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Characterizing the stiff clay foundation soil below a TSF in Western Australia

David Reid, Riccardo Fanni, Peter Chapman
Golder Associates Pty Ltd

ABSTRACT

The recent tailings storage facility (TSF) failure at Mt Polley has highlighted the importance of the proper identification and characterization of stiff clay foundation soils. Specifically, the propensity for over consolidated stiff clays to transition to a normally consolidated state under vertical loading is a critical design consideration for TSFs constructed on stiff clay foundation soils.

In many parts of Western Australia, TSFs are constructed on relatively stiff clayey soils. These materials can be either residual or transported, with materials of both origins found in relatively close proximity in some cases. Such soils may, under some loading conditions, and if saturated, exhibit contractive undrained behavior, giving mobilized shear strengths lower than drained strength. However, such soils are typically unsaturated prior to commencement of tailings deposition within the TSF, and may remain in such a state depending on the life of the facility and the influence of the hydrogeological conditions on seepage from the TSF. Further, residual soils are in some ways more difficult to characterize than transported soils, particularly with respect to defining a pre-consolidation stress and defining normally consolidated conditions.

To characterize the behavior of a clay foundation soil below and adjacent to a TSF in Western Australia, a number of block samples were obtained in field investigations from near-surface locations, adjacent to the toe of the TSF. The material was then tested using a direct simple shear apparatus under a range of vertical effective stresses relevant to existing and future TSF loads. Preliminary results indicate that the material can exhibit contractive behavior when sheared under high vertical effective stresses, which are relevant to future TSF loading. Piezocone Penetration Testing undertaken near to the sample location provided results inconsistent with the laboratory testing. The reasons for potential divergence of the in situ and laboratory testing of the material are discussed.

1. INTRODUCTION

One of the key learnings of the Mt. Polley review panel (Morgenstern et al. 2015) was a reminder of the potential for stiff, over consolidated, likely dilative clays to transition to a normally consolidated, contractive state upon significant loading, whether by embankment construction or tailings deposition. Cognizance of this phenomenon is particularly important when considering that many TSFs are eventually raised higher than that envisaged during the initial geotechnical investigation and design process.

The presence of relatively stiff, presumably over consolidated fine-grained foundation soils in Australia is common. This includes common mining areas, such as the Pilbara and Goldfields regions of Western Australia. The materials are generally stiff to very stiff, and are unsaturated at shallow depths. Importantly, in many cases these materials consist of residual, rather than sedimentary soils. Residual soils are more difficult to characterize in some ways to sedimentary soils, particular with respect to pre-consolidation pressure and peak undrained strength ratio (PUSR). In particular, the use of stress history to conceptualize residual soils may not be appropriate in many cases (for example, Wesley 2009). The stiff material state, and apparent yield stress exhibited by residual soils is often a result of the process by which residual soils form from parent rock, not a result of subsequent stress history. Alternatively, some residual soils may have undergone more "conventional" loading in their geological history, owing to events occurring after the transition of the material from rock to soil. Of particular relevance in this context with respect to the arid and semi-arid conditions of portions of Australia is desiccation, which can result in significant pre-consolidation pressures.

For the purposes of a stability analysis of a TSF underlain by such fine-grained soils, defining both the PUSR, and the pre-consolidation pressure (if it is exists) are essential. While the increasing height of a TSF may cause some proportion of the foundation to return to a normally consolidated state, for a typical facility with relatively flat perimeter embankments, it is likely that a range of PUSRs will exists below the length of the slope. It is noted that the relatively flat embankment slopes observed on many upstream-raised TSFs are not typically based on consideration of foundation materials. Rather, they are typically adopted either for closure considerations, from local "rule of thumb" experience, to minimize the likelihood for sufficient in situ shear stresses to trigger static liquefaction (for example, Davies et al. 2002), or some combination of these factors. However, the relatively flat slopes of such TSFs reduce the likelihood that clay foundation soils transitioning to a normally consolidated state will result in unsatisfactory perimeter stability. Despite this likelihood, characterization of the foundation materials as TSFs heights continue to increase is crucial to the analysis of potential failure mechanics of the perimeter embankments.

