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A foundation investigation to consider the potential for a 'Mt Polley' type slope failure

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ABSTRACT: In the wake of the Mt Polley tailings storage facility (TSF) failure, many mine owners are considering the implications of the findings by the review board (and the findings from many other major TSF failures since). Amongst other key aspects such as developing a thorough understanding of the geology, it was noted in the review that there is the potential for clayey material to transition from over-consolidated to normally consolidated due to increased loading (i.e. due to staged TSF construction), and then be subject to contractive undrained failure, rather than dilating under load. This has prompted an industry-wide review into the foundation conditions for TSFs, particularly in cases where the TSFs have been raised beyond the originally anticipated height.

This paper presents a case study where the height of a TSFs is anticipated to double from the original design height (~30 m to ~60 m). The original design work was carried out in the 1990s, with little or no laboratory testing undertaken as part of the foundation characterisation. A foundation investigation was undertaken with the primary objective of confirming the subsurface conditions and retrieving relatively undisturbed samples to support characterization through laboratory testing. The laboratory results and their implications are presented and discussed. The results of the laboratory testing indicate that the material may transition from an over-consolidated to normally consolidated under higher loads, with relatively modest undrained strengths, prompting consideration of the maximum height that the facilities can be raised without slope augmentation.

1 BACKGROUND

1.1 *Mt Polley Tailings Storage Facility (TSF) Failure*

On 4 August 2014, the tailings storage facility (TSF) at the Mt Polley mine site in Canada, failed releasing 20 Mm^3 of tailings solids and water into surrounding waterways. An independent Expert Engineering Investigation and Review Panel (the panel) completed a review of the failure to investigate the cause of the failure.

The panel identified that the failure was, amongst other items, influenced by the incorrect characterization of the foundation. The foundation failure was established to have been caused by the foundation material transitioning from an "over-consolidated" state to "normally consolidated" under the loads applied due to the construction of the perimeter embankments. The transition of the material from over-consolidated to normally consolidated and the associated strength loss had not been considered as part of the design.

This paper outlines the results from a foundation site investigation to characterize foundation parameters for input into TSF design to address this finding from the Mt Polley failure investigation.

2 LEADING PRACTICE

Professional Practice Guidelines for characterization of Dam Foundations in BC by the Association of Professional engineers and Geoscientists of British Columbia were prepared in response to the failure at Mount Polley. The process outlined can be summarized in Figure 1 and is currently considered to be leading practice. The recommendations for characterisation of dam foundations has been used as the basis for the investigation described in this paper.

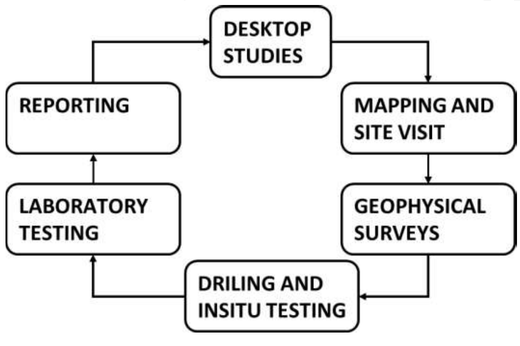

Figure 1. Process for foundation characterisation (APEGBC, 2016)

3 TSF DESIGN

3.1 *TSF Background*

The TSF is located in the Goldfields of Western Australia which is a semi-arid environment. The TSF was recommissioned after a period of care and maintenance and designed to raise the facility to ~45 m. Further extensions to the life of mine necessitated increased tailings storage requirements and the design was modified to target a maximum height of ~ 60 m. The embankments of the facility are raised using the upstream wall raise construction method shown in Figure 2.

