Appendix G - TSF Design Documentation

cdmsmith.com

Volume 2

Contents

HANKING AUSTRALIA INVESTMENT
RUSTLERS ROOST GOLD PROJECT

FEASIBILITY DESIGN REPORT

PREPARED FOR:

Hanking Australia Investment
Level 26, 140 St Georges Terrace
Perth, Western Australia
Australia, 6000

PREPARED BY:

Knight Piésold Pty Limited
Level 1, 184 Adelaide Terrace
East Perth, WA 6004, AUSTRALIA
p. +61 8 9223 6300 • f. +61 8 9223 6399

PE801-00102/04
Rev 1
September, 2022

DOCUMENT CONTROL PAGE

HANKING AUSTRALIA INVESTMENT

RUSTLERS ROOST GOLD PROJECT

FEASIBILITY DESIGN REPORT

KP Job No. PE801-00102/06

KP Report No. PE801-00102/04

DOCUMENT INFORMATION

REV	DESCRIPTION	PREPARED	REVIEW	KNIGHT PIÉSOLD APPROVAL	DATE
A	Issued for Review	AMM	DJTM	DJTM	22/12/2021
B	Modified Basin	AMM AMM	DJTM DJTM	DJTM DJTM	20/05/2022
0	Issued as Final	AMM AMM	DJTM DJTM	DJTM DJTM	16/08/2022
1	Geochemistry Added	[signature] AMM	[signature] DJTM	[signature] DJTM	16/08/2022

DOCUMENT DISTRIBUTION

REV	DESTINATION	HARD COPY	ELECTRONIC COPY
A	HANKING AUSTRALIA INVESTMENT	0	1
B	HANKING AUSTRALIA INVESTMENT	0	1
0	HANKING AUSTRALIA INVESTMENT	0	1
1	HANKING AUSTRALIA INVESTMENT	0	1

Knight Piésold CONSULTING

CONTENTS **PAGE**

CONTENTS PAGE

CONTENTS **PAGE**

CONTENTS **PAGE**

APPENDIX B

Memorandum Ref. PE21-01464, Rustlers Roost Gold Project - Seismic Hazard Assessment

MEMORANDUM

To:	Hanking Australia Investment	**Date:**	19 November 2021
Attn:	John Zimmerman	**Our Ref:**	PE21-01464
		KP File Ref.:	PE801-00102/06-A jl M21009
cc:	Charles Hastie	**From:**	Zhenhe Song/Jim Luo

RE: RUSTLERS ROOST GOLD PROJECT – SEISMIC HAZARD ASSESSMENT

1. INTRODUCTION

A seismic hazard assessment (SHA) has been performed for the Rustlers Roost Project, located approximately at Longitude 131.50°E and Latitude -12.92°S in Western Australia.

Available information including historical earthquake catalogues and publications on the tectonics and seismicity of Australian regions has been reviewed for this study.

Seismic ground motion parameters have been evaluated, using a probabilistic assessment, which was performed using the Openquake software (Ref. 1) and the seismic source models prepared by Geoscience Australia (GA) in 2018 for the National Seismic Hazard Assessment (NSHA18) project (Ref. 2).

Seismic design parameters of Operating Basis Earthquake (OBE) and Safety Evaluation Earthquake (SEE) for the Tailings Storage Facility (TSF) were selected based on the assessment results and the facility's assumed consequence category.

2. REGIONAL SEISMICITY

The seismicity of much of Australia is typical of an intra-plate region, characterised by low levels of seismic activity and earthquakes apparently randomly distributed in location and time. The correlation between recorded earthquakes and geological features is typically not well known or understood.

The Rustlers Roost Project is located on the Northern Australia Craton. The Northern Australian Craton (Ref. 3 and 4) includes Palaeoproterozoic orogens and basins in northern Australia including the Halls Creek, Pine Creek, McArthur, Mount Isa, Tennant Creek, Tanami, and Aileron (northern Arunta) geological regions. Archean basement to the NAC crops out in the Pine Creek and Tanami regions, with ages in the range 2.67 Ga – 2.50 Ga.

The Rustlers Roost Project is shown on the geological mapping by Bagas (2011, Ref. 5) in Figure 2.1.

In addition to the intra-plate seismicity, earthquakes occurring within subducted oceanic slabs related to the Sunda-Banda Arc and the North Papua Subduction Zone are significant for seismic hazard in Northern Australia. In particular, ground motions generated by earthquakes within the eastern Banda Arc slab propagate efficiently through the crust to northern Australia.

The location of the Sunda-Banda Arc which is adopted in the NSHA18 project is shown on Figure 2.2 and approximately 500km away from the project site. The Indonesian subduction sources are shown in Figure 2.3.

Historical earthquake data for the extended region near the project site was obtained from the Geoscience Australia earthquake database (Ref. 6) and International Seismological Centre (ISC) earthquake catalogues (Ref. 7). The data includes all recorded earthquakes (M2.0 and above) within the region from 1900 to 2021. The earthquake database indicates that 437 earthquakes have occurred within 500 km of the project site with earthquake magnitude in the range of M2.0 to M5.3, including a M5.3 (local magnitude scale, mb) event that occurred within 360 km of the site in 1970. The historical earthquake data are presented on Figure 2.4.

It should be noted that the largest Australia earthquake on record, the Tennant Creek earthquake occurred on 22nd January 1988 with a magnitude of M6.6, is approximately 790 km from the project site. The location of the earthquake is shown in Figure 2.5.

As Australia is an intra-plate region, all of the recorded earthquakes occurred at shallow depths within the Earth's crust (focal depths less than 40 km). The focal depths of the recorded earthquakes were typically in the range of 0 to 25 km.

3. PROBABILISTIC ANALYSIS

A probabilistic seismic hazard analysis has been carried out to define a unique probability of occurrence for each possible level of ground acceleration experienced at the site. The methodology used for the probabilistic analysis is based on the methods by Cornell (1968, Ref. 8).