This paper outlines a laboratory and in situ test program undertaken on near-surface block samples of a clayey foundation material relevant to an Australian upstream-raised TSF. The material was obtained from just outside the toe of the TSF perimeter embankment. Consistent with many TSFs, the facility considered here has continued to increase beyond the height envisaged in the initial designs – undertaken over 20 years ago. Therefore, additional characterization of the foundation materials was important as part of ongoing monitoring and analysis of the TSF. Testing focused primarily on direct simple shear (DSS) testing, owing to the importance of this loading direction within foundation materials.

2. SAMPLING LABORATORY CHARACTERIZATION

2.1 Geological and Regolith Setting

The geology and regolith profile of the TSF area was obtained from review of the Kanowna 1:100,000 Geological Series map (issued by the Department of Mines of Western Australia 1995). The map describes the study area as characterized by the following geological units:

- (Czc) Colluvium sand and soil, includes laterite fragments
- (Czl) Laterite (ferricrete) and reworked products
- (Czw) Weathered rock; protolith unrecognisable
- (As) Sedimentary and felsic volcaniclastic rocks, undivided; commonly highly weathered
- (Qa) Alluvium-clay, silt, sand, and gravel

The pediment is typically overlain by up to 25 m of colluvium and alluvium deposits. The residual regolith profiles are typical of lateritic weathering profiles exhibiting variable depths and material types due to the weathering of differing sedimentary and volcanic parent rock types. The residual regolith material generally varies from sandy clay with sparse lag to ferruginous soils with abundant lag, overlying saprolite.

Ferricretes (probably ferruginised saprolite) generally occur on the hill crests and rises, overlying saprolite, and cemented ferruginous gravels that occur either along former drainage lines or as scree deposits (valley-fill) along hill slopes. The alluvial deposits form a thin veneer generally less than 2 m thick and range in materials from sandy clay, silts to clayey gravels.

2.2 Sampling Process

Samples for laboratory testing were obtained in block format adjacent to the toe of the TSF, from depths of 100 – 300 mm. Material was excavated from around blocks, typically 150 mm square, by 200 mm high, following which the blocks themselves were removed by cutting beneath them with a sharp blade. The orientation of the blocks was labelled, and they were then carefully wrapped to prevent moisture loss and placed in bubble-wrap lined boxes. The samples were then transported by road approximately 600km from the mine site to the testing laboratory in Perth, Western Australia. Care was taken during road transport to minimize disturbance. For example, the samples were loaded just prior to departure for Perth, so that the minimum distance possible would be travelled with the samples. Also, the transport vehicle was slowed to approximately 30 km/h when passing over level rail crossings on the route to Perth, rather the typical demarcated speed limit of 110 km/h.

Figure 1. Block sample prior to removal.

In the laboratory, the blocks were inspected to locate areas suitable for trimming, particularly with respect to gravel-sized particles seen throughout the material (discussed below). Cylindrical samples were obtained by incrementally pushing sharp-edged rings into the material while trimming material outside the ring with a scalpel. The commencement of this process is shown in Figure 2. The rings used were either of 63.5 or 60.0 mm internal diameter, for DSS and oedometer testing, respectively. While care was taken in the sampling and trimming process, it is noted that the presence of evenly distributed gravel-sized particles within the material lead to some difficulties. The gravel-sized particles, and their implications on soil type and history, are discussed below.

Figure 2. Block sample trimming for DSS testing

2.3 Index Testing

Index and general characteristic testing was undertaken on the material to enable correlation of test data produced to typical trends for plasticity, gradation, and mineralogy. Testing consisted of particle size distribution (PSD), Atterberg Limits, particle density, cationic exchange capacity (CEC) and semi-quantitative X-Ray Diffraction (XRD). The XRD testing was undertaken on the fines (<75 µm) component of the material, as it was the mineralogy of the fines that was of primary interest. The testing was undertaken on trimmings obtained during preparation of test samples from the blocks of material. The liquid limit, plastic limit, and plasticity index for the material were 41%, 15%, and 26%, respectively. These plasticity characteristics are typical of residual soils. The specific gravity of solids (G_s) was measured as 2.77. The PSD of the material estimated by means of sieving and the Sedigraph technique, indicating material D_{90}, D_{50}, and D_{10} of 3 mm, 0.019 mm, and 0.001 mm, respectively. The gravel particles were black in colour, angular in shape and magnetic, likely characterised by chemically unweathered biotite. Further PSD data is provided in Table 1.