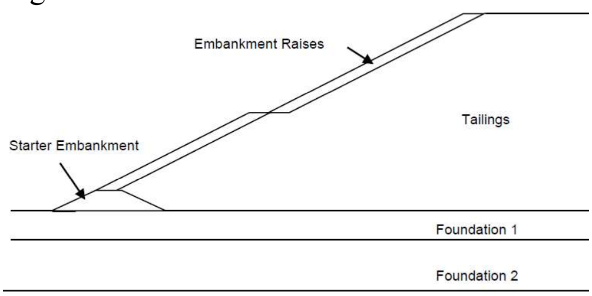

Figure 2. Schematic showing TSF construction

3.2 *Foundation Conditions*

The sub-surface beneath the TSF was original characterized as primarily low to medium plasticity clay, overlying cap-rock at varying depths. The design assumed that due to over-consolidation, the foundation clay would exhibit dilative conditions throughout depth under shear. The undrained strength of the foundation was inferred from the piezocone penetration testing (CPTu) completed in 2009 and was estimated to be between 200 to 500 kPa. The behaviour observed at the time of the investigation indicated dilative conditions.

3.3 *Strength Parameters for design*

The strength parameters input into stability analyses inferred from the CPTu data are summarized in Table 1.

Table 1. Material Strength Parameters

Setting	γ	Φ	c' or s_u	s_u/σ'_{vo}
	kN/m^3	Degrees	kPa	
Tailings	19.3	-	-	0.23-0.28
Starter embankment	21.0	35	5	-
Embankment raises	21.0	35	7	-
Foundation 1	18.0	-	200	-
Foundation 2	20.0	-	500	-

3.4 *Stability Analyses*

To evaluate the factor of safety of the embankments, stability analyses were undertaken using a limit equilibrium software. A typical output from the analysis is shown in Figure 3.

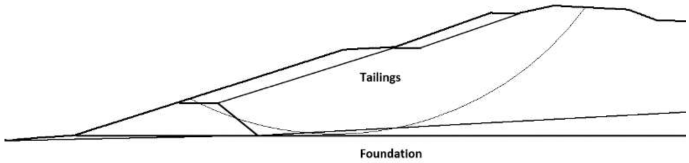

Figure 3. Typical output from the stability analyses during recommissioning studies.

The stability analyses generally indicated that the failure surfaces for the dam and the proposed recommissioned height ~30 m were not expected to pass through the foundation.

3.5 *Proposed changes to design*

Increased tailings storage requirements meant that the storage capacity at existing TSFs were re-evaluated and the decision was made to increase the maximum expected height of the embankments from ~30 to ~60 m. During the TSF operation, several CPTu investigations were completed and vibrating wire piezometers installed. Stability reviews were undertaken at regular intervals. A recent stability review indicated that the ultimate height ~60 m may not be achieved without slope stability improvement measures to address a possible failure surface through the tailings.

As part of the review of the TSF and slope improvement studies a foundation investigation was also recommended.

4 FOUNDATION CHARACTERISATION STUDIES

4.1 *Desktop Review*

The TSF is situated in a dominant relic colluvial domain (Skwarnecki, 2001), characterised by undulating low relief hills that form the margins to a broad pediment.

The typical soil profile is generally summarised as superficial materials generally being loose and shallow to around 300 mm depth, becoming dense to very dense or very stiff to hard with depth. Calcrete typically occurs in the topsoil unit with well-developed ferricrete horizons occurring in the underlying clay. Underlying bedrock horizons consist typically of banded meta-sedimentary rock or massive igneous rock, extensively weathered to depth and lateritised.

The Kanowna 1:100 000 Geological Series map (Department of Mines) describes the following geological units in the vicinity of the TSF:

- (Czc) Colluvium sand and soil, includes laterite fragments
- (Czl) Laterite (ferricrete) and reworked products
- (Czw) Weathered rock; protolith unrecognizable
- (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 colluvial and alluvial deposits (Skwarnecki, 2001). 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 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 <2 m (Skwarnecki, 2001) and range in materials from sandy clay, silts to clayey gravels.

4.2 *Geotechnical Investigation*

A total of eight boreholes (denoted S1 to S8) in pairs on four sections were completed at the TSF (one through the embankment and one at foundation level).

