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Prony Resources New Caledonia
Tailings Storage Facility (TSF) – KO2
Independent Geotechnical Review

Project Summary

Prony Resources is a mining and metallurgical company based in the municipalities of Mont-Dore and Yaté in the Great South of New Caledonia. The mining and process plant site, consisting of an open pit mine, hydrometallurgical plant, port, and tailings storage facility (TSF). The tailings storage facility, located in a valley, is closed by a class A dam, named KO2 dam, which has been mostly completed in 2016. The KO2 dam is located about 2 km northwest of the Goro Nickel NC (GNC) process plant, about 150 km south of Noumea, New Caledonia. The KO2 dam consists of a limonite and quarried rock berm, approximately 65 m high, with the tailings storage area within the valley lined with an LLDPE geomembrane. The tailings storage facility has an authorized storage capacity of 45 million m^3 of wet residues and is now 60% full. The instruments placed in the foundation and in the body of the dam indicate that drained conditions prevail. The Lucy project, which may start in 2021, will include the construction of a residue dewatering plant, allowing for the dry residue placement in a dry stack downstream of the dam.

Scope of Work

In summary, the scope of work is for an independent peer review to assess the liquefaction risk of the KO2 dam fill materials and foundation soils as well as the stability of the dam under long term seismic loading. Required by the local authorities (DIMENC) according to decree n°692 2021/ARR/DIMENC of March 18, 2021, the peer review must also meet the expectations and concerns of the stakeholders, namely Prony Resources, DIMENC and the local Great South communities.

Dr. Peter Robertson, through PK Robertson Inc., was requested to provide the independent peer review to Prony Resources as described in two scope of work documents, dated April 2021, for the KO2 tailings storage facility (KO2-TSF) in New Caledonia, focused on the following topics:

- Assessment of the liquefaction risk of the KO2 dam fill and foundation soils
- Assessment of the KO2 dam ability to resist possible long term seismic events

The scope of work will cover the following main areas:

- Review background documents related to KO2-TSF and Lucy Project provided by Prony Resources
- Evaluate if the current background documents are sufficient to complete the peer review objectives.
- Based on available background information, apply experience and current state of knowledge to address the following questions:

Liquefaction risk of KO2 Dam and foundations

- *Is there a risk of instability or liquefaction due to the non-homogeneous fill materials used to build the dam (layer of rock and limonite)? If yes, what is this risk?*
- *What are the materials (KO2 dam fills or foundation soils) which present a potential risk of liquefaction? Under what kind of conditions?*
- *Is the current instrumentation and monitoring network, in particular the pore pressures, sufficient and robust enough to control the liquefaction risk as defined above?*
- *Would it be possible that seepage observed in the ground, due to the wet tailings in the storage facility, have a significant impact on the liquefaction risk of the materials described above (dam fill and foundation soils)?*
- *Is there a risk of liquefaction of the soils in case of failure or clogging of one or more underdrains, installed in the foundation soils below the dam? Is it possible to implement mitigation measures to avoid this risk?*

Seismic stability of KO2 Dam

- *Is the dam capable of withstanding earthquakes with a 475-year recurrence (earthquakes with a probability of occurrence of 1/475 = 0.2 % per year)?*
- *Is the dam capable of withstanding earthquakes with a 5000-year recurrence (earthquakes with a probability of occurrence of 1/5000 = 0.02 % per year)?*
- *Is the dam capable of withstanding earthquakes with a 10 000-year recurrence (earthquakes with a probability of occurrence of 1/10 000 = 0.01 % per year)?*
- *If the dam is not able to withstand one of these earthquakes, which strategy and execution schedule should be implemented by the operator (eg. dewatering of wet residues stored downstream of KO2 dam)?*
- *Which conditions could lead to a change to undrained conditions and what could be the consequences?*

A list of the documents reviewed as part of this study is provide in the attached Appendix.

Liquefaction Definitions

To address questions related to soil liquefaction it is first important to understand the definition of liquefaction since there are two different phenomena that are described as soil liquefaction, namely: Flow (or static) Liquefaction and Cyclic (or seismic) Liquefaction). The following definitions are provided to aid in the understanding of these two phenomena.

Flow (static) Liquefaction

- Applies only to strain softening (contractive) soils (i.e., soils susceptible to undrained strength loss).
- Requires in-situ shear stresses to be greater than the residual or minimum undrained shear strength.
- Either static or cyclic loading can trigger undrained strength loss (flow liquefaction).
- For failure of a soil structure to occur, such as a slope, a sufficient volume of material must strain soften. The resulting failure can be a slide or a flow depending on the material characteristics and ground geometry. The resulting movements are due to internal causes and occur after some trigger event.
- Can occur in any strain-softening saturated (or near-saturated) soil, such as very loose coarse-grained (sand-like) deposits, very sensitive fine-grained (clay-like), and loose silt deposits.

Soils that are strain softening in undrained shear are contractive at large strains. Flow liquefaction requires loose, saturated soils that are contractive at large strains as well as sloping ground where the static shear stresses exceed the large strain undrained strength of the soil. It also requires some event that can trigger undrained behavior and strength loss to occur. The trigger events can be either static (e.g., rapid construction) or cyclic (e.g., earthquake loading).

Whether a slope or soil structure will fail, and slide will depend on the amount of strain softening soil relative to any strain hardening soil within the structure, the brittleness of the strain softening soil and the geometry of the ground. Examples of flow liquefaction failures are the Aberfan flow slide (Bishop, 1973) and the recent tailings dam failures in Brazil (Morgenstern et al., 2016; Robertson et al., 2019). In general, flow liquefaction failures are not common, however, when they occur, they typically take place rapidly with little warning and can be catastrophic. Hence, the design against flow liquefaction should be carried out cautiously.

Cyclic (seismic) Liquefaction

- Requires undrained cyclic loading with shear stress reversal during cyclic loading.
- Requires enough undrained cyclic loading to allow effective stresses to essentially reach zero, where large deformations can occur.
- Deformations during cyclic loading can accumulate to large values, but generally stabilize shortly after cyclic loading stops. The resulting movements are due to external causes and occur mainly during the cyclic loading.
- Can occur in most saturated coarse-grained (sand-like) soil (either dilative or contractive at large strains) provided that the cyclic loading is sufficiently large in magnitude and duration.

- Fine-grained (clay) soils can experience cyclic softening when the applied cyclic shear stress is close to the peak undrained shear strength. Deformations are generally small due to the cohesive strength at low effective stress. Rate effects (creep) often control deformations in cohesive soils.

Cyclic (seismic) liquefaction is generally limited to level or gently sloping ground where shear stress reversal can occur during cyclic loading. The most common form of cyclic loading to cause cyclic liquefaction is from large earthquakes where the size and duration of cyclic loading is sufficient to achieve zero effective stress. The amount and extent of deformations during cyclic loading will depend on the state (density and stress history) of the soils, the magnitude and duration of the cyclic loading and the extent to which shear stress reversal occurs. If extensive shear stress reversal occurs and the magnitude and duration of cyclic loading are sufficiently large, it is possible for the effective stresses to essentially reach zero in sand-like soils resulting in large deformations. Examples of cyclic liquefaction were common in the major earthquakes in Niigata (1964) and Christchurch (2010/11) and manifest in the form of sand boils, damaged lifelines (pipelines, etc.) lateral spreads, slumping of embankments, ground settlements, and ground surface cracks. Note that strain softening soils can also experience cyclic liquefaction depending on ground geometry.

If cyclic liquefaction occurs and drainage paths are restricted due to overlying less permeable layers, sand-like soil immediately beneath the less permeable soil can become looser due to pore water redistribution, resulting in possible subsequent flow liquefaction, given the right geometry. In cases where drainage is restricted, caution is required to allow for possible void redistribution.

Site Geology

The majority of the KO2 site is underlain by ultramafic rock classified as Peridotite. Overlying the bedrock, is a mature highly weathered and variable profile with the following characteristics in descending order:

- An upper ironstone layer that was removed under the KO2 dam.
- Transported soils (alluvial and colluvium), comprised of loose to medium dense clayey gravel.
- Residual Limonite soil that is 5 to 30m thick and exhibits relic structure and has very high in-situ void ratio and displays sensitivity.
- Alternating peridotite/saprolite that extends up to 80m
- Peridotite bedrock, with high strength.

A summary of the main geologic units is shown in Figure 1.

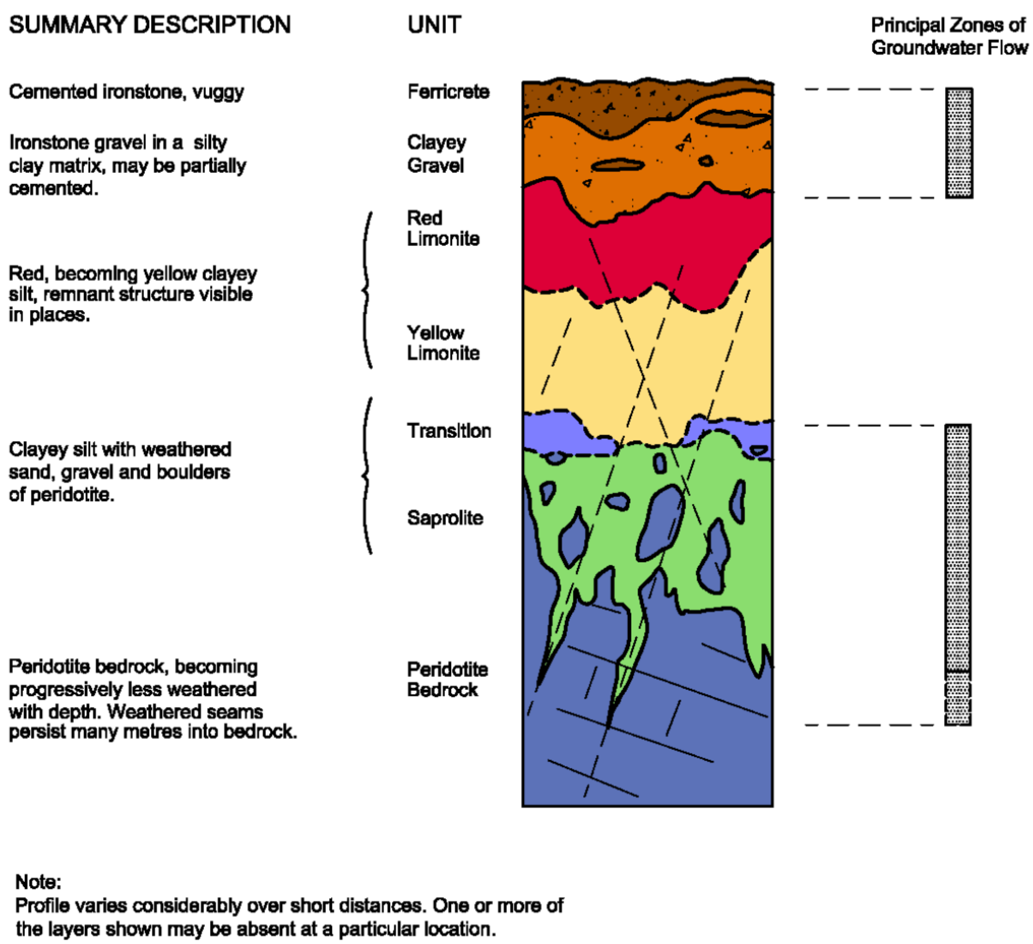

Figure 1. Summary of main geologic units (Golder, 2005)

The peridotite rock is weakly soluble, and over geological time voids form in the rock by movement of groundwater. Where these voids reach the ground surface, “dolines” form by surface collapse, and may eventually fill with loose soil or water. The area is heavily faulted, but not presently tectonically active. Dolines often form along major fault lines, as groundwater flow is concentrated along these structures.

An interpretation of the geologic profile at the KO2 dam is shown in Figure 2 that illustrates the variable and complex nature of the weathering profile.