The likelihood of occurrence of earthquakes within defined seismic source zones can be determined by examining seismicity data. Using historical earthquake records for the region, magnitude-frequency recurrence relationships can be established for potential earthquake source zones. The magnitude recurrence relationships are of the form derived by Gutenberg-Richter (1944, Ref. 9):

$$\log(N) = a - b(M)$$

where, M = Earthquake magnitude
N = Annual frequency of occurrence for earthquakes exceeding magnitude M (1/N = Return Period)

The parameter "a" is dependent on the period of observation and the level of seismicity, whilst the parameter "b" is a measure of seismicity that describes the scaling relationship between the number of small and large earthquakes.

The computer program OpenQuake (Global Earthquake Model (GEM), 2019) was used to develop a probabilistic seismic hazard model for the site using the seismic source models compiled by GA and third parties for the NSHA18 Map. Epistemic uncertainty was modelled through the use of logic trees where expert-assigned weights are given to different possible parameterisations. In total, 16 alternative seismic source models were adopted, consisting of eight developed by Geoscience Australia (Leonard, 2008; Burbidge, 2012; Griffin et al., 2016) and eight by third party contributors (Hall et al., 2007; Cuthbertson, 2016; Dimas et al., 2016; Dimas and Venkatesan, 2016; Sinadinovski and McCue, 2016; Mote et al., 2017). The source models are summarised in Table 3.1.

Table 3.1: List of Geoscience Australia and third-party source models in NSHA18

Model Name	Model Type	Inter-Class Weight	Intra-Class Weight	Reference
AUS6	Regional	0.062	0.293	Dimas et al. (2016)
DIM-AUS			0.301	Dimas and Venkatesan (2016)
NSHM12			0.406	Burbidge (2012) and Leonard et al. (2012)
ARUP	Background	0.201	0.34	Mote et al. (2017)
Leonard			0.174	Leonard (2008)
Neotectonic Domains			0.303	Clark et al. (2012)
NSHM12 Background			0.119	Burbidge (2012) and Leonard et al. (2012)
Sinadinovski & McCue			0.064	Sinadinovski and McCue (2016)
Cuthbertson	Smoothed Seismicity	0.088	0.194	Cuthbertson (2016)
Adaptive Smoothing			0.427	Griffin et al. (2016)
Fixed Kernel Smoothing			0.155	Griffin et al. (2016)
Risk Frontiers			0.224	Hall et al. (2007)
AUS6 with NFSM	Seismotectonic	0.4	0.215	Dimas et al. (2016)
DIM-AUS with NFSM			0.273	Dimas and Venkatesan (2016)
NSHM12 with NFSM			0.512	Griffin et al. (2016) and Clark et al. (2016)
Cuthbertson	Smoothed Seismicity with NFSM	0.249	0.194	Cuthbertson (2016) 2018 update
Adaptive Smoothing			0.427	Griffin et al. (2016)
Fixed Kernel Smoothing			0.155	Griffin et al. (2016)
Risk Frontiers			0.224	Hall et al. (2007) 2018 update

Ground motion prediction equations (GMPE's) relate earthquake source characteristics and propagation path of the seismic waves to the ground motion at a specific site. The estimated ground motion is typically quantified as a function of magnitude, distance, style of faulting as observed in a certain seismotectonic regime. Nineteen GMPEs models were considered as shown in Table 3.2 and weights were assigned for each neotectonic super-domain (cratonic, non-cratonic, extended and subduction zone regions) according to the expert elicitation methodology.

Table 3.2: Ground-motion model weights applied in NSHA18

Model Name	Tectonic Region Type	Weight	Reference	Integration Distance (km)
Allen2012	Non-cratonic, Extended, Oceanic and Active Crust	0.208	Allen (2012)	400 km
AtkinsonBoore2006		0.138	Atkinson and Boore (2006)	
BooreEtAl2014		0.166	Boore *et al.* (2014)	
ChiouYoungs2008SWISS01		0.153	Edwards *et al.* (2016)	
ChiouYoungs2014		0.13	Chiou and Youngs (2014)	
SomervilleEtAl2009 NonCratonic		0.205	Somerville *et al.* (2009)	
Allen2012	Cratonic	0.192	Allen (2012)	400 km
AtkinsonBoore2006		0.125	Atkinson and Boore (2006)	
AtkinsonBoore2006 Modified2011		0.118	Atkinson and Boore (2011)	
BooreEtAl2014		0.106	Boore *et al.* (2014)	
ChiouYoungs2014		0.085	Chiou and Youngs (2014)	
SomervilleEtAl2009 YilgarnCraton		0.228	Somerville *et al.* (2009)	
ZhaoEtAl2006AscSWISS05		0.146	Edwards *et al.* (2016)	
AbrahamsonEtAl2015SSlab	Subduction*	0.222	Abrahamson *et al.* (2016)	1000 km
Allen2012		0.104	Allen (2012)	
AtkinsonBoore2006		0.139	Atkinson and Boore (2006)	
AtkinsonBoore2006 Modified2011		0.17	Atkinson and Boore (2011)	
AtkinsonBoore2003SSlab		0.141	Atkinson and Boore (2003)	
BooreEtAl2014		0.125	Boore *et al.* (2014)	
SomervilleEtAl2009 NonCratonic		0.099	Somerville *et al.* (2009)	

Note: * The subduction source zones in GA model are more than 1,000km away from the Boddington site, thus the sources do not have impact on the SHA.

The probabilistic analysis was used to estimate the peak ground accelerations for seismic events with different annual frequencies of occurrence. The peak ground acceleration (PGA) was calculated for each attenuation model and the mean values of peak ground acceleration have been determined for return periods ranging from 100 to 10,000 years.

Table 3.3 presents the estimated peak ground accelerations and probabilities of exceedance corresponding to earthquake return periods of between 100 and 10,000 years and design lives of 10, 20, 30 and 50 years respectively. The peak ground accelerations are the weighted mean, 50% fractile and 85% fractile values calculated from the attenuation functions used in OpenQuake.