The subsurface conditions encountered during the drilling investigation were generally consistent with those presented in the geological series maps, and generally the subsurface encountered excluding the tailings unit included:

> CLAY (CL-CH), medium to high plasticity, with some medium grained gravel. Firm transitioning to very stiff/cemented at a depth of approximately 8 m below foundation level.

A total of 37 samples were collected during the investigation for laboratory testing. The aim of the laboratory testing was to characterise the foundation materials in terms of geotechnical properties i.e. material type, estimate consolidation properties, and identify undrained and drained strength parameters. A summary of the laboratory testing completed is provided below.

Table 2. Laboratory test program summary

Sample ID (ID/Depth bgl)	PSD*	PI**	SG***	CRS^	DSS^^	CID^^^
S1/8.0 – 8.2					•	
S2/2.0 – 2.4	•	•	•	•	•	
S3/2.0 – 2.4						•
S4/2.4 – 2.8						•
S5/6.0 – 6.3					•	
S6/2.1 – 2.6	•	•	•	•	•	

*Particle Size Distribution, **Atterberg Limit, ***Specific Gravity, ^Constant Rate of Strain, ^^ Direct Simple Shear, ^^^Consolidated Isotropically Drained Triaxial

5 LABORATORY RESULTS

5.1 *Index Testing*

The results of the index testing are shown in Table 3 and Table 4.

Table 3. Particle Size Distribution Summary

Grain Size mm	S2	S3
>4.75	<2%	<2%
<4.75 and >0.075	25-30%	25-30%
<0.075 and 0.002	25-30%	25-30%
<0.002	~45%	~45%

Table 4. Atterberg Limits and Specific Gravity

	S2	S3
Liquid Limit (LL) %	64	57
Plastic Limit (PL) %	17	16
Plasticity Index (PI)	47	41
Specific Gravity	2.77	2.72

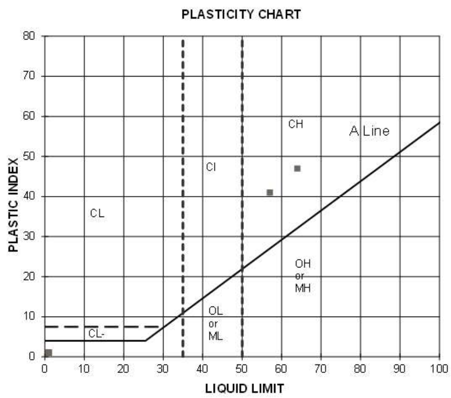

Figure 4. Results of Atterberg Limits testing shown on the Aline

The results of the index and characterization testing indicate that the samples recovered are high plasticity clay.

5.2 *Constant Rate of Strain Testing*

CRS testing was carried out to provide information on the compressibility, extent of over-consolidation of the materials and to identify pre-consolidation pressure if evident. The sample quality was assessed using the methods proposed by Tezaghi et al. (1996)

and Lunne et. al (1997). A summary of the assessment is provided in Table 5.

Table 5. Sample Quality Assessment

Sample ID	Terzaghi et al.		Lunne et al.	
	Strain (%)	SQD*	$\Delta e/e_0$	Rating
S2	1.5	B	0.038	Good
S6	2.0	B	0.048	Good

*Sample Quality Designation

The void ratio versus vertical effective stress plot is shown in Figure 5.

Examining the void ratio against vertical effective stress using the linear plot rather than the semi-log plot, the compressibility of the material appears to initially decrease as the vertical effective stress increases to a stress of approximately 200 kPa and start to increase again once this stress is overcome. The log plot appears to mask these trends that are otherwise clear examining both Figure 5 and Figure 6. The linear plot and constrained modulus show that the clay is strain hardening consistent with the soil classification based on compressibility provided by Wesley, . This behaviour is not unexpected for residual soils.

Despite this trend, which may indicate that the material does not have a clear yield pressure if examined as a conventional soil, the Becker or Work method, which estimates the yield pressure plotting strain energy and vertical effective stress in linear axis, indicates that the material yields to a stress higher than its in situ vertical stress at a pressure ranging between 350 to 450 kPa. This outcome is not evident examining the compressibility behaviour of the material due to the strain hardening observed, which would indicate a significant higher yield-pressure.