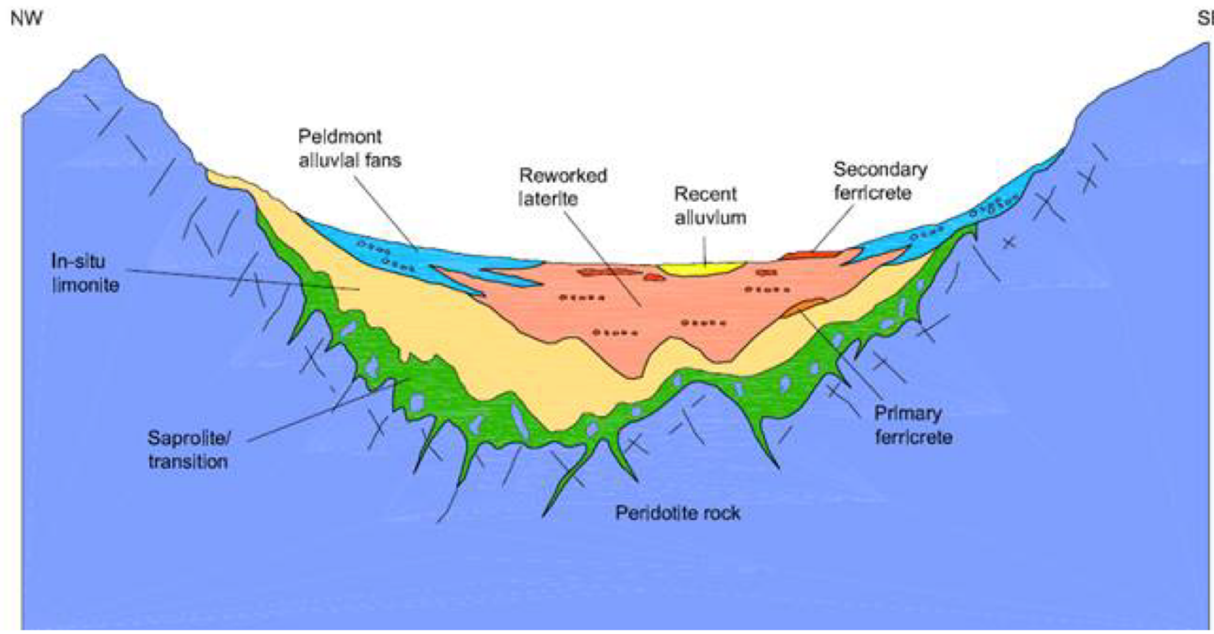

Figure 2 Interpreted geologic profile at the KO2 Dam (Golder, 2009)

Foundation soils

The main foundation units from a stability perspective are the weaker residual soil layers of mainly limonite. Geologically these are residual soils that retain some of the original bonding from the parent rock. The mineralogy of the limonite is predominately goethite (hydrated iron oxide) that is known to be a bonding agent. The goethite is composed of nanoparticles that bond together to form porous particle aggregates, that appear plate-like and have silt to sand sizes. The soil matrix of interlocking goethite crystals of varying sizes produces high void ratios and high moisture contents. The bonding creates a stiff to very stiff material at small strains, but shearing can break down the bonding and goethite matrix that can result in significant strength reduction at larger strains.

In-situ limonite is comprised of clayey to sandy silt with occasional ferruginous gravel (or iron shots). The limonite deposit ranges between 5m and 30 m in thickness and tends to grade from a reddish hematite-rich laterite (Upper Red Limonite) to a yellowish nickel-rich limonite (Lower Yellow Limonite). The Red Limonite is characterized by a moisture content of generally less than 40%, low-grade nickel (less than 1%) and iron content greater than 40%. The Yellow Limonite is characterized by a higher concentration of nickel and cobalt than the layers above it and is a very fine-grained layer with goethite as the predominant mineral, as opposed to hematite which is the major mineral in the overlying red limonite. The Yellow Limonite can have higher moisture contents up to 120%.

The geologic origin and chemical weathering of the foundations soils has produced soils with significant microstructure, mainly due to bonding both from the parent rock as well as the bonding of nanoparticles to form porous particle aggregates. This has created a highly variable deposit with significant variation in both composition and behavior.

Interpretation of available data

Foundation soils

A series of geotechnical investigations were carried out to characterize the foundation soils that included test pits, boreholes, sampling, laboratory testing and in-situ testing. The in-situ testing comprised of cone penetration testing (CPT), flat plate dilatometer testing (DMT), field vane testing (FVT) and pressuremeter testing (PMT). Some CPTs and DMTs included shear wave velocity (Vs) measurements. Much of the interpretation for this study was based on the more recent 2018 site investigation data (GEOs4D Factual Report 2018).

A site plan showing the test location for the 2018 investigation is shown in Figure 3.

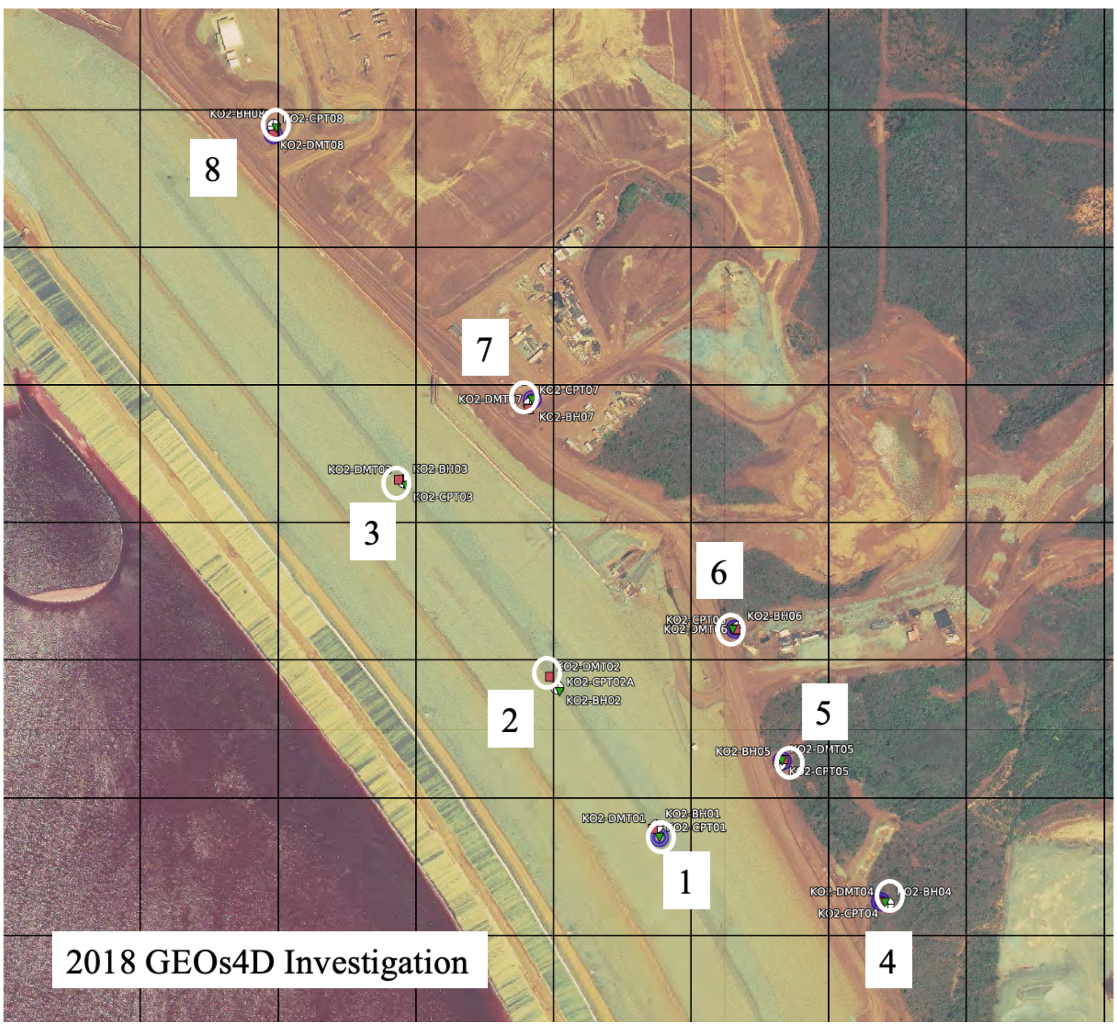

Figure 3. Site plan showing test locations for 2018 investigation (GEOs4D)

Each test location included a borehole (BH) using a small sonic drill rig, a CPT and a DMT sounding. Seismic shear wave velocity measurements (Vs) were made using a seismic CPT (SCPT) at locations 1, 5 and 7 as well as seismic DMT (SDMT) at locations 4, 6 and 8. Digital Vs data were only available for the SDMT. Vs measurements (SCPT) were not successful under the embankment at location 1. Downhole field vane shear tests (FVT) were carried out in seven BH's at locations 1 to 7.

An overlay plot of the normalized CPT parameters for the CPTs along the toe of the KO2 dam (adjusted for elevation) at locations 4, 5, 6 and 7 is shown in Figure 4 to illustrate the variability. The variability observed in the CPTs is higher in the Upper Limonite compared to the Lower Limonite. The profiles in Figure 4 also show; CPT interpreted peak undrained strength ($s_{u(p)}$), peak undrained strength ratio ($s_{u(p)}/\sigma'_{vo}$) using N_{kt} = 14 and OCR (using k_{OCR} = 0.33).

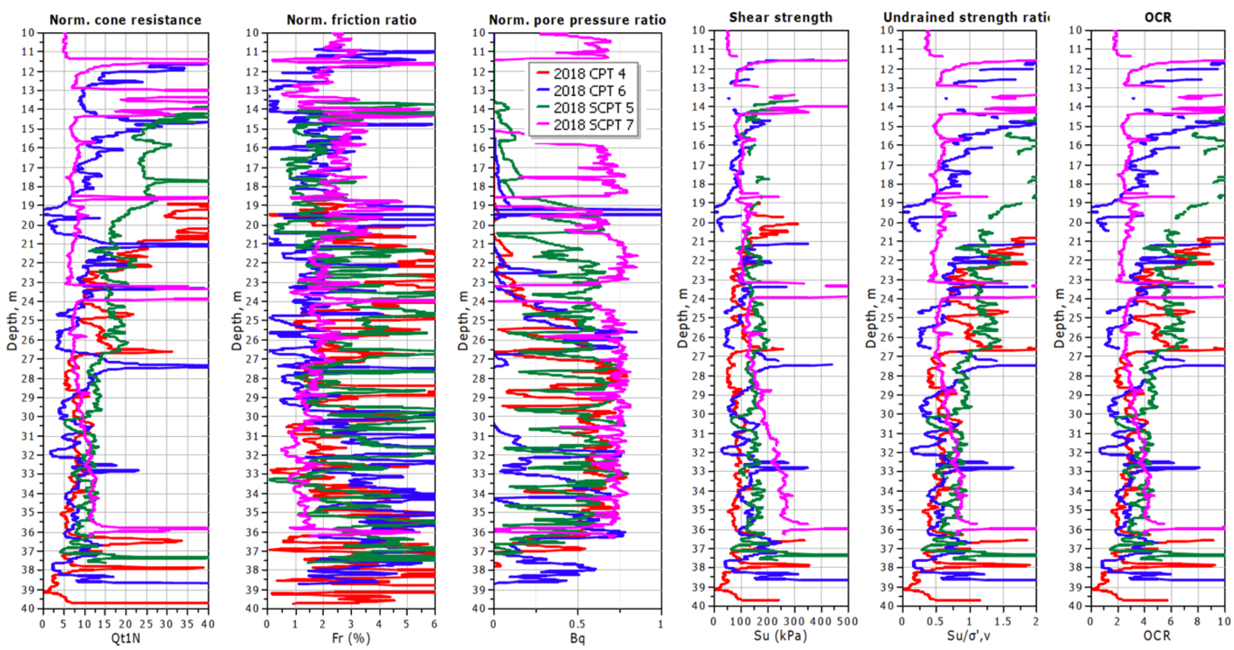

Figure 4. Overlay plot of the CPTs along the toe of the KO2 dam at locations 4, 5, 6 and 7 in terms of normalized parameters and interpreted peak undrained strength ($s_{u(p)}$), strength ratio ($s_{u(p)}/\sigma'_{vo}$) (using N_{kt} = 14) and OCR profiles (adjusted for elevation) (Depth = 10 is elevation 168.6m)

A typical CPT profile at location 5 (SCPT 5) is shown in Figures 5 (basic parameters) and 6 (normalized parameters). The frequent drops in measured CPT pore pressures are due to the longer pauses during the Vs measurements at 1m depth intervals. The dissipations at each 1m interval influences the CPT sleeve friction (fs) that is observed by frequent increases in friction ratio (R_f) and normalized friction ratio (Fr) at the 1m intervals. Figure 7 presents the normalized CPT data from SCPT 5 on the updated Soil Behavior Type (SBT_n) charts. Figure 7 identifies the microstructure (bonding) where the normalized rigidity index (K_G) exceeds 330. The very high values of excess pore pressure parameter ($U_2 = \Delta u/\sigma'_{vo}$) also indicates significant microstructure due to in-situ bonding and that the soils are also contractive at large strains.