Table 3.3: Summary of probabilistic analysis

Return Period (Years)	Annual Probability of Exceedance	Probability of Exceedance for Design Life				PGA Mean (g)	PGA 50% fractile (g)	PGA 85% fractile (g)
		10 years	20 years	30 years	50 years			
100	0.01	9.5%	18.1%	25.9%	39.3%	0.012	0.013	0.016
475	0.0021	2.1%	4.1%	6.1%	10.0%	0.024	0.024	0.031
1,000	0.001	1.0%	2.0%	3.0%	4.9%	0.033	0.031	0.043
2,000	0.0005	0.5%	1.0%	1.5%	2.5%	0.044	0.040	0.057
2,500	0.0004	0.4%	0.8%	1.2%	2.0%	0.049	0.044	0.063
5,000	0.0002	0.2%	0.4%	0.6%	1.0%	0.068	0.061	0.088
10,000	0.0001	0.1%	0.2%	0.3%	0.5%	0.103	0.087	0.140

Note: Site Class B/C condition with an average shear wave velocity of 760 m/s in the top 30 m

GA published the seismic hazard assessment results as Peak Ground Acceleration (PGA) maps for earthquake with return periods of 475 and 2,475 years. Figure 3.1 shows the mean PGA map with a return period of 475 years, in which the project is located in a 0.02 – 0.03g zone. Figure 3.2 shows the mean PGA map with a return period of 2,475 years, in which the project is located in a 0.04g to 0.06g zone. These PGAs are consistent with the assessment results summarised in Table 3.3.

The maximum accelerations presented in this memorandum are based on rock site conditions, as defined by the ASCE/SEI 7-10 (Ref. 28) with an average shear wave velocity of 760 m/s in the top 30 m (Table 3.4). Peak ground accelerations within the embankments / dumps / tailings deposit may be higher due to amplification of ground motion through the embankment and tailings. Empirical relationships indicate that ground accelerations may amplify by a factor of 1.5 to 3.0 (Ref. 29) depending on the ground conditions.

The foundation conditions at the Rustlers Roost site could be classified as Site Class C according to ASCE/SEI 7-10 and the site investigation being undertaken. The standard penetration test (SPT) results at the TSF site are typically refusal within the top 0.5 m except BH-01 where two results of 32 and 8 were recorded to about 5 m. Approximate amplification factors according to ASCE/SEI 7-10 should be applied to account for the potential seismic site response through the foundation materials.

Table 3.4: Site Classification (Adapted from ASCE/SEI 7-10, Table 20.3-1)

<table>
<tr><th rowspan="2">Site Class</th><th rowspan="2">Soil profile name</th><th colspan="3">Average properties in top 30 m</th></tr>
<tr><th>Soil shear wave velocity, v_S, (m/s)</th><th>Standard Penetration Resistance 'N'</th><th>Soil undrained shear strength, s_U (kPa)</th></tr>
<tr><td>A</td><td>Hard rock</td><td>v_S > 1,500</td><td>N/A</td><td>N/A</td></tr>
<tr><td>B</td><td>Rock</td><td>760 < v_S ≤ 1,500</td><td>N/A</td><td>N/A</td></tr>
<tr><td>C</td><td>Very dense soil and soft rock</td><td>360 < v_S ≤ 760</td><td>N ≥ 50</td><td>s_U ≥ 96</td></tr>
<tr><td>D</td><td>Stiff soil</td><td>180 < v_S ≤ 360</td><td>15 ≤ N ≤ 50</td><td>48 ≤ s_U ≤ 96</td></tr>
<tr><td rowspan="2">E</td><td rowspan="2">Soft clay soil</td><td>v_S < 180</td><td>N ≤ 15</td><td>s_U ≤ 48</td></tr>
<tr><td colspan="3">Any profile with more than 3 m of soil with the following characteristics:
1. Plasticity index PI > 20,
2. Moisture content w ≥ 40% and
3. Undrained shear strength s_U < 24</td></tr>
<tr><td>F</td><td>Soils requiring site response analysis in accordance with Section 21.1</td><td colspan="3">Any profile containing soils having one or more of the following characteristics:
1. Soils vulnerable to potential failure or collapse under seismic loading, such as liquefiable soils, quick and highly sensitive clays, and collapsible weakly cemented soils.
2. Peats and/or highly organic clays (H > 3 m) of peat and/or highly organic clay where H = thickness of soil).
3. Very high plasticity clays (H > 7.6 m with PI > 75).
4. Very thick soft/medium stiff clays (H > 37 m) with s_u < 48</td></tr>
</table>

4. SEISMIC DESIGN PARAMETERS

4.1 SEISMIC DESIGN CATEGORIES

According to ANCOLD guidelines (Ref. 31) for earthquake design of Dams and Appurtenant Structures, three levels of design earthquake are typically considered: Operating Basis Earthquake (OBE); Safety Evaluation Earthquake (SEE); and Maximum Credible Earthquake (MCE). The SEE replaces the term Maximum Design Earthquake (MDE) in previous ANCOLD guidelines (Ref. 32 and 33).

Appropriate return intervals for the design earthquakes need to be determined based on the consequence category for the TSF, which considers the consequences of failure. This assessment typically includes consideration of the potential loss of life and environmental and economic impacts due to failure (Ref. 32). The TSF has been assumed to have a High A consequence category. Table 4.1 summarises the recommended Annual Exceedance Probability (AEP) corresponding to different earthquakes and failure consequences categories.

Table 4.1: Recommended Design Earthquake Loadings (AEP)

Dam Consequence Category	Operating Basis Earthquake OBE(1)	Safety Evaluation Earthquake SEE(2,8)
Extreme Consequence Category Dams	Commonly 1 in 475 AEP up to 1 in 1,000 AEP	The greater of: Ground motion from MCE on known active faults(3) or Probabilistic ground motion Extreme: 1 in 10,000 AEP(4)
High A, B and C Consequence Category Dams	Commonly 1 in 475 AEP up to 1 in 1,000 AEP	Probabilistic ground motion(5,6,7): High A: 1 in 10,000 AEP High B: 1 in 5,000 AEP High C: 1 in 2,000 AEP
Significant Consequence Category Dams	Commonly 1 in 475 AEP	Probabilistic ground motion(5,6): 1 in 1,000 AEP
Low Consequence Category Dams	Commonly 1 in 475 AEP	Probabilistic ground motion(5,6): 1 in 1,000 AEP