Comparison of the compressibility of the clay investigated with the two results from the Mt Polley investigation identifies that the clay in this study are significantly stiffer (see Figure 7)

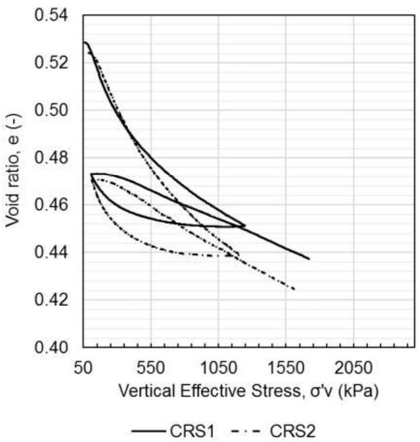

Figure 5.Void ratio versus vertical effective stress (log scale)

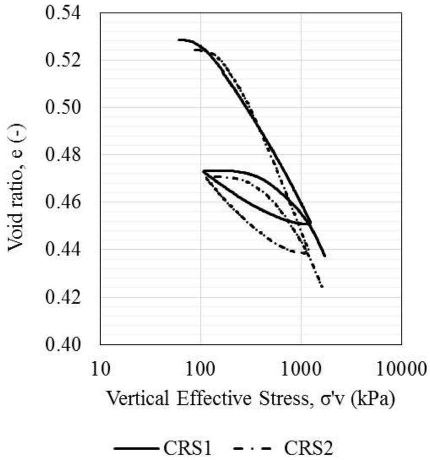

Figure 6.Void ratio versus vertical effective stress (linear scale)

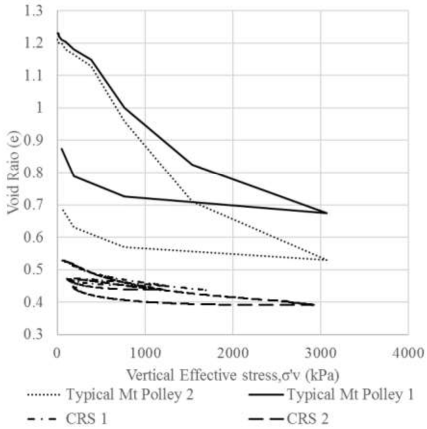

Figure 7. Comparison to Mt Polley results

5.3 *Direct Simple Shear Testing*

DSS testing was undertaken to provide information on the undrained shear strength of the samples. A summary of the results is shown in Table 4 and Figure 8.

Table 4. DSS results

Sample ID	Vertical Stress kPa	σ'vy/σ'v	su/σ'v
S1	1000	1.0	0.76
S1	1000	1.0	0.47
S2	1000	1.0	0.25
S2	400	1.43	0.33
S5	250	2.28	0.91
S6	1000	1.0	0.27
S6	500	1.14	0.31
S6	400	1.43	0.38

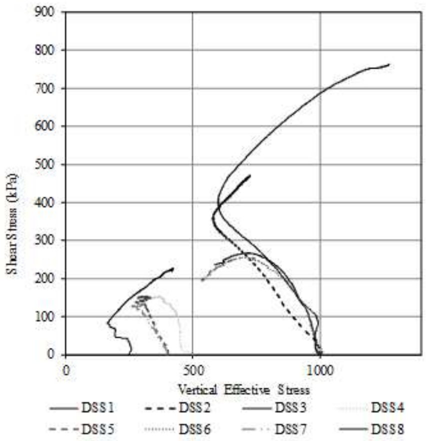

Figure 8. Stress path from DSS testing

The undrained shear strength ratio versus vertical effective stress is shown in Figure 9. The SHANSEP equation was used to infer the strength envelope for the material based on testing undertaken using the recompression technique. As recommended by Ladd 1991, the SHANSEP technique of loading and unloading at different over-consolidation ratios (OCRs) is strictly applicable to truly mechanical over-consolidated and normally consolidated soils, or when the quality of the sample is poor. However, the technique is questionable in highly weathered clay crusts in which mechanical overconsolidation does not represents the main mechanism for the overconsolidation of the clay. Therefore, acknowledging the residual nature of the clay investigated and the good quality of the tube samples, the recompression technique was preferred to infer undrained strength parameters, with tests undertaken at a stress slightly higher than the in situ stress and at the maximum stress that the clay is expected to experience during the TSF life.