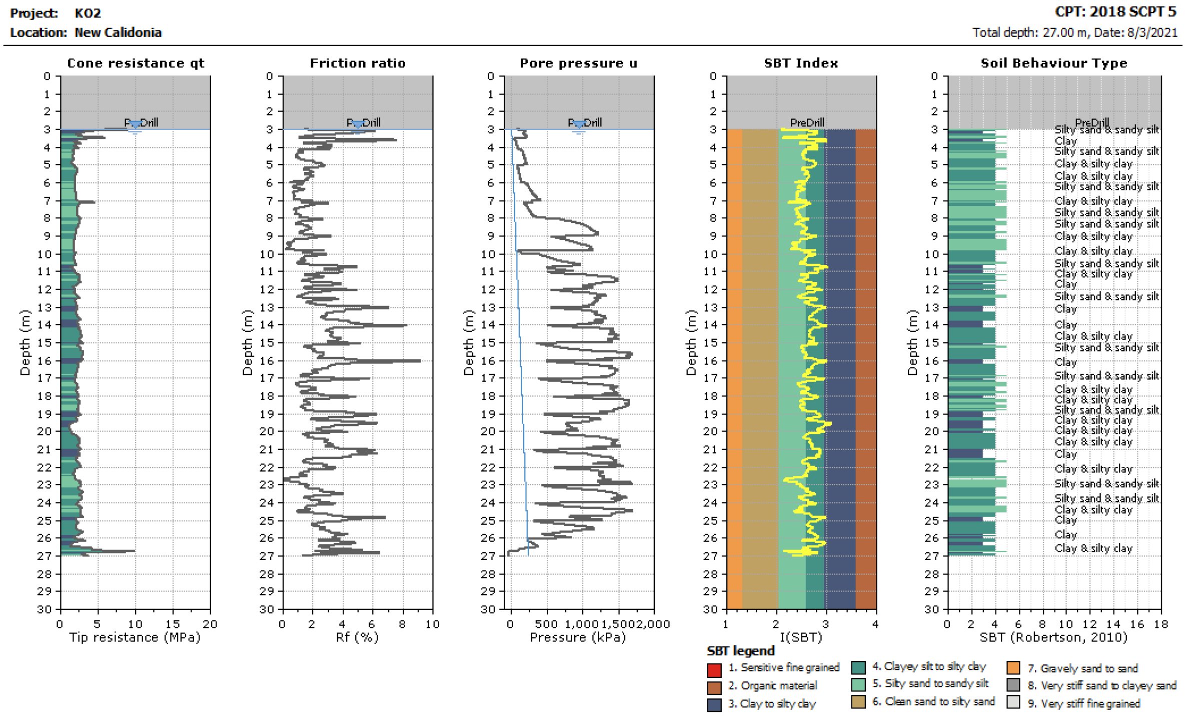

Figure 5. CPT profile at location 5 (toe of KO2 dam)

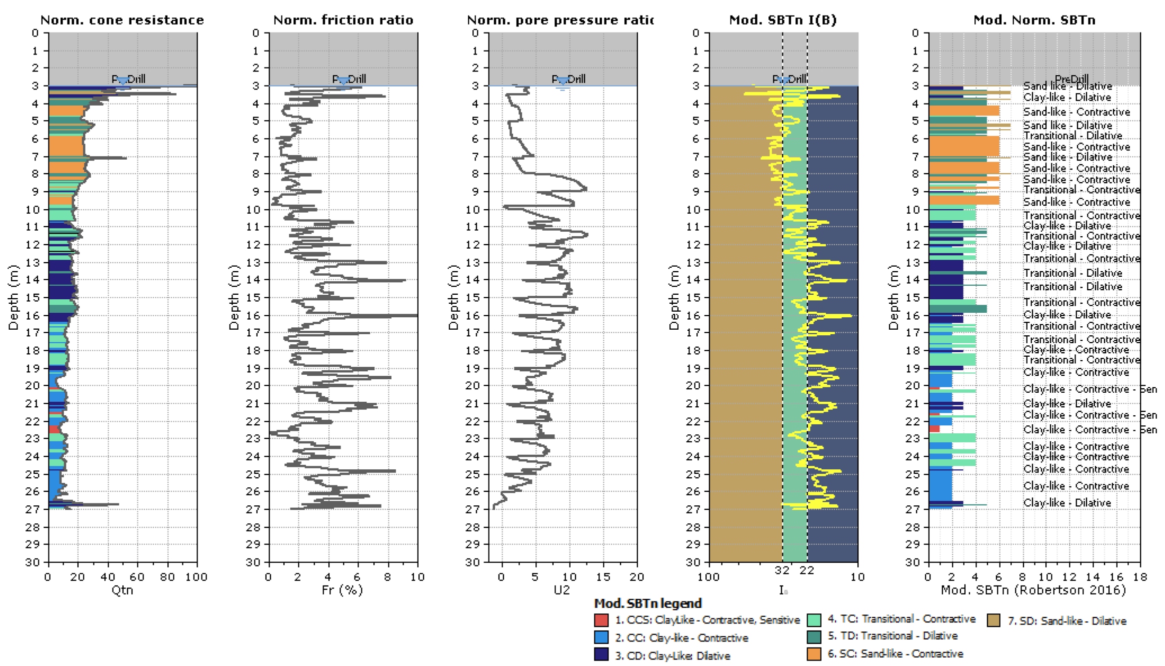

Figure 6. CPT profile in terms of normalized parameters at location 5 (toe of KO2 dam)

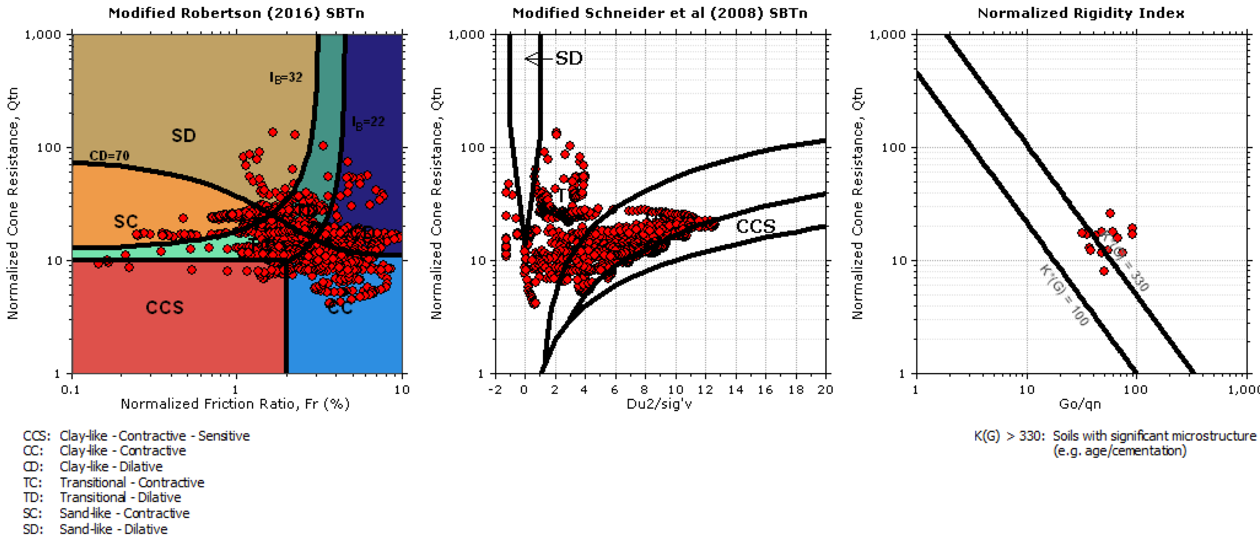

Figure 7. Normalize CPT data for SCPT 5 plotted on the updated SBTn charts (After Robertson, 2016)

Figure 8 shows the interpretation plots at SCPT 5 with the DMT and FVT data overlain for comparison.

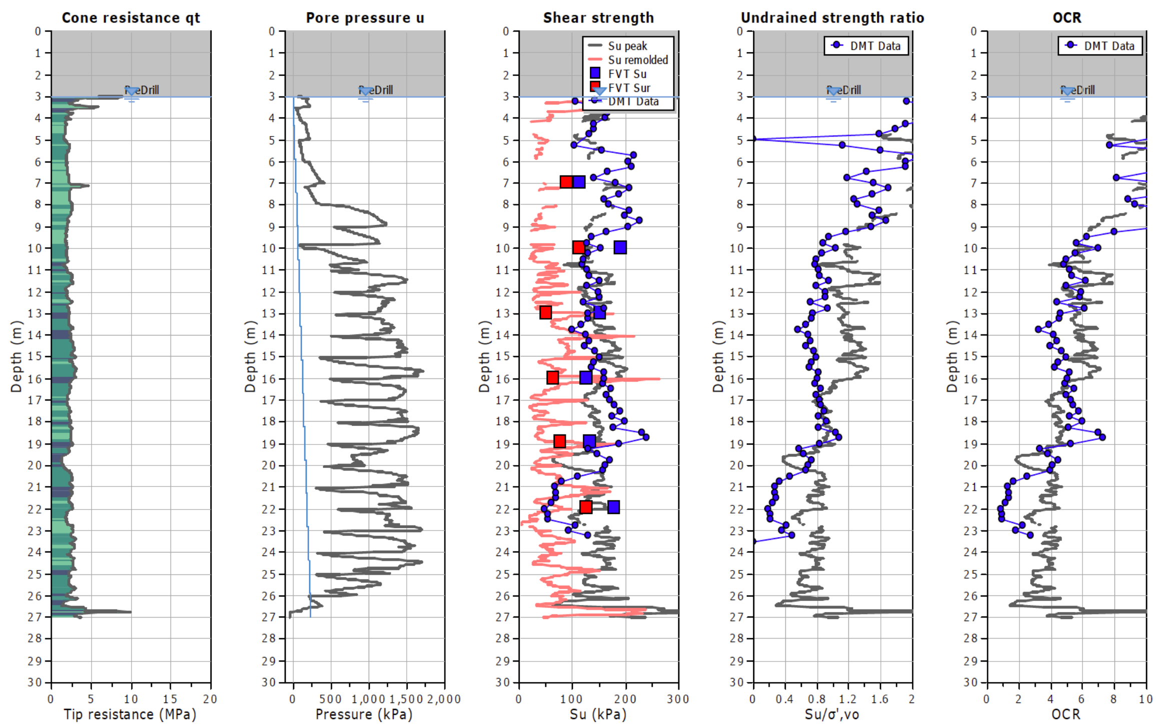

Figure 8. SCPT 5 interpretation of peak undrained strength ($s_{u(p)}$), strength ratio ($s_{u(p)}/\sigma'_{vo}$) (based on N_{kt} = 14) and OCR (at toe of KO2 dam) compared to DMT and FVT interpretation

There is generally good agreement between the CPT, FVT and DMT in terms of peak and remolded undrained shear strength as well as estimated OCR. The upper Limonite appears to extend from 3m to around 10m, based on the normalized CPT values (Figure 6). Based on the laboratory results the average unit weight of the Limonite is around 17kN/m^3, and this value was used in the CPT interpretations. The DMT interpretation used a variable unit weight where the average was close to 18.5kN/m^3. Although the specific gravity of the Limonite is high, when combined with the very high voids ratio produces a relatively normal unit weight. Both the CPT and the DMT results indicate that the majority residual Limonite soils are clay-like in their behavior. The upper Limonite is somewhat more sand-like (e.g., CPT5) but was not observed in all CPTs and boreholes.

An advantage of in-situ testing, such as the CPT and DMT, is that it can provide a large amount of data to quantify in-situ variability, both vertically and laterally. However, much of the interpretation is based on semi-empirical correlations that may not fully apply to bonded residual soils with complex mineralogy. The mineralogy of the residual soils at KO2 are very different compared to the soils that comprised the source of most in-situ test empirical correlations. Hence, there can be some uncertainty in some interpreted parameters from the in-situ tests. However, the consistency in interpretation between the different in-situ tests (CPT, DMT and FVT) provides some confidence in the general interpretation.

Several dissipation tests were carried out in each CPT to provide an estimate of drainage and consolidation characteristics. The rate of dissipation can be captured by the time it takes to dissipate 50% of the excess pore pressures (t_{50}). The t_{50} values in the CPTs along the toe of the KO2 embankment ranged from 40 to 250s, with an average of around 100s. The more rapid dissipations were generally in the Upper Limonite. These values confirm that the CPT penetration process is essentially undrained, consistent with a more clay-like behavior in these residual soils. The t_{50} values indicate an in-situ horizontal permeability of around 1x10^{-10} m/s.

Based on the available CPT data, the following approximate parameters are estimated for the Limonite outside the footprint of the KO2 dam (i.e., with no embankment fill).