From ANCOLD (2019), Guidelines for Design of Dams and Appurtenant Structures for Earthquake
(1) To be determined by the Owner and other Stakeholders in consultation with the Consultant.
(2) The design of the dam should be such that there will be a low likelihood of the dam failing given the SEE.
(3) Active faults are as defined in ANCOLD (2019). Use the 85th percentile of the hazard from the deterministic MCE. This represents the stand deviation of the random variability about the best estimate of the hazard level from that single earthquake scenario.
(4) 85th fractile. This is required so that the design is more likely to have a sufficiently low likelihood of failure given the SEE, than if the median loading was used.
(5) Median, 50th fractile.
(6) For High B, High C, Significant, and Low Dam Failure Consequence Category dams, if the structure is susceptible to liquefaction or has components that will fail at ground motions only a little greater than those presented in the Table, check the design for the critical ground motion and assess the adequacy of the design using risk assessment methods.
(7) Adoption of these SEE criteria for High B and High C Dam Failure Consequence Category dams may not provide an acceptable level of risk in accordance with ANCOLD risk management guidelines. It is therefore recommended that some level of risk assessment should be undertaken in these cases before adopting the AEP stated in the table. If it cannot be demonstrated that an acceptable level of risk would be achieved, an AEP of 1 in 10,000 should be adopted.
(8) Post closure, tailings dams should be designed in accordance with the Guidelines on Tailings Dams, ANCOLD (2019).

4.2 OPERATING BASIS EARTHQUAKE

The OBE is the level of ground motion at the dam site for which only minor damage is acceptable (Ref. 31). An appropriate OBE for a TSF is typically determined based on the Consequence Category of the facility. The dam, appurtenant structures, and equipment should remain functional, and damage from the occurrence of earthquake shaking not exceeding the OBE should be easily repairable (Ref. 31).

The ANCOLD earthquake guideline (Ref. 31) recommend that the 1 in 1,000 AEP earthquake be adopted for the OBE, based on a consequence category of High A. The PGAs for an OBE earthquake are calculated in the probabilistic analysis as:

- 1 in 1,000 AEP earthquake PGA of 0.033 g for site Class B (higher value of mean and 50% fractile).
- The estimated PGA for site Class C is approximately 0.04 g.

The uniform hazard spectra from the probabilistic seismic hazard analysis for earthquakes with an average return period of 1,000 years are presented on Figure 4.1. The following earthquake scenarios were selected based on a review of historical seismicity and the findings of the seismic hazard analyses including deaggregation of the seismic hazard based on the assumed natural period of the TSF embankment being ~ 0.5 seconds (embankment typically has a natural period in the range of 0.5 seconds to 1.5 seconds):

- A design earthquake of M5.0 located at a distance of approximately 5 km or magnitude M7.9 located at a distance of approximately 680 km has been selected for the 1 in 1,000 AEP OBE.

The deaggregation plot for the earthquakes with an average return period of 1,000 years is shown on Figure 4.2.

4.3 SAFETY EVALUATION EARTHQUAKE

The SEE is the recommended maximum level of ground motion for which the dam should be designed or analysed. Appropriate SEE for a TSF is typically determined based on the Consequence Category of the facility. Damage can be accepted, but there should be no uncontrolled release of water or tailings.

In accordance with the ANCOLD guideline (Ref. 31), it is recommended that the 1 in 10,000 AEP peak ground acceleration be adopted for the SEE, based on a High A consequence category.

The PGAs for a SEE earthquake are calculated in the probabilistic analysis as:

- 1 in 10,000 AEP earthquake PGA of 0.087 g for site Class B (50% fractile, ANCOLD 2019).
- The estimated PGA for site Class C is approximately 0.105 g.

The uniform hazard spectra from the probabilistic seismic hazard analysis for earthquakes with an average return period of 10,000 years are presented on Figure 4.3. The following earthquake scenarios were selected based on a review of historical seismicity and the findings of the seismic hazard analyses including deaggregation of the seismic hazard based on the assumed natural period of the TSF embankment being ~ 0.5 seconds.

- A design earthquake of magnitude M5.5 located at a distance of approximately 5 km or magnitude M7.9 located at a distance of approximately 500 km has been selected for the 1 in 10,000 AEP SEE.

The deaggregation plots for the earthquakes with average return periods of 10,000 years are shown on Figure 4.4.

4.4 MAXIMUM CREDIBLE EARTHQUAKE

The 2019 ANCOLD Guidelines for Design of Dams for Earthquake (Ref. 33) include the following definitions:

- Maximum Credible Earthquake (MCE) – the MCE is the largest reasonably conceivable earthquake magnitude that is considered possible along a recognised fault or within a geographically defined tectonic province, under the presently known or resumed tectonic framework.
- Active fault – a fault, reasonably identified and located, known to have produced historical earthquakes or showing evidence of movements in Holocene time (i.e. in the last 11,000 years), long faults which have moved in Latest Pleistocene time (i.e. between 11,000 and 35,000 years ago).
- Neotectonic fault - A fault, not active as defined above, that has experienced displacement under conditions imposed in the current crustal stress regime and hence may move again in the future.

A site will have surface rupture potential if an existing fault is found, which has been active during the current stress regime. Clark et al. (2011, Ref. 38) explained that neotectonic features have undergone displacement under the current stress regime in Australia. Hence, they may have the potential for displacement in the future. The age of the current stress regime in Australia is estimated to lie in the range of 5 to 10 million years (Sandiford et al. 2004, Ref. 39).

The 2019 ANCOLD guidelines only require consideration of the ground motions from the MCE for Extreme Consequence dams, and the MCE applies only to the Active fault. Since the proposed TSF in this study is assumed to have a High A Consequence, the MCE is not required. However, for completeness of the seismic hazard assessment, the MCE was estimated according to available Neotectonic features.

An Australia-wide assessment of faulting was conducted by Clark et al. (2011, 2012, Ref. 40) indicating that the nearest Neotectonic feature to the project site is the Tennant Creek scarps, which is approximately 790 km west and shown in Figure 4.5. The Scarp has an age range of 2.58 Ma to present. There is general consensus among Australian Seismologists that the maximum credible magnitude of an earthquake in Australia is approximately magnitude 7.5. If Scarp is considered as an active fault, the MCE can be assumed to be M7.5 earthquake occurring at a distance to the rupture (R_{rup}) of 790 km from the site, causing a peak ground acceleration of less than 0.01 g (mean plus one standard deviation value from the 8 GMPEs for Cratonic) for Site Class B.