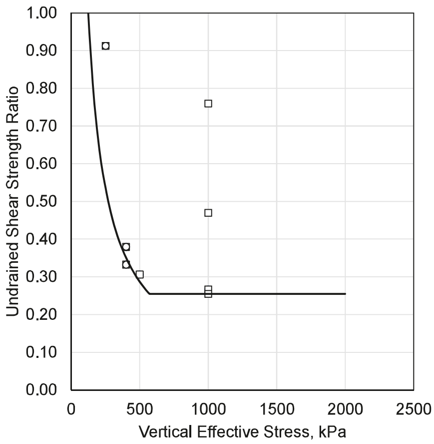

Figure 9. Strengths versus vertical effective stress (with SHANSEP relationship)

The DSS results indicate the following:

- The material exhibits a lower bound normally consolidated strength ratio of 0.255, which is typical of a CI-CH (medium to high plasticity clay) material in the DSS loading direction. This result is within the range of strength from DSS testing reported by Ladd and DeGroot 2006 for materials of similar PI (see Figure 10).

The test results suggest the following SHANSEP properties:

- s = 0.255 (normally consolidated strength ratio)
- m = 0.9 (OCR scaling exponent for undrained strength)
- σ'_{vy} = 570 kPa (yield pressure referred to in the SHANSEP framework as pre-consolidation pressure, σ'_{p})

- The OCR scaling exponent suggested by the DSS testing is consistent with typical values seen for clayey soils.

- The yield pressure inferred from the DSS testing is higher than the one based on the Work method estimated from the CRS testing. Acknowledging the limitations of the conventional approach applied to residual soils to explain strength behavior, which correlates yield pressure based on consolidation testing to strength, the DSS appears to provide a better estimate of the transitioning between the OC and NC range purely based on strength.

- Two results appear to be cemented consistent with observations during the site investigation, which has led to dilative conditions. The higher value recorded on the specimen tested at the in situ moisture content may be attributed to the material in an unsaturated state. These results have been excluded from the SHANSEP relationship but acknowledged in the slope stability model as properties of Foundation Soil 2 representative of the increasing cementation with depth.

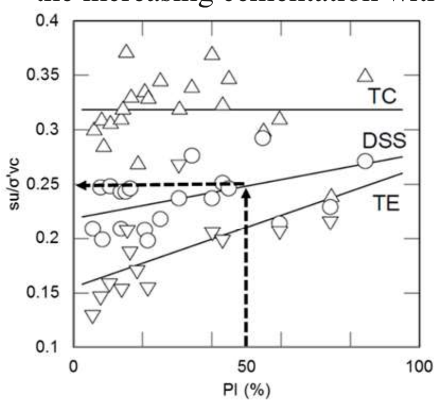

Figure 10. Expected undrained shear strength based on material PI (%) (Ladd and Degroot, 2003)

5.4 *Triaxial Testing*

Triaxial testing was completed to estimate the effective frictional strength of the over-consolidated or dilative material where undrained shear strength in DSS may overestimate the strengths of these materials. This is consistent with ANCOLD 2012 recommendations and Ladd 1991, in which the strength within the over-consolidated range is capped to the effective frictional strength as suction that develops in over consolidated soils in the laboratory is unlikely to manifest in a real failure scenario. Two consolidated isotropically drained (CID) triaxial tests were copleted to estimate the effective frictional strength of these materials. The results of the triaxial testing are shown in Table 5.