No Embankment:

Upper Limonite (CPT-based)

Q_{tn} ~ 12MPa (+/- 5MPa)
F_r ~ 2% (+/- 1%)
B_q ~ 0.1 (+/- 0.1)
I_c ~ 2.6 (+/- 0.4)
OCR ~ 5.0 (+/- 2.0)
$s_{u(p)}/\sigma'_{vo}$ ~ 0.80 (+/- 0.20) N_{kt} =14
$s_{u(r)}/\sigma'_{vo}$ ~ 0.25 (+/- 0.15)
V_{s1} ~ 250m/s (+/- 50m/s)
t_{50} ~ 60s
k ~ 3x10^{-10} m/s
ϕ' ~ 40^o

Lower Limonite (CPT-based)

Q_{tn} ~ 10MPa (+/- 2MPa)
F_r ~ 3% (+/- 2%)
B_q ~ 0.7 (+/- 0.1)
I_c ~ 3.0 (+/- 0.2)
OCR ~ 3.0 (+/- 1.0)
$s_{u(p)}/\sigma'_{vo}$ ~ 0.50 (+/- 0.15) N_{kt} =14
$s_{u(r)}/\sigma'_{vo}$ ~ 0.15 (+/- 0.10)
V_{s1} ~ 250m/s (+/- 50m/s)
t_{50} ~ 100s
k ~ 1x10^{-10} m/s
ϕ' ~ 40^o

The division between the Upper and Lower Limonite was not also clear from the CPT profiles.

In general, the in-situ Limonite residual soils retain bonding from the parent rock and that the strength of the bonding is highly variable, which is common in residual soils. The in-situ Limonite is stiff and relatively strong at small strains, due to the bonding, but can become weaker at larger strains as the bonding becomes destroyed. The strength of the residual bonding creates a yield stress that make the soils behave like an over consolidated clay-like soil under low overburden stresses. Based on the CPT data, the estimated yield stress is variable due to the natural variability in the residual bonding but ranges from around 300 kPa to 700kPa depending on depth.

The behavior of the in-situ residual soils at the KO2 site are influenced by changes in overburden stress since they behave like over consolidated clay-like soils. The residual soils outside the footprint of the KO2 dam are under effective confining stresses that are smaller than the estimated yield stresses and behave like over consolidated clay-like soils, whereas the residual soils under the KO2 dam have the added weight from the embankment fill that can exceed the yield stress resulting a behavior more like a normally consolidated clay-like soil. The available CPT data confirms this variation in soil behavior.

The CPTs carried out from the downstream slope of the dam (CPT 1, 2 and 3) provide an opportunity to evaluate the behavior of the Limonite below approximately 20m of embankment fill. These CPTs indicate that the estimate OCR is reduced and is close to OCR = 1. An overlay plot of the normalized CPT parameters for the CPTs carried out on the KO2 dam (adjusted for elevation) at locations 1, 2 and 3 is shown in Figure 9. Figure 9 indicates that the 3 CPTs under the dam are generally more homogeneous, compared to those at the toe that were shown in Figure 3, although the number of tests is small. The profiles in Figure 9 also show; interpreted peak undrained strength ($s_{u(p)}$), undrained strength ratio ($s_{u(p)}/\sigma'_{vo}$) (using N_{kt} = 10) and OCR.

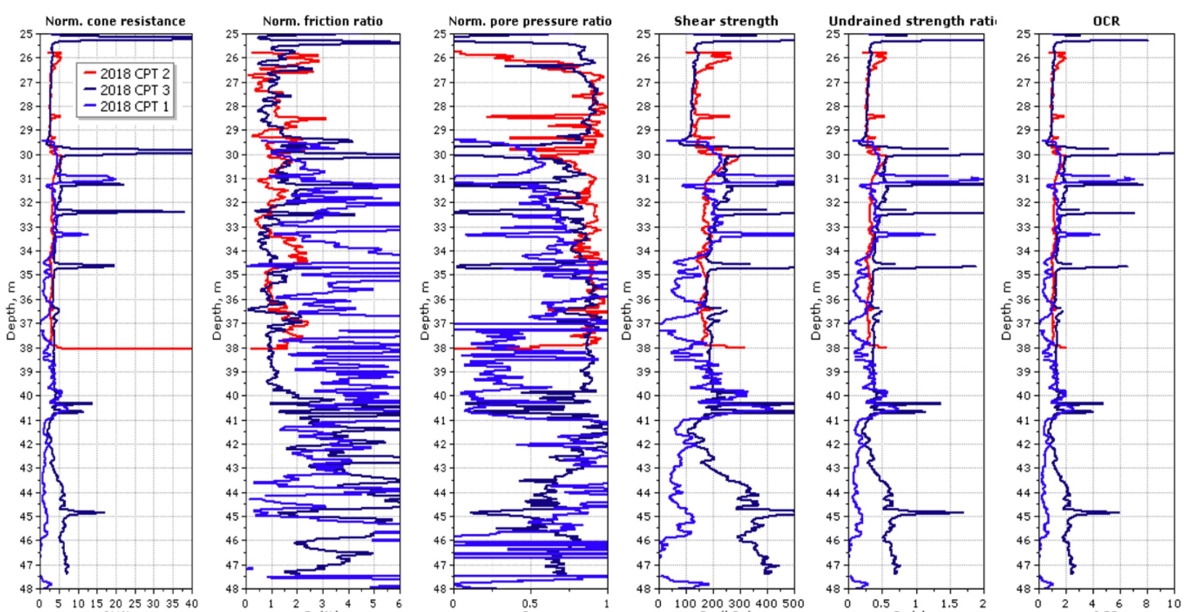

Figure 9. Overlay plot of the CPTs carried out on the KO2 dam at locations 1, 2 and 3 in terms of normalized parameters and interpreted peak undrained strength ($s_{u(p)}$), strength ratio ($s_{u(p)}/\sigma'_{vo}$) (using N_{kt} = 10) and OCR profiles (adjusted for elevation)

(Depth = 25m is elevation 166m)

Figure 10 shows the interpretation plots at CPT 2 with the DMT and FVT data overlain (CPT used N_{kt} = 10). An N_{kt} = 10 provided better comparison with both the DMT and FVT data and is consistent with the observed increased sensitivity. The remolded undrained strength from the FVT is much larger than the values estimated from the CPT (based on sleeve friction values). This may reflect a variation of undrained strength as a function of strain level since the shear strains around the CPT sleeve are very high. The high measured CPT pore pressures and low sleeve friction values support the lower remolded strength values estimated from the CPT. The FVT data are also very limited in numbers.

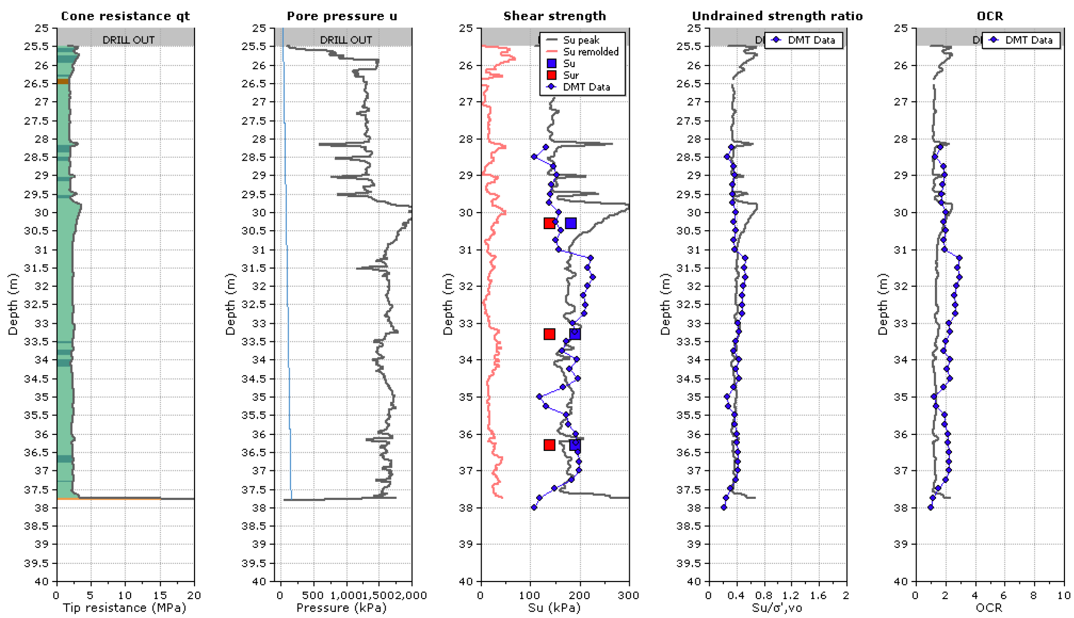

Figure 10. CPT 2 interpretation of peak undrained strength ($s_{u(p)}$), strength ratio ($s_{u(p)}/\sigma'_{vo}$) (based on N_{kt} = 10) and OCR (from KO2 dam compared to DMT and FVT interpretation)

A comparison between two nearby CPTs (CPT 03 under the dam and CPT 07 at toe of the dam) is shown in Figure 11 to illustrate the change in behavior due to the added embankment fill (at CPT 03).

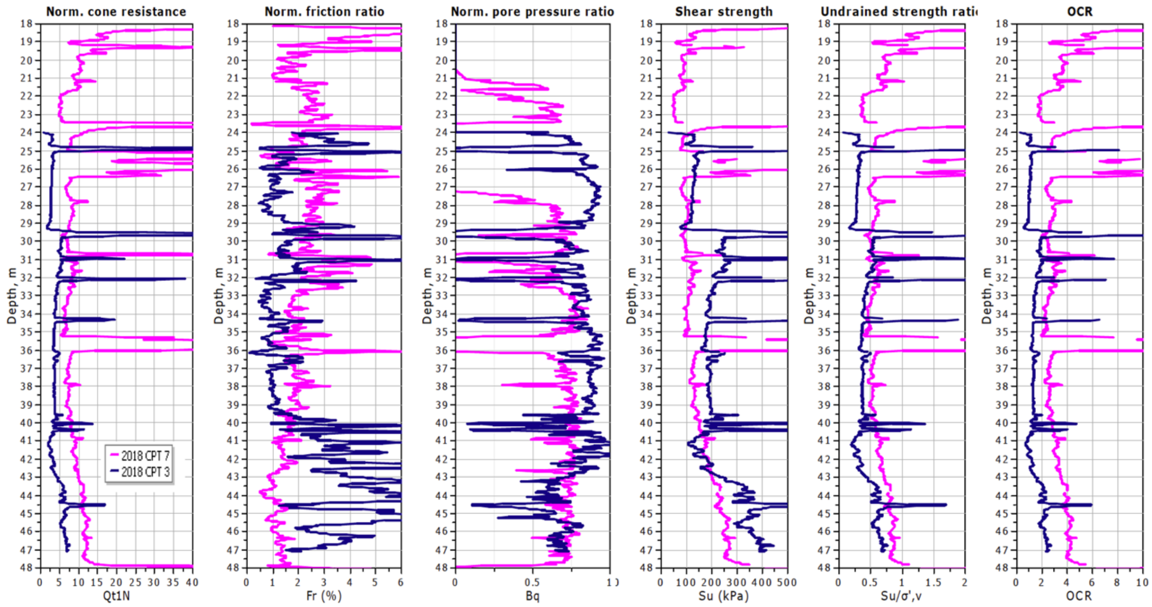

Figure 11. Comparison between two nearby CPTs:
CPT 03 below and CPT 07 at toe of dam (Depth = 18m is elevation 172.65m)

Figure 11 shows how the estimated OCR has changed from values around 3 to 4 at the toe of the dam down to 1.0 below the dam. The estimated peak undrained shear strength ($s_{u(p)}/\sigma'_{vo}$) values also show a slight decrease. The imposed stress from the embankment at these locations is around 400kPa (approximately 20m of fill) and this appears to have exceeded the average yield stress, since the resulting OCR is close to 1.0. This is also consistent with the observed large settlements under the KO2 dam.

Figure 12 compares the normalized CPT parameters for CPT 2 (below dam) and CPT 5 (at toe of dam) on the updated SBTn chart. The CPT data at the toe of the dam indicates significant microstructure (high values of $\Delta u/\sigma'_{vo}$) whereas the CPT below the dam (CPT 2) indicates no microstructure and a more sensitive behavior. This is true for all 3 CPTs carried out through the dam.