The Indonesian subduction sources have significant influence on seismic hazard in Northern Territory. If the “Megathrust Timor” subduction source shown in Figure 2.3 is considered with a magnitude of 7.9 and a distance of 500 km. The estimated peak ground acceleration is approximately 0.02g (mean plus one standard deviation value from Abrahamson et al. (2016) GMPE) for Site Class B.

For the post-closure case, ANCOLD recommend that the tailings dams are evaluated for stability for the greater of:

- Ground motion from the MCE on known active faults (only where MCE can be calculated reliably); or
- Probabilistic ground motion from a 1 in 10,000 AEP event. This is due to the long life expectation of tailings dams in the post-closure phase. Evaluation of stability can consider the long-term future condition of the closed tailings dam in terms of degree of saturation, state condition of the tailings and the consequent likelihood of liquefaction.

The 1 in 10,000 AEP event has a greater PGA than the deterministic values, therefore the 1 in 10,000 AEP event is selected as the post closure case. The PGA for site Class C is approximately 0.105 g.

The long-term properties of the tailings should be taken into account when considering stability of the TSF under the post closure case. This could include a lowered phreatic surface, increased strength from consolidation and possible chemical bonding (Ref. 33).

4.5 STRUCTURAL DESIGN

Building structures for the project should be designed to an appropriate seismic design code, such as the Australian Standard AS1170.4-2007 (Ref. 36) or International Building Code (Ref. 37). The AS1170.4-2007 is recommended for structural design in Australia, however, the results according to International Building Code are also presented for comparison.

4.5.1 AS1170.4-2007

The acceleration coefficient applicable to the design structures at the site was determined from the Earthquake Hazard Map of Western Australia 2003, as given in AS1170.4-2007. A seismic hazard factor (acceleration coefficient) of 0.08 g is recommended for the design of structures at the site. The probability of this coefficient being exceeded in a 50 year period is 10%, which is equivalent to a return period of 500 years.

For design purposes site sub-soil class C_e (shallow soil) is recommended for the plant structures, in accordance with Section 4 of AS1170.4-2007.

4.5.2 International Building Code

Seismic design to the International Building Code requires determination of seismic coefficients, SS and S1, defined as follows:

- Seismic coefficient, SS: maximum considered earthquake ground motion of 0.2 seconds spectral response acceleration (5% of critical damping).
- Seismic coefficient, S1: maximum considered earthquake ground motion of 1.0 second spectral response acceleration (5% of critical damping).

In accordance with the International Building Code, the maximum considered earthquake ground motion has been defined as the ground motion with a 2% probability of exceedance in 50 years (average return period of 2,500 years). Seismic parameters for use with IBC are provided below for the site based on the probabilistic seismic hazard analysis results.

Table 4.2: Structural Design Parameters (International Building Code)

Seismic coefficient	Return Period (years)	Acceleration Site B (g)	Acceleration Site C (g)
PGA	2,500	0.049	0.058
SS	2,500	0.083	0.099
S1	2,500	0.055	0.094

The uniform hazard spectra for an average return period of 2,500 years are presented on Figure 4.6. Figure 4.7 shows the deaggregation of the seismic hazard at periods of 0.2 seconds and 1.0 second respectively.

For geotechnical foundation design of mine site structures, an earthquake magnitude of M4.6 at a distance of 10 km (short period) or M7.9 at a distance of 670 km (long period) is recommended for seismic design analyses.

5. SUMMARY

A site-specific seismic hazard assessment has been carried out for the Rustlers Roost Project. Available historical data, earthquake catalogues, and technical publications on the tectonics and seismicity of the region have been reviewed.

The computer program OpenQuake was used to develop a probabilistic seismic hazard model for the site. Appropriate attenuation models defining the relationship between earthquake magnitude, source to site distance, and peak ground acceleration have been used in the probabilistic hazard analysis.

Seismic design parameters have been recommended for the TSF. Seismic ground motion parameters (including peak ground acceleration and earthquake magnitude) have been estimated based on probabilistic seismic hazard analysis.

It is recommended that the 1 in 1,000 AEP earthquake is adopted for the OBE, based on a High A consequence category. The estimated mean values of PGA are:

- 1,000 year average return period earthquake PGA of 0.04 g for Site Class C.

The dam, appurtenant structures, and equipment should remain functional, and damage from the occurrence of earthquake shaking not exceeding the OBE should be easily repairable.

The PGAs for a SEE earthquake are calculated in the probabilistic analysis as:

- 10,000-year return period earthquake PGA of 0.105 g for Site Class C (50% fractile, ANCOLD 2019).

The estimated PGA and design earthquakes are summarised in Table 5.1. Damage due to a SEE earthquake can be accepted, but there should be no uncontrolled release of water from the reservoir or tailings from tailing dams.

The MCE has been assumed as the “Megathrust Timor” subduction source with a magnitude of 7.9 and a distance of 500 km, causing a peak ground acceleration of 0.02 g. The 1 in 10,000 AEP event with a PGA of 0.105 g for site Class C is recommended for post closure design.

Table 5.1: Recommended Seismic Design Parameters

Design Event	Average Return Period (years)	PGA Site Class B (g)	PGA Site Class C (g)	Magnitude	Distance from site (km)
OBE	1000	0.033	0.04	5 7.9	5 680
SEE/ Post Closure	10,000	0.087	0.105	5.5 7.9	5 500

The foundation conditions at the Rustlers Roost site could be classified as Site Class C according to ASCE/SEI 7-10 based on the site investigation being undertaken.

The natural period of the TSF embankment is assumed as 0.5 second and this should be confirmed according to the designed TSF embankment height.

Building structures for the project can be designed to Australian Standard AS1170.4-2007. A seismic hazard factor (acceleration coefficient) of 0.08 g is recommended for the design. Seismic design parameters according to the International Building Code are also provided for reference.

We trust this memorandum is sufficient for your requirements. Please contact us should you have any queries relating to this memo.

