (corresponding to a yield stress about 400 kPa).
- An undrained shear strength ratio of 0.26.

The recommended minimum undrained shear strength of 100 kPa is approximately 80 per cent confidence level of the materials with vertical effective stress less than 300 kPa. The mean minus half standard deviation value is around 130 kPa, which corresponds to a yield stress about 500 kPa. This is consistent with FLAC numerical modelling calibration.

CONCLUSIONS

Both *in situ* and laboratory tests have their inherent benefits and limitations. By conducting both types of tests, it is possible to calibrate the *in situ* tests and identify any improperly conducted laboratory tests.

This study has illustrated that:

- Numerical modelling can be used to calibrate undrained shear strengths and residual soil yield stresses.
- It is possible to simulate soil behaviour under complex loading conditions using advanced soil constitutive models.

By utilising the benefits of *in situ* tests, laboratory tests and numerical modelling, the undrained strengths of tailings and residual soil could be more accurately determined for different stress conditions.

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