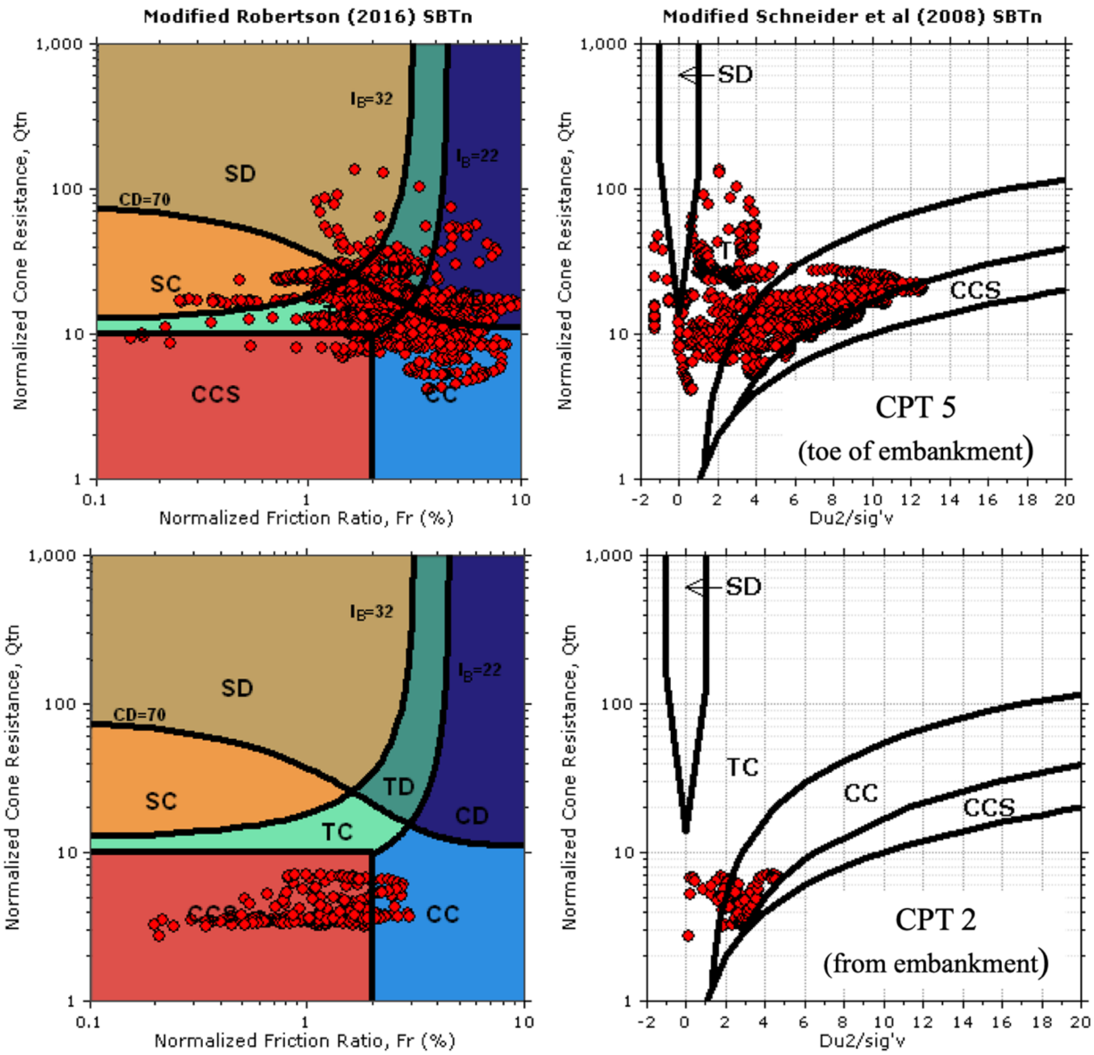

Figure 12. Normalize CPT parameters for CPT 2 (below dam) and CPT 5 (toe of dam) plotted on the updated SBTn charts
(no Vs data were measured at CPT 2)

Based on the available CPT data, the following approximate average parameters are estimated for the Limonite under the KO2 dam.

Under the KO2 embankment:

Upper and Lower Limonite (CPT-based)

Q_{tn} ~ 5MPa (+/- 2MPa)
F_r ~ 1.5% (+/- 1%)
B_q ~ 0.8 (+/- 0.1)
I_c ~ 3.2 (+/- 0.2)
OCR ~ 1.5 (+/- 0.5)
$s_{u(p)}/\sigma'_{vo}$ ~ 0.30 (+/- 0.10) N_{kt} =10
$s_{u(r)}/\sigma'_{vo}$ ~ 0.05 (+/- 0.02)
t_{50} ~ 200s (+/- 100s)

Laboratory testing was carried out at various stages of development for the KO2 dam. Much of the interpretation for this study was based on the more recent testing in 2016, 2017 and 2018 by Golder and Fugro. The laboratory testing shows the following average index parameters.

Upper Limonite:

Red brown color – Silty sand/sandy silt
Fines content (FC) = 20 to 30%
Plasticity Index (PI) ~ 11% (+/- 2%)
Liquid Limit (LL) ~ 43% (+/- 5%)
In-situ water content (w_n) ~ 37% (+/- 5%)
Specific gravity (G_s) ~ 3.5
In-situ void ratio (e) ~ 1.3 (+/- 0.2)

Lower Limonite:

Yellow color – High plastic silt
Fines content (FC) > 90%
Plasticity Index (PI) ~ 25% (+/- 10%)
Liquid Limit (LL) ~ 80% (+/- 20%)
In-situ water content (w_n) ~ 90% (+/- 10%)
Specific gravity (G_s) ~ 4.0
In-situ void ratio (e) ~ 4.5 (+/- 0.4)

Figure 13 shows a summary of some Atterberg Limit values to illustrate the range.

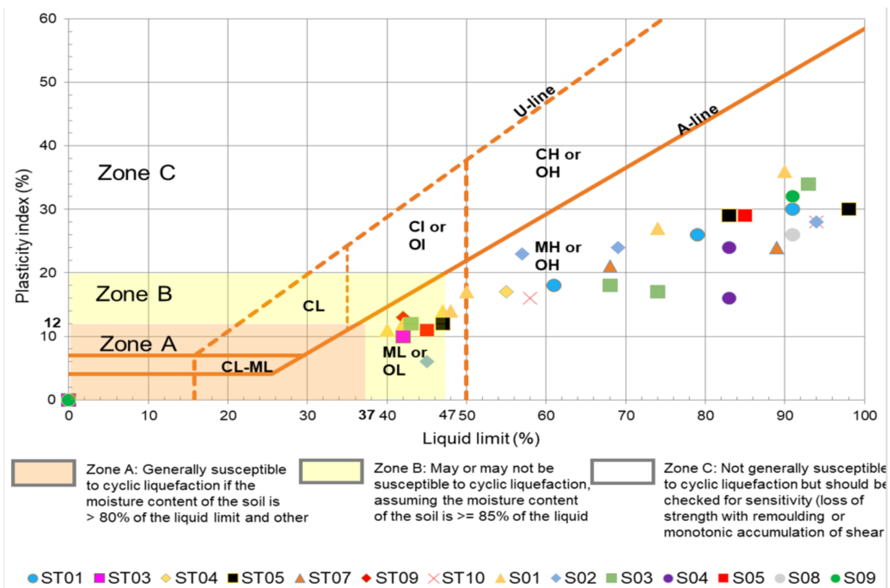

Figure 13. Summary of Atterberg Limits values for CDSS samples (Golder, 2020)

Laboratory testing on relatively undisturbed samples at low effective confining stresses indicates high but variable yield stresses consistent with variable amounts of bonding. The high yield stress combined with relatively high plasticity index (PI) values indicates that the Limonite is behaving like an over consolidated clay-like soil at low effective confining stress with resulting high peak undrained shear strength ratio ($s_{u(p)}/\sigma'_{vo}$). However, because the high yield stresses are due to bonding, when strains are large and the bonding is destroyed, the soil becomes contractive resulting in large positive pore pressures and a strain softening response in undrained shear. The reduced undrained shear strengths at large strains are consistent with the high in-situ water contents relative to the Liquid Limit. As the effective confining stress increases the apparent over consolidation ratio (OCR) decreases resulting in a smaller undrained strength ratio. When the effective overburden stress exceeds the in-situ yield stress the soil behaves like a normally consolidated clay with a peak undrained shear strength ratio close to $s_{u(p)}/\sigma'_{vo} = 0.25$. Figure 14 presents a summary of the laboratory measured peak undrained shear strength ratio ($s_{u(p)}/\sigma'_{vo}$) as a function of vertical effective confining stress (Golder, 2020).

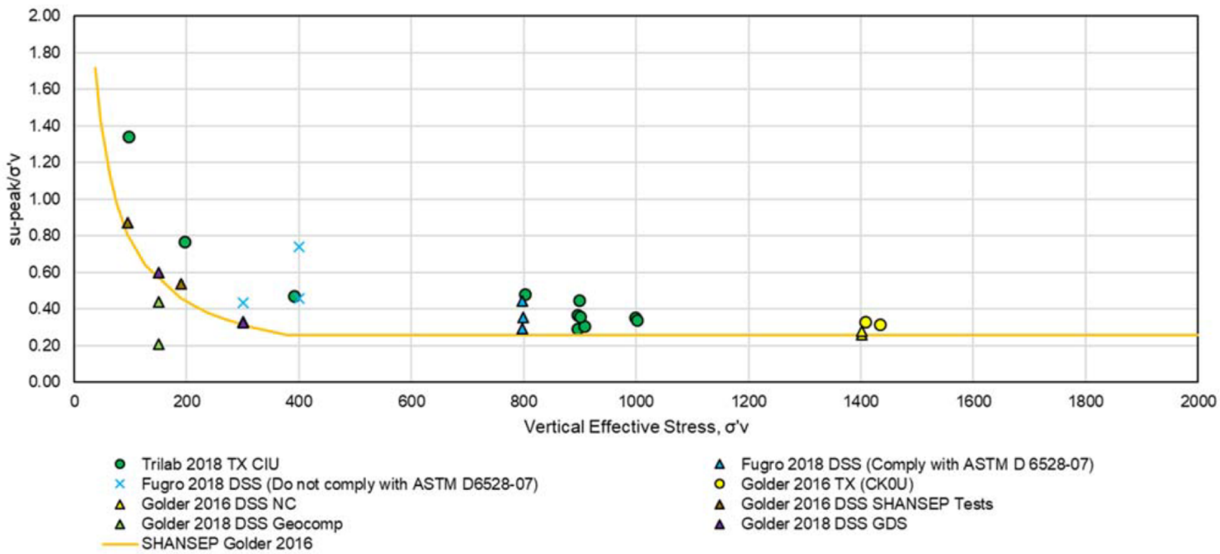

Figure 14. Summary of laboratory measured peak undrained shear strength ratio as a function of vertical effective confining stress (Golder, 2020)

As illustrated by the results in Figure 14, a limitation with laboratory data is that the small amount of test results is often not sufficient to fully quantify the in-situ variability.

The high vertical effective stresses under the KO2 dam make the Limonite behavior more like a normally consolidated clay with resulting low peak undrained strength ratio. This observation is supported by the in-situ test results. The laboratory testing at high stresses shows that the Limonite is also ductile in undrained shear and experiences strength loss (softening) with increasing strain. Most laboratory tests showed continued strength reduction (softening) with increasing strain until the test was stopped at the strain limit of the equipment. The resulting large strain residual undrained strength ratio was often around $s_{u(r)}/\sigma'_{vo} = 0.15$, although final constant residual strength had not been reached (see Figure 15). The observed ductile response of the Limonite tested at higher stresses is consistent with the relative high plasticity of these residual soils. In summary, the Limonite under the KO2 dam can be expected to behavior like a plastic normally consolidated clay with a peak undrained strength ratio ($s_{u(p)}/\sigma'_{vo}$) of around 0.25 and a ductile stress-strain response and post-peak strain softening at large strains.

The behavior of the Limonite residual soil under cyclic loading had been evaluated using cyclic direct simple shear (CDSS) tests. The results of the CDSS show that the Limonite behavior under cyclic loading is like that observed in most clay-like soils. The resistance to cyclic loading is closely linked to the magnitude of applied cyclic stress ratio (CSR) relative to the peak undrained shear strength ratio ($s_{u(p)}/\sigma'_{vo}$). As the CSR approaches the DSS $s_{u(p)}/\sigma'_{vo}$ large strains can occur like those under static loading since the CSR is approaching the peak undrained shear strength ratio. It is common to define cyclic liquefaction in laboratory test using a defined level of double amplitude strain (often 3.75%) rather than when the soil reaches zero effective stress. Hence, cyclic laboratory tests on clay-like soils can be somewhat misleading. Clay-like soils generally do not experience cyclic liquefaction (i.e., do not reach zero effective stress) but accumulate strains under high levels of CSR relative to the peak undrained strength, which is referred to as 'cyclic softening'. In the CDSS tests performed on Limonite the samples rarely reached the condition of zero effective stress, except in a few cases when a very large number of cycles were applied. Most samples experienced 'cyclic softening' at CSR values that approached 0.20. Idriss and Boulanger (2008) suggested that the resistance to cyclic softening in clay-like soils could be defined by 0.8 ($s_{u(p)}/\sigma'_{vo}$). The CDSS results are consistent with this criterion of cyclic softening.