On the basis of the results available, the USCS[1] classification of the material is CL, lean clay with sand.

Table1. Particle size distribution

Particle Size (mm)	Percent Finer (%)
4.75	93
0.075	60
0.002	21

The XRD testing showed that the fines are predominantly characterised by quartz and muscovite minerals. The presence of dickite minerals was also detected. Dickite is a phyllosillicate clay mineral composed by aluminium, silicon, hydrogen and oxygen that is part of the Kaolinite-Serpentine group. Dickite is commonly developed from the weathering of feldspars and muscovite. Its formation is due to hydrothermal activity and it has also been found as an authigenic component of sandstones and other sedimentary rocks. As described in the geological setting section, the study area is characterised by metamorphosed volcanic rocks. Therefore, it is more likely that the presence of dickite in the soil is the result of the presence of a hydrothermal system, which formed *in situ* through the alteration of aluminosilicate minerals (e.g. muscovite) by hydrothermal acid waters.

Although the presence of dickite is minor compared to other minerals detected by the XRD analysis, this mineral may have contributed to provide the clayey type of behaviour in term of plasticity to the soil. Clay minerals from the kaoline group do not significantly swell or disperse and they tend to over consolidate as result of shrinkage due to desiccation and/or due to seasonal changes of the groundwater table.

It is important to highlight some incongruence between the semi-quantitative XRD analysis, the CEC testing and the clay activity. The CEC of kaolin minerals generally is between 3 – 15 meq/100g. This CEC range is lower than the CEC value detected for the studied soil of 28 meq/100g, which is more typical of chlorite and illite type of clay minerals. The high clay activity value of 1.37 could also be due to presence of other clay minerals (e.g. illite has generally activity up to 1.3) as kaolin minerals have activity in the range of 0.3-0.5. These incongruences may be due to the presence of other clay minerals in the soil that may have not been detected by the semi-quantitative XRD analysis.

2.4 Direct Simple Shear Testing

Samples for DSS testing obtained through trimming of the blocks were covered by filter stones at either end, and submerged in a bath of decant water obtained from the TSF. Water collected from the surface of the TSF (the supernatant pond, or decant pond) was used as it is likely to form the primary source for the potential saturation on the clay foundation, and as the mechanical behavior of fine-grained soils may be effected by pore fluid chemistry. The samples were saturated for 24 hours prior to testing. It is noted that there is no assurance that such a procedure will result in fully saturated conditions. However, as noted by Al-Tarhouni et al. (2011), when adopting constant-volume shearing techniques with the DSS device, achieving fully saturated conditions is not required to produce results consistent with those of a saturated specimen. In other words, provided

[1] Unified Soil Classification System (ASTM D2487-11)

suctions within the soil are significantly reduced, the enforced constant-volume condition of the DSS will prevent unsaturated conditions from diminishing the contractive nature of the soil, and the influence of contractive behavior on PUSR.

As noted, the tests were undertaken in a constant-volume condition. The DSS device used was of the "SGI Type", that is, the sample was laterally restrained by a series of Teflon rings. Machine corrections based on compressibility of the device were applied to the testing.

The end-platens of the device included small aluminum "pins" embedded in the filter stone, to reduce the potential for the sample to slide on the ends during shear. While these pins are consistent with current DSS testing practice (ASTM D6528-07, NORSOK 2004), they may produce difficulties if attempting to infer pre-consolidation pressure using the DSS results. Specifically, when placing a stiff clay sample onto platens with such pins, it may be difficult to ensure they are fully embedded within the specimen. Hence, as increasing increments of vertical stress are applied to the specimen the pins may penetrate further in the soil, influencing the vertical displacements recorded. This is noted where relevant below.