Yours faithfully
KNIGHT PIÉSOLD PTY LTD

ZHENHE SONG
Senior Geotechnical Engineer

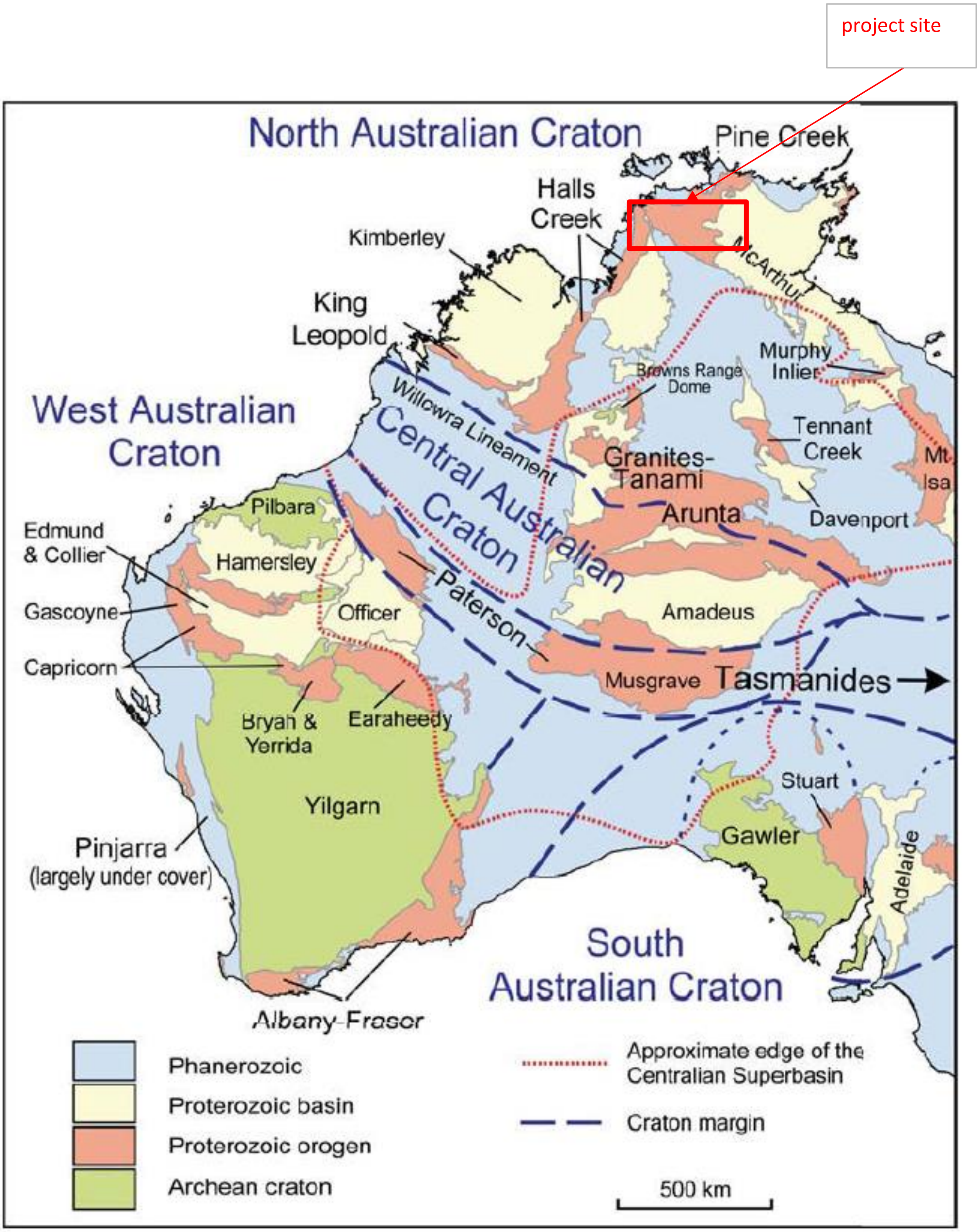
JIM LUO
Manager Geotechnical Services

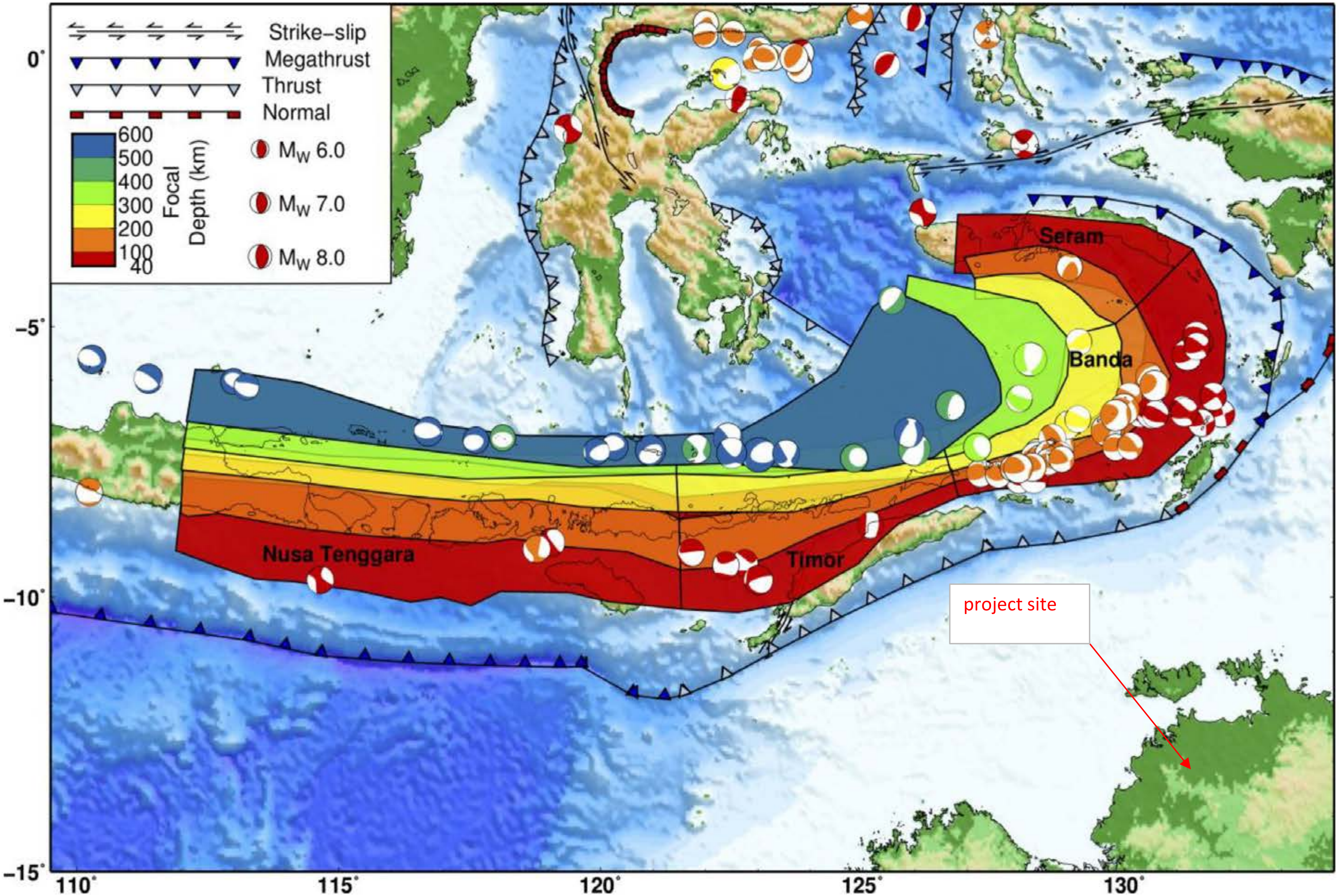
REFERENCES

1. GEM (2017), The openquake-engine User Instruction Manual
2. Australian Government – Geoscience Australia, The 2018 National Seismic Hazard Assessment for Australia, Record 2018/33.
3. Lyons, P. | Huston, D.L (2006), “Evolution and Metallogenesis of the Northern Australian Craton”. Geoscience Australia Record 2006/16
4. Myers, J. S., Shaw, R. D., & Tyler, I. M. (1996). Tectonic evolution of proterozoic Australia. Tectonics, 15(6), 1431-1446.
5. Leon Bagas (2010), “Evolution and tectonic setting of the Paleoproterozoic Giranites-tanami Orogen Western Australia”, Ph.D thesis, University of Western Australia.
6. Geoscience Australia. Online earthquake archives search facility: http://www.ga.gov.au/earthquakes/searchQuake.do.
7. International Seismological Centre, On-line Bulletin, http://www.isc.ac.uk, Internatl. Seis. Cent., Thatcham, United Kingdom, 2013.
8. Cornell, C.A., (1968), “Engineering Seismic Risk Analysis”, Bulletin of the Seismological Society of America, Vol. 58, p.1583-1606.
9. Gutenberg, B. and Richter, C.F., (1944), “Frequency of Earthquakes in California”, Bulletin of the Seismological Society of America, Vol. 34, p.185-188.
10. Paul Somerville, P.; Graves, R.; Collins, N.; Song, S.G.; Ni, S.; and Cummins, P.; Source and Ground Motion Models for Australia Earthquakes, 2009.
11. Leonard, M. (2008). One hundred years of earthquake recording in Australia, Bull. Seism. Soc. Am. 98, 1458–1470, doi: 10.1785/0120050193.
12. Burbidge, D. R., Ed. (2012). The 2012 Australian Earthquake Hazard Map, Geoscience Australia Record 2012/71.