All the CDSS tests were carried with no static shear stress bias. The Limonite under and near the toe of the KO2 dam has relatively high static shear stresses due to the overlying sloping embankment. To estimate the in-situ static shear stresses in the foundation soils would require a numerical analysis using representative soil parameters and construction history to model the in-situ stress. The laboratory and in-situ testing have shown that the Limonite is contractive at large strains and research has shown that contractive soils show a decrease resistance to cyclic loading with a static bias. If the static bias is close to the peak undrained strength ratio, a relatively small CSR from an earthquake can be sufficient to exceed the peak undrained strength ratio resulting in large strains producing some strain softening. The influence of static shear stress bias can be illustrated by two undrained triaxial compression tests on Limonite carried out under anisotropic stresses ($K_o = 0.5$) and shown in Figure 15. Note that the amount of additional shear stress to reach peak strength is very small and the subsequent strain softening continuous to very large strains. Note that the undrained shear strength ratio measured in triaxial compression is always higher than in DSS. Although no CDSS tests were done with a static bias, the results in Figure 15 illustrate that the likely outcome would show that relatively small amounts of CSR could result in some strain softening.

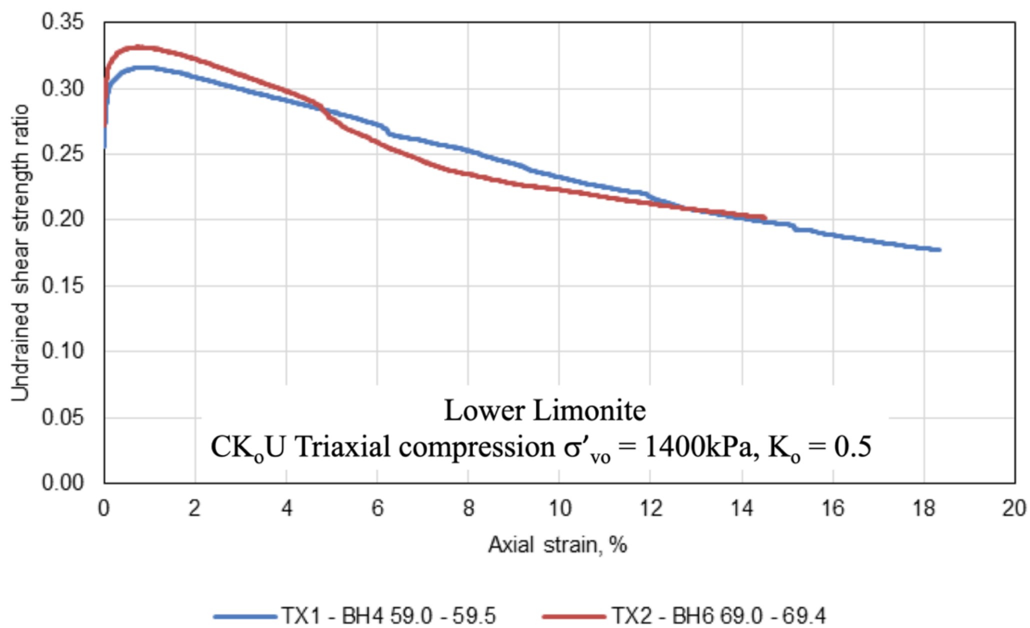

Figure 15. Two anisotropically ($K_o = 0.5$) consolidated triaxial undrained compression tests on Lower Limonite consolidated to 1400kPa

KO2 embankment Fill

The existing KO2 dam was constructed from limonite borrowed from the KO2 Residue Storage Facility (RSF) basin and rock from excavations and local quarries (some within the existing basin). The material zones used for constructing the existing KO2 dam, as described in the KO2 Residue Storage Facility Design Report by Golder (2009), are summarized below. Figure 16 shows a schematic cross section of KO2 dam showing the main fill types.

Material Zones	Name	Description
1A	Limonite Fill	Selected limonite with greater than 30% passing 75µm and a moisture content less than 60%.
2C	Selected Limonite	Selected limonite with greater than 30% passing 75µm and a moisture content less than 60%. The material is dried and compacted to 95% compaction.
2D	Drainage Gravel	Drainage layer from selected low fines rock
3A	1.5:1.5 Interlayered Rockfill and Limonite	Alternating layers of: 1.5m thick rockfill and 1.5m thick compacted Limonite
3B	1.5:4.5 Interlayered Rockfill and Limonite	Alternating layers of: 1.5m thick rockfill and 4.5m thick compacted Limonite
3C	1.5:7.5 Interlayered Rockfill and Limonite	Alternating layers of: 1.5m thick rockfill of and 7.5m thick compacted Limonite

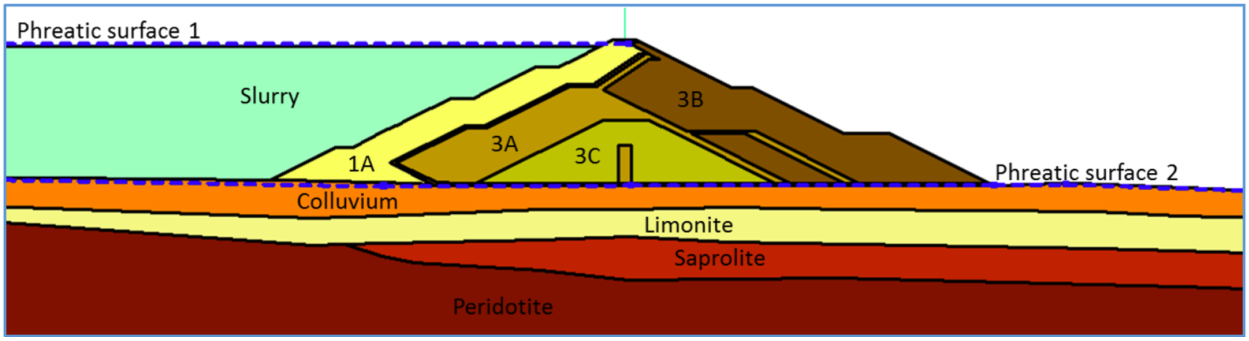

Figure 16. Idealized section of KO2 dam (CH500, Golder, 2020)

For the design of the KO2 dam Golder (2009) constructed an instrumented test berm (ITB). The only shear strength tests performed on the compacted Limonite fill were laboratory and field vane tests (FVT). It appears that no laboratory triaxial or DSS strength tests were carried out on compacted samples. The FVT tests showed relatively high values of peak undrained shear strength and when normalized by estimated vertical effective stress indicate peak undrained shear strength ratio ($s_{u(p)}/\sigma'_{vo}$) values of more than 1.0. These high values suggest a generally dilative response consistent with a compacted clay-like fill. The piezometers in the dam indicate a predominately unsaturated downstream fill and descriptions from recent boreholes performed through the embankment also indicate a generally dry material. Hence, the compacted Limonite fill in the downstream slope of the KO2 dam can be expected to behave in a drained manner, consistent with the design assumptions.

Liquefaction risk of KO2 Dam and foundations

The following section attempts to address the questions related to liquefaction risk of KO2 dam and foundations from the scope of work.

- *Is there a risk of instability or liquefaction due to the non-homogeneous fill materials used to build the dam (layer of rock and limonite)? If yes, what is this risk?*

The risk of liquefaction in the fill materials used to build the dam is very low. The available data indicates that the fill material is composed of compacted rock fill and clay-like Limonite. The data indicates that the fill is predominately unsaturated and dilative at large strains. Hence, both cyclic and flow liquefaction is unlikely. The compacted Limonite fill in the downstream slope of the KO2 dam can be expected to behave in a drained manner, consistent with the design assumptions.

- *What are the materials (KO2 dam fills or foundation soils) which present a potential risk of liquefaction? Under what kind of conditions?*

The compacted fill used to construct the KO2 dam has a very low probability of liquefaction, as described above.

The foundation soils are more complex in that the residual soils below the embankment appear to behave like a normally consolidated clay-like material whereas the residual soils in front of the toe of the embankment appear to behave more like an over consolidated clay-like material. The residual soils in front of the toe of the embankment retain some bonding from the parent rock which makes them stiff and strong at small strains but contractive and weaker at large strains. The bonding is variable due to variations in weathering resulting in a heterogeneous deposit. The weight of the embankment fill appears to have destroyed most of the bonding in the residual soils below the embankment resulting in a more homogeneous softer normally consolidated clay-like deposit. The residual soils below the embankment and in front of the embankment both have the potential for strain softening in undrained shear at large strains. The residual soils below the embankment appear to be ductile where large strains are required to achieve any significant softening.

The risk of cyclic liquefaction in the foundation soils is low, in part because the soils are generally clay-like with relatively high plasticity. The upper Limonite has a lower plasticity and more sand-like behavior, but the CPT data indicates that the upper Limonite may not be continuous over large distances.

There is a risk of flow liquefaction (i.e., strength loss) in the foundation soils. Under current static conditions these soils are behaving in a drained manner consistent with the current performance of KO2 embankment. However, since earthquake loading is very rapid, the residual soils can be expected to behave undrained during and shortly after the earthquake. Current stability analyses using shear strength ratio values from 0.25 to 0.30 in the foundation residual soils indicate calculated factor of safety values slightly larger than 1.0. This implies that the average current in-situ static shear stress ratio in the residual soils under the dam are high and maybe close to 0.20. Hence, earthquakes that induce a combined stress ratio (static plus cyclic) that would exceed the undrained peak shear strength ratio will likely trigger some undrained strain softening. Since the current factor of safety using peak undrained conditions is close to 1.0, any strain softening could result in large deformations. The high plasticity of the residual soils indicate that any strain softening will likely result in relatively ductile movements, since the rate of softening appears to be gradual (i.e., the soils do not appear to be brittle).

The level of CSR induced by an earthquake depend mainly on the peak ground acceleration (PGA) and the depth of the residual soil. Increasing depth reduces the CSR for any given PGA. Hence, the CSR in the foundation residual soil under the central part of the KO2 dam will generally be small, due to the large thickness of fill above. However, regions of foundation residual soil closer to the toe of the downstream slope have smaller thickness of fill but continue to have high static shear stresses. Hence, the foundation residual soils under the downstream slope of the dam close to the toe have the highest likelihood of experiencing some undrained softening due to an earthquake. Without additional laboratory testing and detailed numerical analyses, it is not possible to evaluate the relationship between return period of the earthquake and the likelihood and amount of softening. To evaluate the performance of the structure under the design earthquake loading would require a complex effective stress dynamic numerical analysis using representative soil parameters. This would require additional laboratory testing on high quality undisturbed samples to model the soil behavior under various input earthquake records. Based on the current data, earthquakes with a short return period (i.e., small PGA) have a low probability of triggering any softening in the foundation residual soils and earthquakes with a longer return period (i.e., high PGA) have a higher probability of triggering softening in the foundation soils.

The residual soils near the toe of the embankment fill, where the thickness of fill is either zero or small, will behave like an over consolidated clay with a higher peak undrained shear strength ratio (due to the residual bonding). Hence, these soils would require larger earthquakes to induce any softening.

The residual soils under the downstream slope of the KO2 dam have relatively high static shear stresses due to the sloping ground above. Earthquake loading occurs rapidly and will initiate undrained response in the foundation soils. It is possible that an earthquake could produce a combination of cyclic and static stresses that may exceed the peak undrained strength of the softer residual soils under the dam resulting in subsequent undrained softening. Currently there is no laboratory data on the Limonite soils at high stress with a static shear stress (i.e., CDSS with static bias) to evaluate this possibility. Based on the available data, it is prudent to assume that the likelihood of undrained softening in the residual soils under the dam is high. However, the available data shows that the Limonite generally has high plasticity and is ductile in its behavior, which implies that strains would develop slowly during undrained softening.

- *Is the current instrumentation and monitoring network, in particular the pore pressures, sufficient and robust enough to control the liquefaction risk as defined above?*

As described above, there is a risk of flow liquefaction resulting from undrained softening in the residual foundation soils under the KO2 dam during and shortly after a major earthquake. Flow liquefaction is due to undrained softening in soils and is observed by an increase in pore pressures. Since earthquakes occur rapidly, with a duration of usually less than 1 minute, and any resulting deformations also occur rapidly (i.e., during or shortly after an earthquake), usually over a period of minutes and not hours, it can be very difficult to respond to any change in instrumentation readings. Hence, instrumentation is not a robust and reliable method to monitor and control liquefaction. Instrumentation is helpful to monitor that the embankment of generally performing as designed in terms of static pore pressures and deformations but is generally not helpful for monitoring the risk of flow liquefaction, especially when triggered by an earthquake.

- *Would it be possible that seepage observed in the ground, due to the wet tailings in the storage facility, have a significant impact on the liquefaction risk of the materials described above (dam fill and foundation soils)?*

As stated above, the compacted fill in the KO2 dam is generally unsaturated and dilative at large strains with a very low risk of either flow or cyclic liquefaction. The residual foundation soils under the KO2 dam are saturated and contractive at large strains and represent a risk for flow liquefaction. Flow liquefaction can be triggered in loose contractive soils with high static shear stresses by earthquake loading (discussed above) as well as increases in piezometric pore pressures. Based on current instrumentation and monitoring, the piezometric pore pressures under the KO2 dam appear to be stable and respond to seasonal changes in precipitation. The current KO2 TSF is a fully lined facility to restrict any seepage. Any seepage from the wet tailings stored behind the KO2 dam appears to be negligible and currently appears to have little influence on the piezometric pore pressures under the dam. There is a risk that if the liner should become damaged in any significant way, for example after earthquake loading, then any sudden significant increase in seepage could result in an increase in piezometric pore pressures under the dam. To evaluate this risk in more detail would require a detailed risk assessment. This will be discussed later.