Tests were undertaken at a range of vertical effective stresses (100, 250, 500, 1000 kPa), to provide an indication as to the material behavior at loads corresponding to various locations across the length of the slope of the TSF – which may eventually reach heights of approximately 60 m. An additional test was undertaken by consolidating the specimen to 1000 kPa and unloading to 250 kPa prior to shear. This test was conducted to assess the undrained shear behavior of the material at a known OCR, for comparison to the inferred OCRs for the other samples, where relevant. The tests are listed in Table 2.

Table 2. DSS Tests

Test No.	Max. Vertical Effective Stress (kPa)	Pre-Shear Vertical Effective Stress (kPa)
1	1000	1000
2	500	500
3	250	250
4	100	100
5	1000	250

2.5 Oedometer Testing

Oedometer testing was undertaken on samples obtained in a similar manner to DSS testing. A bedding load of 12 kPa was applied, following which the material was flooded. The material was then loaded in relatively small increments of 25 – 50 kPa up to 300 kPa, and 100 kPa subsequently, to improve the ability of the test to provide an indication of pre-consolidation pressure.

2.6 Peak Undrained Shear Strengths

The shear strength results for DSS tests 1 - 4 are outlined in Figure 3. Strength is presented in terms of strength ratio, to allow the various tests at different vertical effective stress to be effectively compared. Tests from 250, 500, and 1000 kPa resulted in a relatively similar set of strengths, with an average PUSR of 0.27. Test 4, undertaken from 100 kPa vertical effective stress gives a higher strength ratio of 0.36. Compared to the other tests at higher stresses, this result may indicate a pre-consolidation pressure for the material in excess of 100 kPa.

Test 5, which was consolidated to a known OCR of 5, achieved a PUSR of 0.86. Assuming that the SHANSEP method (Ladd and Foot 1974) could be applied to this material, the five tests undertaken indicate SHANSEP parameters of s = 0.27 and m = 0.84. Applying these parameters to the interpretation of Test 4 suggests a pre-consolidation pressure of 144 kPa for the material tested. Whether this is a "real" pre-consolidation, i.e. a result of stress history, is unknown.

However, in practice, it may not be important. It is clear that the material exhibits higher PUSR at lower vertical effective stresses, indicating that for the purposes of accounting for the strength of the material across the length of the slope, PUSR should be made a function of vertical effective stress.

Tests 1 and 2 show some evidence of post-peak strain softening, while Test 3 does not. It is interesting to note that Test 3, at 250 kPa, is under the lowest vertical effective stress of the tests that are inferred to be normally consolidated. Therefore, perhaps the proximity of this sample to the pre-consolidation pressure of the material is influencing its behavior by reducing brittleness. However, the PUSR for this test was within the range of the other normally consolidated tests.