13. Griffin, J., T. Volti, D. Clark, H. Ghasemi, M. Leonard, and T. Allen (2016). Development of the 2018 Australian National Seismic Hazard Assessment (NSHA), Australian Earthquake Engineering Society 2016 Conference, Melbourne, Victoria.
14. Hall, L., F. Dimer, and P. Somerville (2007). A spatially distributed earthquake source model of Australia, Australian Earthquake Engineering Society 2007 Conference, Wollongong, New South Wales.
15. Cuthbertson, R. J. (2016). Automatic determination of seismicity rates in Australia, Australian Earthquake Engineering Society 2016 Conference, Melbourne, Victoria.
16. Dimas, V.-A., G. Gibson, and R. Cuthbertson (2016). Revised AUS6 model: Significant changes & approaches to the seismotectonic model, Australian Earthquake Engineering Society 2016 Conference, Melbourne, Victoria.
17. Dimas, V.-A., and S. Venkatesan (2016). Seismotectonic model for the Australian Plate – beyond borders, Australian Earthquake Engineering Society 2016 Conference, Melbourne, Victoria.
18. Sinadinovski, C., and K. McCue (2016). A proposed PSHA source zone for Australia, Australian Earthquake Engineering Society 2016 Conference, Melbourne, Victoria.
19. Mote, T. I., M. L. So, and J. W. Pappin (2017). Arup NSHM - Australian National Seismic Hazard Model, Australian Earthquake Engineering Society 2017 Conference, Canberra, ACT.
20. Allen, T. I. (2012). Stochastic ground motion prediction equations for south-eastern Australian earthquakes using updated source and attenuation parameters. Geoscience Australia Record, 2012/69.
21. Atkinson, G. M., and D. M. Boore (2006). Earthquake ground-motion prediction equations for eastern North America, Bull. Seism. Soc. Am. 96, 2181-2205, doi: 10.1785/0120050245.