- *Is there a risk of liquefaction of the soils in case of failure or clogging of one or more underdrains, installed in the foundation soils below the dam? Is it possible to implement mitigation measures to avoid this risk?*

As state above, based on current instrumentation and monitoring, the piezometric pore pressures under the KO2 dam appear to be stable and respond to seasonal changes in precipitation. It is possible that clogging of one or more of the underdrains installed in the foundation soils below the dam could result in an increase in pore pressures. However, to evaluate the risk of this and other potential failure modes requires a detailed risk assessment.

Seismic stability of KO2 Dam

The following section attempts to address the questions related to seismic stability of KO2 dam from the scope of work.

- *Is the dam capable of withstanding earthquakes with a 475-year recurrence (earthquakes with a probability of occurrence of 1/475 = 0.2 % per year)?*
- *Is the dam capable of withstanding earthquakes with a 5000-year recurrence (earthquakes with a probability of occurrence of 1/5000 = 0.02 % per year)?*
- *Is the dam capable of withstanding earthquakes with a 10 000-year recurrence (earthquakes with a probability of occurrence of 1/10 000 = 0.01 % per year)?*

The above questions can be addresses together.

As described above, there is a risk of undrained strain softening in the saturated residual soils below the KO2 dam. However, no laboratory testing has been carried out to evaluate and quantify the potential for undrained strain softening when the foundation soils have a static shear stress bias (i.e., an existing static shear stress before cyclic loading). Hence, there is no available data on the foundation soils that would relate the size and return period of an earthquake to the likelihood of any undrained strain softening.

Golder (2018) performed a numerical analysis of the KO2 dam under both static and seismic loading. The seismic analyses applied 475 and 10,000-year recurrence earthquakes. The results of these analyses can be summarized as follows:

- The static factor of safety of the KO2 dam using peak undrained strength parameters for the saturated foundation residual soils was marginally above 1.0.
- The analyses were controlled by the assumed shear strength parameters for the saturated foundation residual soils.

- The critical mode of global shear surface and deformations is nearly horizontal as it passes through the foundation residual Limonite as captured by DSS mode of failure (see Figure 17).
- No strain softening was assumed to occur in the residual foundation soils.
- Estimated deformations were:
  - Crest settlement and translation of 1.2m and 1.5m, respectively
  - Toe heave and translation of 1.6m and 3.6m, respectively.

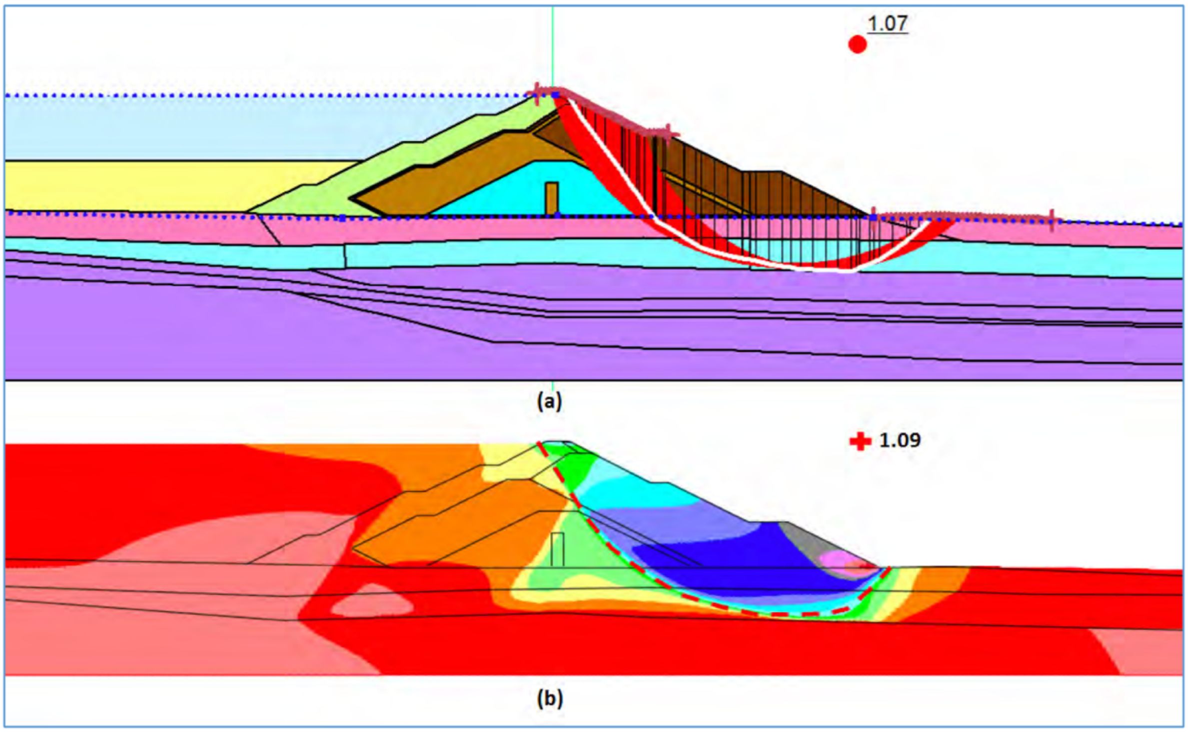

Figure 17. Summary of static stability analyses under undrained conditions performed by Golder (2018) using (a) Limit Equilibrium and (b) FLAC.

The calculated factor of safety values for the KO2 dam are below ANCOLD recommended minimum values and Golder had recommended further study to be done in 2021 that included: additional laboratory testing, additional site investigation to include geophysical testing to determine the Vs values in the fill and underlying residual soils and to analysis additional dam cross sections. A review of the preliminary results from the 2021 investigation indicate that the 2021 results are consistent with the 2018 results and confirm that the in-situ residual soils retain some bonding from the parent rock. The residual bonding tends to make the soils stiff and relatively strong at small strains but contractive at large strains as the bonding becomes destroyed resulting in a weaker shear strength under undrained conditions. High overburden stresses can also destroy the beneficial effects of the residual bonding. The strength of the residual bonding is variable resulting in variable behavior characteristics with a high level of uncertainty.

The Golder (2018) analyses assumed that no strain softening would occur in the softer residual soils below the dam. If strain softening does occur, the resulting deformations will be larger, but it is currently not possible to estimate if these deformations would be acceptable.

- *If the dam is not able to withstand one of these earthquakes, which strategy and execution schedule should be implemented by the operator (e.g., dewatering of wet residues stored downstream of KO2 dam)?*

Since the KO2 dam does not currently meet ANCOLD standards for static undrained conditions, it is assumed that some form of the Lucy Project will be carried out to ensure that the dam will meet these standards. Since the primary driver for undrained conditions is earthquake loading, which is probabilistic in nature, the schedule to improve the response of the dam to earthquake loading should also be based on probabilities which would require a quantitative risk assessment.

- *Which conditions could lead to a change to undrained conditions and what could be the consequences?*

When soils are saturated and contractive at large strains (e.g., the residual foundation soils under KO2 dam), there are various events that can initiate undrained conditions. The most common are as follows:

- Rapid construction
- Rapid cyclic loading, such as earthquakes
- Unloading, such as:
    - Rising pore pressures within the soil; and
    - Movements, such as within the foundation or due to weak layers
- Overtopping, internal erosion and/or piping that can create erosion at the toe
- Human interaction.

The main issue is whether undrained conditions will initiate any strain softening resulting in possible large deformations. The regions with conditions most likely to experience any strength reduction are the regions were saturated, contractive residual soils exist under high static shear stresses, i.e., under the downstream slope of the KO2 dam, as illustrated in Figure 17.

The event with the highest likelihood to initiate undrained conditions is rapid cyclic loading from an earthquake, although other trigger events are possible. Since earthquakes are probabilistic in nature, the likelihood of an earthquake triggering strength reduction is also probabilistic in nature. The correct way to evaluate the risk of events triggering undrained conditions and possible strain softening is by performing a detailed risk assessment.

Risk and Uncertainty

Geotechnical design of soil structures involves some uncertainty due to variability in soil composition and behavior. Ideally geotechnical design should be carried out within a risk-based framework that acknowledges and accounts for uncertainty

Risk-informed decision-making is underpinned by risk assessment, which comprises a series of steps: risk identification, risk analysis, and risk evaluation. In turn, risk-informed decision making improves and informs risk management (risk reduction) activities. Risk management includes implementation of risk reduction measures, surveillance and review, risk communication, and risk recording and reporting. Assessing risk involves consideration of both the potential consequences of an event and the likelihood of that event occurring and an adverse structural response to the event.

A risk assessment process usually has the following main steps:

- Potential Failure Modes Analysis (PFMA) to identify credible failure modes.
- Event tree analyses, on the credible failure modes, to formulate and analyze sequences of events, using event trees, leading to credible outcomes.
- Consequence analyses to determine the likely consequences related to each credible failure mode outcome that should include potential for loss of life, economic, and environmental consequences.
- Applying ALARP (As Low as Reasonable Possible) principles to evaluate mitigation measures, where appropriate.

The likelihood of various events within each event tree should be evaluated based on all the available data using appropriate methods.

A Quantitative Risk Analysis (QRA) process should be used to evaluate the credible failure modes that were identified by the PFMA. The QRA process is a risk categorization system that can be used to assign likelihood and consequence categories to potential failure modes based on existing data and available consequence estimates. QRA uses a risk matrix approach to assess individual potential failure modes as well as the total risk for a project, which can be used to prioritize dam safety activities and determine if higher level studies would be beneficial for specific potential failure modes.

Risk is represented in a general sense by the product of the likelihood of an adverse outcome occurring and the consequences of that outcome:

Risk = Likelihood x Consequence

Risk combines the probability and severity of an adverse event. To identify risk, three questions must be addressed:

1. What can happen?
2. How likely is it that it will happen?
3. If it does happen, what are the consequences?

In general terms, risk is higher when the likelihood and consequence of failure is higher, and risk is lower when the likelihood and consequence is lower. However, risk prediction is complex and uncertain, and dam failures often result from a complex series of adverse conditions, flaws, or errors in combination rather than a simple design or construction flaw.

A key element in any risk assessment is for the owner and key stakeholders to define the levels of acceptable risk. This should include potential loss of life, economic and environmental loss as well loss of reputation.

Summary

This report presents an independent review to assess the liquefaction risk of the KO2 dam fill materials and foundation soils as well as the stability of the dam under long term seismic loading.

Since the KO2 dam does not currently meet ANCOLD standards for undrained conditions, remediation measures are required to meet these standards. A suitable remediation measure would be the construction of a stabilizing buttress on the downstream slope of the dam. It is understood that this will be part of the Lucy Project that would include the construction of a residue dewatering plant, allowing for the placement of dewatered residue to support the downstream slope of the KO2 dam.

The KO2 dam is currently performing well and the need for a buttress is driven primarily by the earthquake loading criteria, since this will be the main loading condition that could initiate undrained conditions. The risks from earthquake loading are associated with the size of any event which is linked to the expected probabilistic return period of the earthquake. Small earthquakes with a shorter return period (e.g., 100-yrs. with annual probability of 10^{-2}) have a lower likelihood of triggering undrained conditions, whereas large earthquakes with a longer return period (e.g., 10,000-yrs. with annual probability of 10^{-4}) have a higher likelihood of triggering undrained conditions. In general terms, a higher likelihood of undrained conditions leading to potential strain softening in the residual foundation soils is associated with lower probability large earthquake events. Since remediation in the form of a buttress is planned, a detailed risk assessment should guide the decision process related to scheduling of the buttress as well as identify and quantify any other potential failure modes.

It is understood that there is currently a tailings review board (TRB) for the KO2 dam and that they have meet three times (2018, 2019 and 2020). The TRB had the opportunity to meet and discuss with the various engineers involved in the project, visit the site twice and receive presentations that likely contained more information than was provided for this review. It appears that the TRB has acknowledged the need for remediation of the KO2 dam and support the general concept of the Lucy Project.

I trust this review has achieved the stated objectives.

Yours truly,

P.K. Robertson, Ph.D.