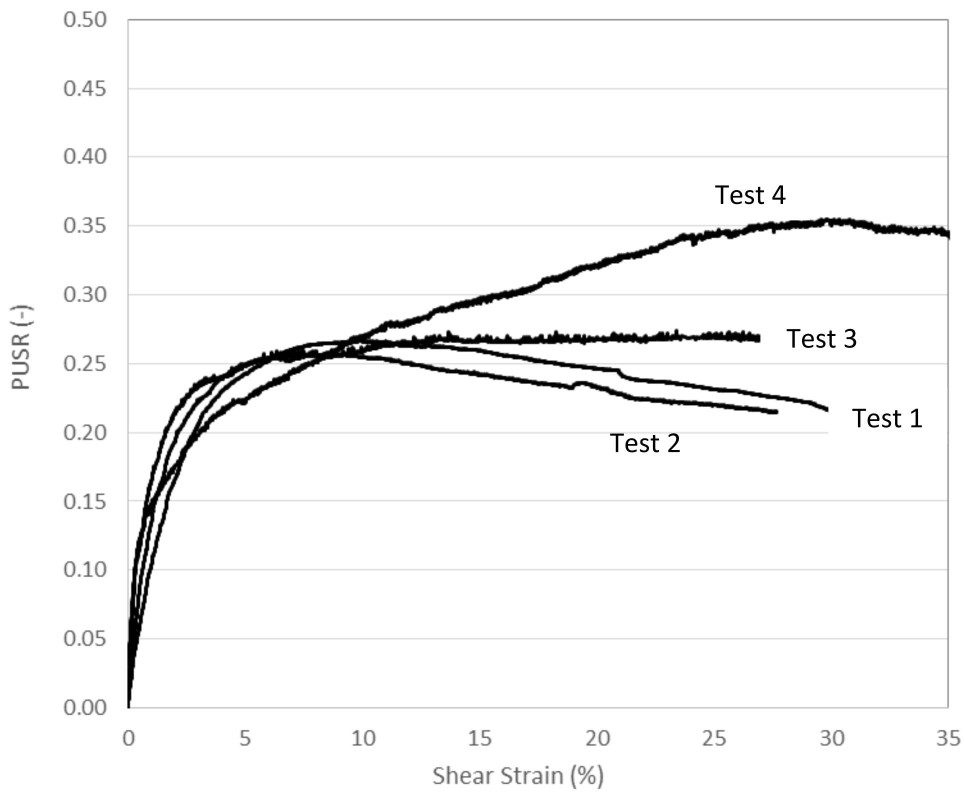

Figure 3. DSS Test Results.

2.7 Pre-consolidation Pressure

The void ratio – vertical effective stress consolidation results for the DSS and oedometer test are presented in Figure 4. The oedometer and DSS results agree reasonably at vertical effective stresses from 250 kPa and above, whereas at lower stresses the oedometer indicates higher densities. This may be a result of the increasing penetration of the DSS end platen pins, as noted above, or sample non-uniformity across the blocks sampled. For example, the presence of a higher quantity of iron-rich gravel particles could significantly influence the density of the sample tested.

All of the results were interpreted to assess pre-consolidation pressure based on the Strain Energy technique (Becker et al. 1987). However, this result did not provide a clear indication of pre-consolidation pressure for the DSS or oedometer tests. The oedometer test result was interpreted on the basis of the Casagrande Method (1936), indicating a pre-consolidation pf 185 kPa. This is of a similar magnitude to that inferred based on the PUSR results from DSS testing.