Table 5. CID results

Sample ID	Estimated *in situ*, σ'$_v$	Test, σ'$_v$	Φ	c'
	(kPa)	(kPa)	Degrees	(kPa)
S3	50	250	12.6	97
S4	50	1000	12.6	97

6 STABILITY ANALYSES

A relationship for input into stability analyses was based on the results of the laboratory testing, taking into account the *in situ* state of the material and its apparent stress history. The strength relationship accounts for the transient of the material from 'dilative/drained conditions to contractive normally consolidated conditions at higher stresses. The relationship is based on the following assumptions:

Drained strength parameters of the material of Φ = 12.6° and c'=97 kPa, when the material is in a dilative state (i.e. stresses below yield pressure)

The SHANSEP relationship as noted in Section 5.6

A summary of the strength profile is shown in Figure 12.

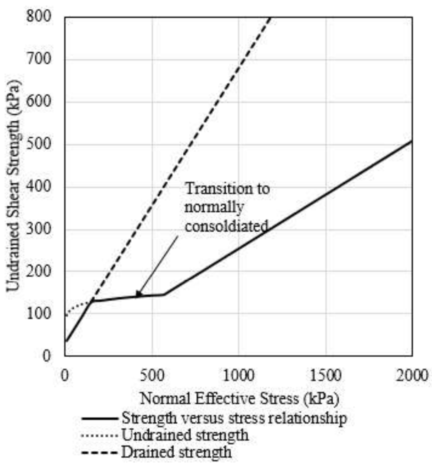

Figure 12. Strength envelope based on laboratory results

The stability was re-analysed with the updated foundation parameters. The results indicated that the global minimum fail surface now passes through the foundation whilst still above the recommended minimum values. The stability analyses indicated that slope stability measures will need to be implemented to allow the TSF to achieve the planned expanded height of ~60m maintaining the design FoS. The prferred method of slope improvement currently being investigated is buttressing of the slope. The updated foundation parameters necessitate that buttress design need to consider failure of the slope through the foundation.

7 KEY FINDINGS

Based on the field investigation and laboratory testing the following summarizes the key findings:

- The CRS results indicate that the foundation materials exhibit yield pressures between 350 and 450 kPa.
- The material appears to strain harden in terms of compressibility.
- The DSS results indicate that the material strain softens in terms of undrained shear strength with increasing vertical effective stress.
- Drained strengths will overestimate the undrained strengths for normally consolidated clay above the pre-consolidation/yield pressure.
- Updated stability analyses identified that failure the foundation will need to be considered as part of future slope stability assessments and selection of slope improvement measures.

REFERENCES

Association of Professional Engineers and Geologists of British Columbia. 2016. Site Characterisation for Dam Foundations in BC. V1.2 October, 2016

Ladd, C. (1991). Stability Evaluation during Staged Construction. 22nd Terzaghi Lecture (pp. 540-615). JGE117.

Ladd, C., & Degroot, D. (2003). Recommended Practice for Soft Ground Site Characterization. Arthur Casagrande Lecture, 12th Panamerican Conference on Soil Mechanics and Geotechnical Engineering.

Ladd, C.C. and Foote, R. (1974). A new design procedure for stability of soft clays. Journal of the Geotechnical Engineering Division. ASCE, Vol. 100, No. GT7. pp. 763-786.

Lunne, T., Berre, T., and Strandvik, S. (1997a). "Sample disturbance effects in soft low plasticity Norwegian clay." Proc. of Conference on Recent Developments in Soil and Pavement Mechanics, Rio de Janeiro, 81-102.

Skwarnecki, M. 2001. Regolith-Landform Mapping of the Kalgoorlie-Kanowna Area, Geological Survey of Western Australia

Terzaghi, K., Peck, R.B., and Mesri, G. (1996). Soil Mechanics in Engineering Practice. John Wiley and Sons

Wesley, L.D. 2013. Residual soils and the teaching of soil mechanics. Proceedings of the 18th International Conference on Soil Mechanics and Geotechnical Engineering, Paris 2013

ANCOLD, Guidelines on Tailings Dams: Planning, Design, Construction, Operation and Closure, 2012