Appendix:
List of documents reviewed

KO2 studies

117301008-003-R-Rev0 KO4 Bore Hole 2011

137301006-017-R-Rev1-1000-Logs KO4

147301002-038-M-Rev0-KWRSF Waste Dump

147301002-036-M-Rev0-1000-Borrow pit and material reuse

147301002-052-M-Rev1-GI09_Piezometers

1781015-012-R-Rev2-DWP2 Geotech Factual Report

253-097301001r-KWRSF Stage2 Instruments Installation-Rev0

382-097301001pm-South Outlet Investigations Results-Rev 0

481-097301001-R-Rev1-Installation Instruments Complémentaires

B-170 - Seismic Hazard Evaluation

B-200 Seismic induced liquefaction

904-03639030pm-pc-KWRSF Berm Seismic Analysis-Rev 1

474-03639030-R-pc-KWRSF Geotech and Hydro Report Rev 0

692-03639030r-pc-KWRSF Berm Seismic Analysis Rev A1

1250-03639030-r-KWRSF Design Report - Rev 1

504-03639030r-KWRSF Design Criteria-Ver 12

711-03639030r-pc-KWRSF Geological Report-Draft 1

1263-03639030pm-bd--Rev 0 KWRSF CE Responses Weak Layer Liquefaction Potential

1458-03639030pm-Limonite Piping-Rev 0

1506-03639030r-sc-KWRSF Site Inspection-Rev 0

1511-03639030r-bd-KWRSF Anomalous Settlement-Rev 0

1554-03639030pm-HET Spec Rev O

1569-03639030P-Paris presentation GSF 25 September 2008

1573-03639030pm-KWRSF Liquefaction Potential Sandy Material Rev 0

1590-03639030r-Geotechnical Close Out Report Rev 0

015-097301001r-fj-KWRWSF Const Audit-Rev A

030-097301001pm-KWRSF Laboratory Audit-Rev 0

032-097301001pm-KWRSF Results of Limonite Erosion Testing-Rev 0

046-097301001r-fj-KWRSF Berm Foundation Settlement rev0

049-097301001r-fj-KWRSF Reconnaissances Carriere Laterites Nord-rev1

073-097301001r-fj-Log Book KWRSF-rev0

097301001 T1040 Nov 09 lab audit

101-097301001R-KWE West Construction Audit-Rev 0

127-097301001pm-Rev 0-July 2010 Site Visit Record

166-097301001pm-KWRSF Properties of Goro Limonites - Summary for Information -Rev0

217-097301001pm-KWRSF Test Pit Old Chromium Mine-Rev0

232-097301001r-KWRSF Old Chromium Mine-rev0

248 - 097301001-R-Rev1-BackAnalysis_KWRSF Settlement

259-097301001pm-KWRSF Use of Limonite from Mine-Rev 0

262-097301001pm-KWRSF Compaction Trial with Limonite from Mine Dump V4-Rev 0

263-097631001pm-Berm Stability vs Piezometer-Rev 0

265-097301001pm-KWRSF Use of Limonite from Mine-Zone TOP-Rev 0

280-097631001pm-Berm Stability vs Piezometer-Rev 0

286-097301001pm-KWRSF Mine Limonite for 1A material -Rev1

288-097631001pm-Rev 0-Recommended Approach to Calibrating Settlement Model

298-097301001-Conduit Settlement Analysis-Rev 1

299-097301001r-KWRSF Rapport Première Mise en Eau-Rev2

305-097301001pm-KWRSF Use of Limonite from Mine_New test-Rev 0

476-097301001-R-Rev 0-Construction Audit December 2015

497-097631001pm-Pore water pressure Rise in Berm Zone 1A-Rev 0

502-097301001pm-Berm stability due to pwp rises-Rev 1

004-1648818-Rev0 - Logbook

005-1648818R-Laboratory test results-Rev B

007-1648818R-Geotechnical Characterisation Foundation Clay-Rev 0

009-1648818R-Limonite lab testing-Rev 0

2017 KW BERM COMPLETION REPORT MK 1 - complete

001-1783848pm-Closing drains-Rev 0

002-1783848pm-KO2 berm-Rev 1

005-1783848R-Stability assessment of KO2 berm-Rev 5

006-1783848pm-Stability-Response to Email-Rev 0

009-1783848pm-External review reports-Rev 1

010-1783848pm-Liner under drain pipe loads-Rev 1

011-19118422L-KO2 water storage risk-Rev 0

012-1783848pm-Undrained vs Drained-Rev 3

013-1783848R-Upstream stability analysis of Berm-Rev 4

014-1783848pm-Dynamic deformation analysis parameters-Rev 0

016-1783848-R-Trigger levels for KO2 berm-Rev 3

017-1783848R-Deformation analysis of Berm-Rev 0

018-1783848L-KO2 Storage Effects-Rev 1

021-1783848mn-Berm stability discussion-Rev 0

022-1783848pm-Response to comments from Vale on dynamic deformation -Rev 0

023-1783848R-Deformation analysis of Berm (2005 parameters) - Rev 1

005-19118422R-Initial KO2 dam break-Rev 0

008-19118422-R-Rev1 - KO2 dam break

017-19118422-R-Review of Berm testing data-Rev 2

020-19118422R-Site visit 191014-Rev 0

023-19118422R-Reassessment of Berm stability using recent data-Rev 2

024-19118422R-Residue modelling-Rev 7

028-19118422-R-Rev 1 -KO2 dam break Stage3

030-19118422-R-KO2 DSI-Rev 0

042-19118422pm-Berm stability question responses-Rev 0

050-19118422-TARP Berm-Rev 3

050-19118422-TARP Drains-Rev 4

054-19118422R-Background to TARP TRIGGER LEVELS-Rev 1

059-19118422pm-Site investigation and instrumentation plan-Rev 2

061-19118422pm-Comparison of ANCOLD guidelines-Rev 2

066-19118422pm-Berm stability background-Rev 0

067-19118422pm-Responses to Vale reviewers queries-Rev 0

071-19118422pm-Review of CDSS testing- Rev 0

072-19118422pm-Summary of CDSS testing review-Rev 1

076-19118422PP-CTPBH close-out report-Rev 0

078-19118422-TM- Stage3 DBA Downstream flood details-Rev 0

1671587-043-R-Rev0-Rapport de 2nde phase de mise en eau

Rapport d'essai LG090-BR4048-230616+annexes TX_2

BRGM/RP-54935-FR - Évaluation probabiliste de l'aléa sismique de la Nouvelle-Calédonie

BRGM/RC-64881-FR - Probabilistic Seismic Hazard Assessment for KO2 and KO4 dams in Goro (New Caledonia)

BRGMRC71199FR

1781012-009-R-Rev2-KO2 Geotech Factual Report

FG104-2 - V2 - T1997 - Lucy pied de berme FEL 3 - VALE

CR investigation géotechnique - Lucy FEL 3 - Bassin KO2 - VALE NC V1

Rapport d interpretation des essais geotech in situ projet Lucy_V1

H354600-1000-22A-030-0003_AB KO2 Berm Stability Review

H354600-1000-22A-030-0004 KO2 Berm Stability Review - Back analysis (Part II)

H354600-1000-22A-030-0005_KO2 Berm Stability Review - Seismic Analysis and Decant Pond Levels

GoroITRB Report112252018(rev.0303312019)

GoroTRBREportNo.202242020Final

TRBMtg3FinalSigned

BR701-00034 TDR M20001 Technical Review of Goro Tailings Management

LUCY 1.0 Feasibility Study

H350607-1000-220-030-0001 - Geotechnical Materials Testing

H350607-1000-220-030-0003 - Dewatered Residue Laboratory Testing

H350607-1000-220-030-0004 - Dilute Residue Laboratory Testing

H350607-1000-22A-030-0001 - Laboratory Results - Interpretive - Residue Dry

H350607-1000-22A-030-0002 - Laboratory Results - Interpretive - Residue Slurry

H350607-1000-220-210-0001 - KO2 - Design Basis

H350607-1000-229-230-0001 - KO2 Consolidation Modelling & Settlement Analysis Report

H350607-1000-229-230-0002 - KO2 Static Slope Stability Analysis Report

H350607-1000-229-230-0004 - KO2 Stack Seismic Stability & Liquefaction Assessment

H350607-1000-229-272-0001 - Layout - Project LUCY - KO2 Geotechnical Investigation - Borehole Location Plan

H350607-1000-22A-242-0002 - Technical Specification - Geotechnical Testing In KO2

H350607-1000-22A-242-0003 - Geotechnical Site Investigation

H350607-1100-229-272-0002 - Project LUCY - K02 - Residue Foundation - In-Situ Geotechnical Test Plan

LUCY 1.0 Detailed Design

H354600-1000-220-202-0003 - PROJECT LUCY - KO2 DRY STACK GEOTECHNICAL ANALYSIS

H354600-1000-220-230-0006 - INTERPRETIVE GEOTECHNICAL REPORT FOR KO2 DOWNSTREAM AREA

H354600-1000-220-230-0001 -PROJECT LUCY - REPORT - KO2 DRY STACK DESIGN REPORT

H354600-1000-229-270-0008 - 140 - TAILINGS DISPOSAL - KO2 - AS-CONSTRUCTED GEOT GENERAL ARRANGEMENT

H354600-1000-229-270-0009 – 140 - KO2 - AS-CONSTRUCTED GEOTECHNICAL UPSTREAM STACK - PLAN

H354600-1000-229-270-0010 - 140 - KO2 - AS-CONSTRUCTED GEOTECHNICAL DOWNSTREAM STACK - PLAN

H354600-1000-229-270-0011 - 140 - KO2 - AS-CONSTRUCTED GEOTECHNICAL CWP - PLAN

H354600-1000-229-273-0016 - 140 - KO2 UPSTREAM AND DOWNSTREAM STACK - GEOLOGICAL SECTION A - SECTIONS

H354600-1000-229-273-0017 - 140 - KO2 UPSTREAM STACK - GEOLOGICAL SECTION B - SECTIONS

H354600-1000-229-273-0018 - 140 - KO2 UPSTREAM STACK - GEOLOGICAL SECTION C - SECTIONS

H354600-1000-229-273-0019 - 140 - KO2 UPSTREAM STACK - GEOLOGICAL SECTION D - SECTIONS

H354600-1000-229-273-0020 - 140 - KO2 UPSTREAM STACK - GEOLOGICAL SECTION E - SECTIONS

H354600-1000-229-273-0021 - 140 - KO2 UPSTREAM STACK - GEOLOGICAL SECTION F - SECTIONS

H354600-1000-229-273-0022 - 140 - KO2 UPSTREAM STACK - GEOLOGICAL SECTION G - SECTIONS

H354600-1000-229-273-0023 - 140 - KO2 UPSTREAM STACK - GEOLOGICAL SECTION H - SECTIONS

H354600-1000-229-273-0024 - 140 - KO2 DOWNSTREAM STACK - GEOLOGICAL SECTION J - SECTIONS

H354600-1000-229-273-0026 - 140 - KO2 DOWNSTREAM STACK - GEOLOGICAL SECTION K - SECTIONS

H354600-1000-229-273-0027 - 140 - KO2 CWP - GEOLOGICAL SECTION L, M, N AND P - SECTIONS

H354600-1000-229-273-0028 - 140 - KO2 CWP - GEOLOGICAL SECTION Q - SECTIONS

H354600-1000-229-202-0003 - PROJECT LUCY - REPORT - KO2 DRY STACK SLOPE STABILITY ANALYSIS

LUCY 2.0 Prefeasibility Design

H354600-1000-229-202-2001 - PROJECT LUCY 2.0 - KO2 DRY STACK SLOPE STABILITY ANALYSIS

H354600-1000-220-230-2001 - PROJECT LUCY 2.0 - KO2 DRY STACK DESIGN REPORT

LUCY 2.0 Detailed Design

H354600-1100-229-202-2001 - PROJECT LUCY 2.0 - KO2 DOWNSTREAM STACK STAGE 1 - SEEPAGE ANALYSIS

H354600-1000-220-230-2101 - PROJECT LUCY 2.0 - KO2 DRY STACK DESIGN REPORT

Buttress and Lucy design studies - Mecater

MC-20-197-VAL-04-R01-C - Available geotechnical data and additional geotechnical investigation requested

MC-20-197-VAL-04-R02-F - KO2 stability assessement and buttress design

MC_20_197_VAL_04_R05-B - Geological model

MC_20_197_VAL_04_R03-D - Buttress design report

MC-21-102-VAL-01-R01-C - Lucy drystack stability assessment

MC-21-102-VAL-01-R02-E - Lucy design report

Raw data

KO2 downstream - Pressiometer tests data

KO2 downstream - Vane shear tests data

KO2 downstream - SDMT data

KO2 basin - Tailings - CPT data

KO2 basin - Tailings - DMT data

KO2 basin - Tailings - T-bar data

KO2 basin - Tailings - Vane shear tests data

KO2 - CPTu data

KO2 - SDMT data

KO2 - Vane shear tests data

HD004101INT-OPS-RPT-CPT results

Lucy - CPTu data

Lucy - DMT data

Lucy - CPTu data

Lucy - SDMT data

Lucy - PMT data

Lucy - lab testing results