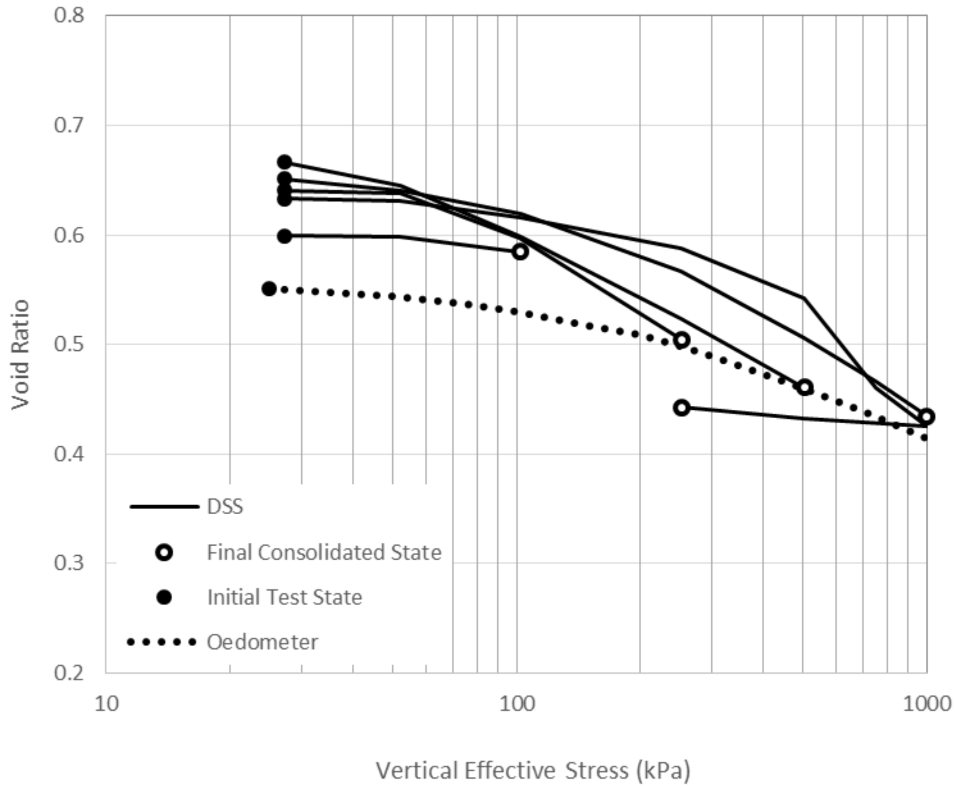

Figure 4. Consolidation Plots

3. CPTU CHARACTERIZATION

The sampling of the surficial clay described previously was undertaken coincident to a Piezocone Penetration Test (CPTu) investigation of the TSF. The CPTu investigation included testing through the crest and benches of the TSF in the area of the surficial clay sampling. A CPTu test on the lowest bench of the TSF was located approximately 40 m from the clay samples obtained. This test was interpreted to investigate if useful information on the pre-consolidation pressures and general mechanical behavior of the clay could be inferred.

The depth of tailings was inferred with the friction sleeve response of the CPTu test, by means of Friction Ratio (F_r), and the pore pressure response during penetration (u_2). These are presented in Figure 5. It is noted that a cone bearing offset of 100 mm was used in this interpretation, to account for the offset distance from the cone tip to the CPTu friction sleeve.

The results indicate a significant increase in F_r commencing at approximately 13.8 m depth, reaching a maximum at 14.1m. Similarly, u_2 begins to decrease steeply at 13.9m. On the basis of these results, the depth to natural soil was inferred to be 13.9 m. This implies that the CPTu depths corresponding to the sampled blocks likely range from 14.0 – 14.2 m. This depth is highlighted on the results in Figures 5 and 6. However, this assumes that similar surficial soils exist at the CPTu, located 40 m from where the surficial samples were taken.

CPTu-inferred values for pre-consolidation pressure and OCR are presented in Figure 6. Pre-consolidation pressure is based on the techniques proposed by Kulhawy and Mayne (1990). OCR is also presented, estimated using two methods: (i) comparison of pre-consolidation pressure to estimated vertical effective stress, (ii) comparison of the undrained strength inferred with the CPT (assuming N_{kt} = 15), to the SHANSEP parameters developed for the clay through DSS testing. These two methods are seen to agree well.

An average OCR of approximately five is inferred for the layer of surficial clay, consistent with a pre-consolidation pressure of 1000 – 1500 kPa. This high level of OCR for the material is consistent with the dilative pore pressure response of the clay during penetration through this area. This level of pre-consolidation is, however, far in excess to that inferred through the laboratory testing undertaken. If correct, this would imply significant historical loading and/or desiccation processes. While suctions of 1000 kPa within surficial clay in such an environment would be likely, whether these would manifest themselves as pre-consolidation pressure is unknown. For example, the implications of suction-induced stresses beyond the air entry value (AEV) do not appear to be fully accounted for in the soil mechanics state of practice (for example, Al-Tarhouni et al. 2011). Further assessment of the unsaturated properties of the material may be undertaken to provide additional information in this area.

It is noted that the zone of interest in this study represents a particularly difficult zone to assess based on corrected forms of CPTu resistance, owing to the rapid transition from tailings to stiffer clay. The combinations of uncertainties regarding cone-bearing offset, averaging of the friction sleeve readings, and response time of the piezometer during transition to the stiff clay make reliable interpretation of the surficial layer difficult. Further, although the CPTu was conducted relatively close to the sampling, there is no assurance that the materials are the same. Perhaps the only conclusion that can be made in the context of this study is that the CPTu results are inconclusive in providing a second indicator of pre-consolidation to the sampled materials.

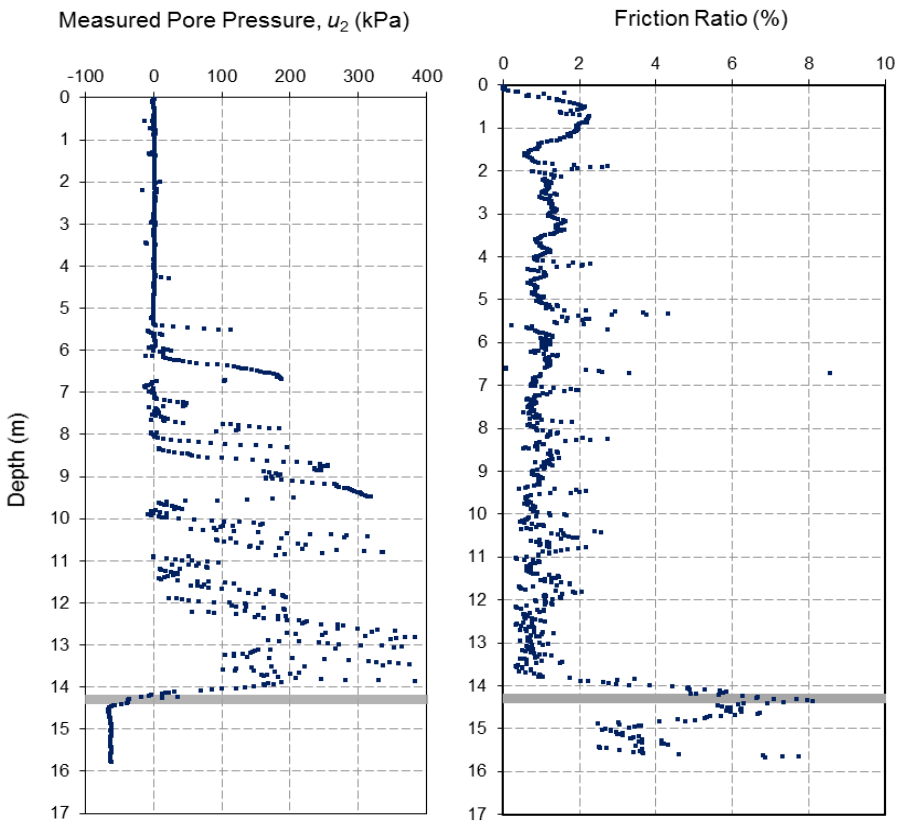

Figure 5. Pore pressure and Friction Ratio results

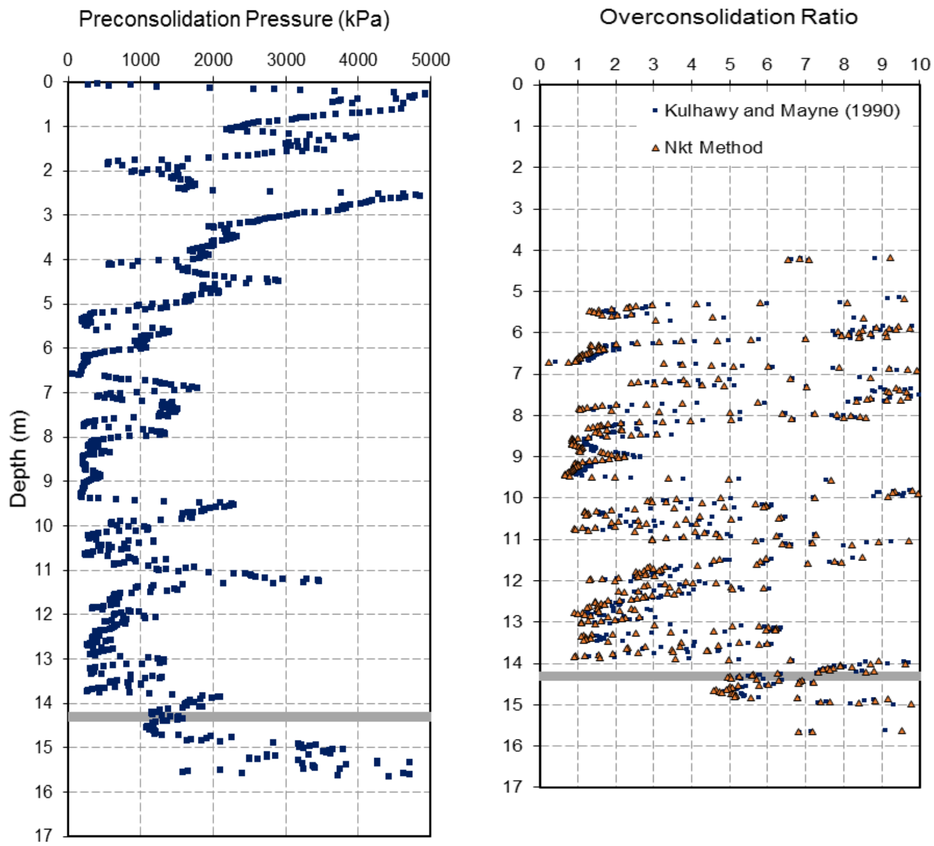

Figure 6. Pre-consolidation pressure and OCR

4. IMPLICATIONS

While there is uncertainty in the results obtained as part of this investigation, the available data is sufficient to indicate the following:

- The material will exhibit contractive behavior in shear at relatively low vertical effective stresses, despite the stiff condition in situ.
- A set of PUSRs dependent on vertical effective stress have been developed for the material, which can be input to stability analyses for the TSF.
- Two different methods of inferring pre-consolidation pressure from laboratory tests give similar results. While cognizance of the residual nature of the soil must be taken, the inferred range of approximately 140 – 190 kPa is conceivable on the basis of desiccation processes acting on surficial soil in the area.

In this particular example, the PUSR within some of the saturated layers of tailings are likely lower than the normally consolidated PUSR for the clay. Hence, when even relatively minor over-consolidation of the perimeter clay is included, this material is unlikely to control undrained stability of the perimeter embankment. However, even were the tailings stronger than the clay, the perimeter embankment slopes used for the TSF of 1V:4H are such that satisfactory stability is likely. It is further noted that saturation of significant portions of the clay material near the perimeter of the TSF is not assured. However, owing to the difficulty in accurately modelling or assessing saturation of this material below the TSF, it has been conservatively assumed to be saturated for the purposes of the analyses developed from this investigation. Should portions of the material have a sufficiently low saturation level, then PUSRs lower than effective frictional strengths would not be relevant in those areas.

5. CONCLUSIONS

A series of laboratory tests on material obtained from near-surface blocks were undertaken to assess the behavior of a stiff clay in undrained shearing. In particular, the potential for reduction in PUSR with increasing vertical effective stress was investigated. In addition, oedometer tests and nearby CPTu tests were utilized in an attempt to assess pre-consolidation pressure, where possible.

The results indicated that PUSR decreased significantly when the vertical effective stress increased from 100 to 250 kPa. Above 250 kPa, an average PUSR of 0.27 was found in the simple shear loading condition. Testing on a material unloaded to a known OCR suggested a SHANSEP m value of 0.84, and implying a pre-consolidation pressure for the other specimens of approximately 140 – 150 kPa. This result was somewhat similar with the pre-consolidation pressure of 185 kPa inferred using the Cassagrane Method on the oedometer test. However, the Strain Energy Method and empirical CPTu results did not provide similar values for pre-consolidation pressure. The source of these uncertainties is unknown, although the residual nature of the soil may be a factor.

The results provide reasonable, and probably conservative, strength inputs for analysis of the TSF perimeter embankment. Conservatism of the results is based on the implicit assumption, in the planned use of PUSR for all of the clay that the material will saturate, and in the potential indication from CPTu results that higher pre-consolidation pressures may exists at other relevant locations. Further study of the effects of desiccation processes on pre-consolidation for this material, and generally, may be useful.

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