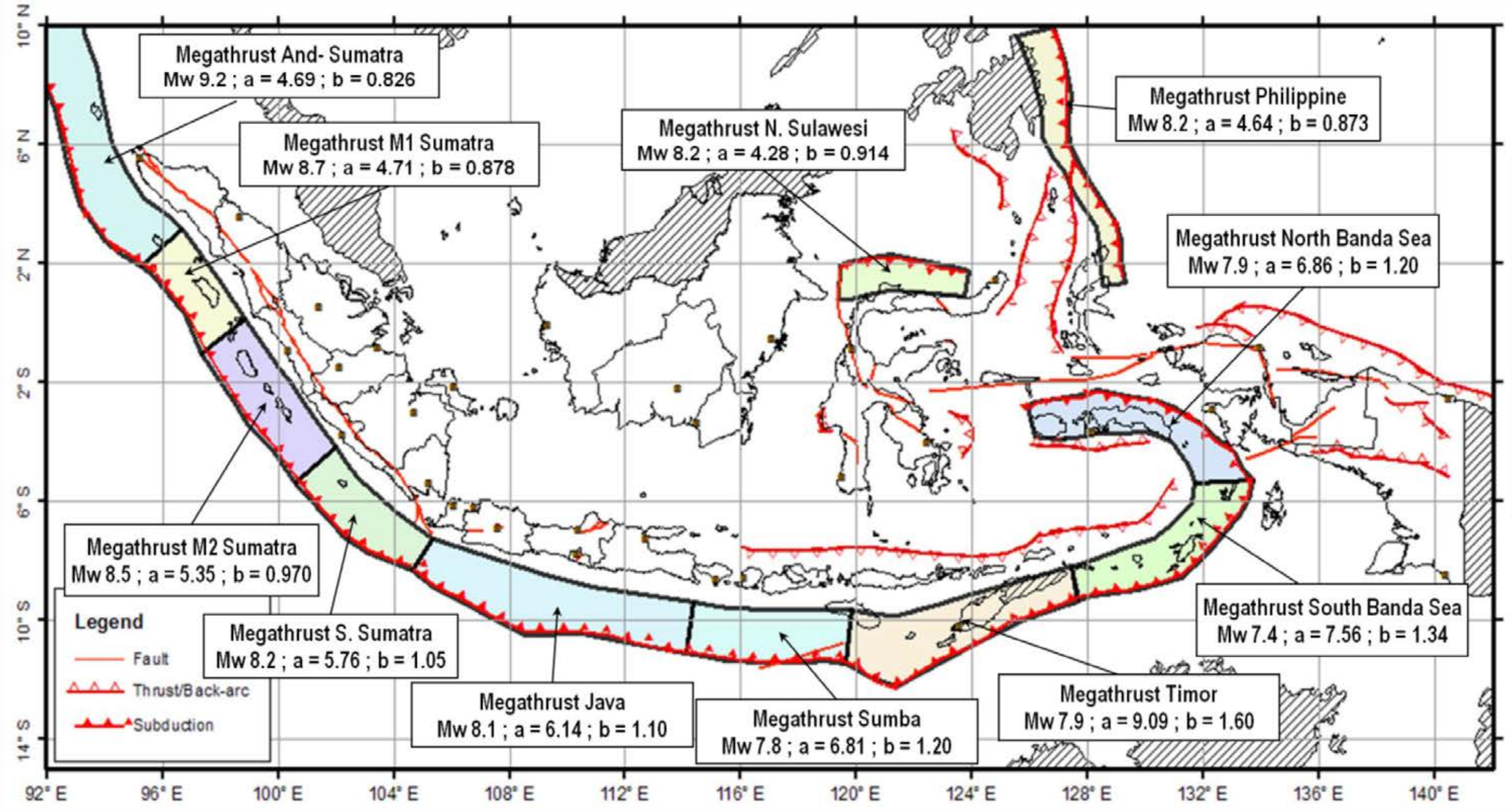
22. Boore, D. M., J. P. Stewart, E. Seyhan, and G. M. Atkinson (2014). NGA-West 2 equations for predicting PGA, PGV, and 5%-damped PSA for shallow crustal earthquakes, Earthq. Spectra 30, 1057-1085, doi: 10.1193/070113EQS184M.
23. Edwards, B., C. Cauzzi, L. Danciu, and D. Fäh (2016). Region-specific assessment, adjustment, and weighting of ground-motion prediction models: Application to the 2015 Swiss seismic-hazard maps, Bull. Seism. Soc. Am. 106, 1840-1857, doi: 10.1785/0120150367.
24. Chiou, B. S.-J., and R. R. Youngs (2014). Update of the Chiou and Youngs NGA model for the average horizontal component of peak ground motion and response spectra, Earthq. Spectra 30, 1117–1153, doi: 10.1193/072813EQS219M.
25. Atkinson, G. M., and D. M. Boore (2011). Modifications to existing ground-motion prediction equations in light of new data, Bull. Seism. Soc. Am. 101, 1121–1135, doi: 10.1785/0120100270.
26. Abrahamson, N., N. Gregor, and K. Addo (2016). BC Hydro ground motion prediction equations for subduction earthquakes, Earthq. Spectra 32, 23-44, doi: 10.1193/051712EQS188MR.
27. Atkinson, G. M., and D. M. Boore (2003). Empirical ground-motion relations for subduction-zone earthquakes and their application to Cascadia and other regions, Bull. Seism. Soc. Am. 93, 1703-1729, doi: 10.1785/0120020156.
28. American Society of Civil Engineers, ASCE / Structural Engineering Institute, SEI (2013). Minimum Design Loads for Buildings and Other Structures, ASCE/SEI 7-10.
29. Idriss, 1990, “H. Bolton Seed Memorial Symposium Proceeding”, Vol 2, page 285, BiTech Publishers, Ltd.
30. Knight Piésold (2021), “Rustlers Roost Project - Geotechnical Investigation Report” work being undertaken
31. ANCOLD (2019), Guidelines for Design of Dams and Appurtenant Structures for Earthquake.
32. ANCOLD - Australian National Committee on Large Dams (2012). Guidelines on the Consequence Categories for Dams, October 2012.
33. ANCOLD - Australian National Committee on Large Dams (2019). Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure, July 2019.
34. ICOLD (2016), Bulletin 148, Selecting Seismic Parameters for Large Dams.
35. ANCOLD (1998) “Guidelines for Design of. Dams for Earthquake”
36. Structural Design Actions, Part 4, Earthquake Actions in Australia, Australian Standards AS 1170.4 – 2007.
37. International Code Council (2015). “International Building Code”.
38. Clark, D.; McPherson, A.; Collins, C.D.N. 2011. Australia’s seismogenic neotectonic record: A case for heterogeneous intra-plate deformation. Geoscience Australia Record 2011/11.
39. Sandiford, M.; Wallace, N.; Coblentz, N. 2004. Origin of the in situ stress field in south-eastern Australia. Basin Research 16: 325-338.
40. Clark, D.; McPherson, A.; Van Dissen, R. 2012. Long-term behaviour of Australian stable continental region (SCR) faults. Tectonophysics 566-567:1-30.

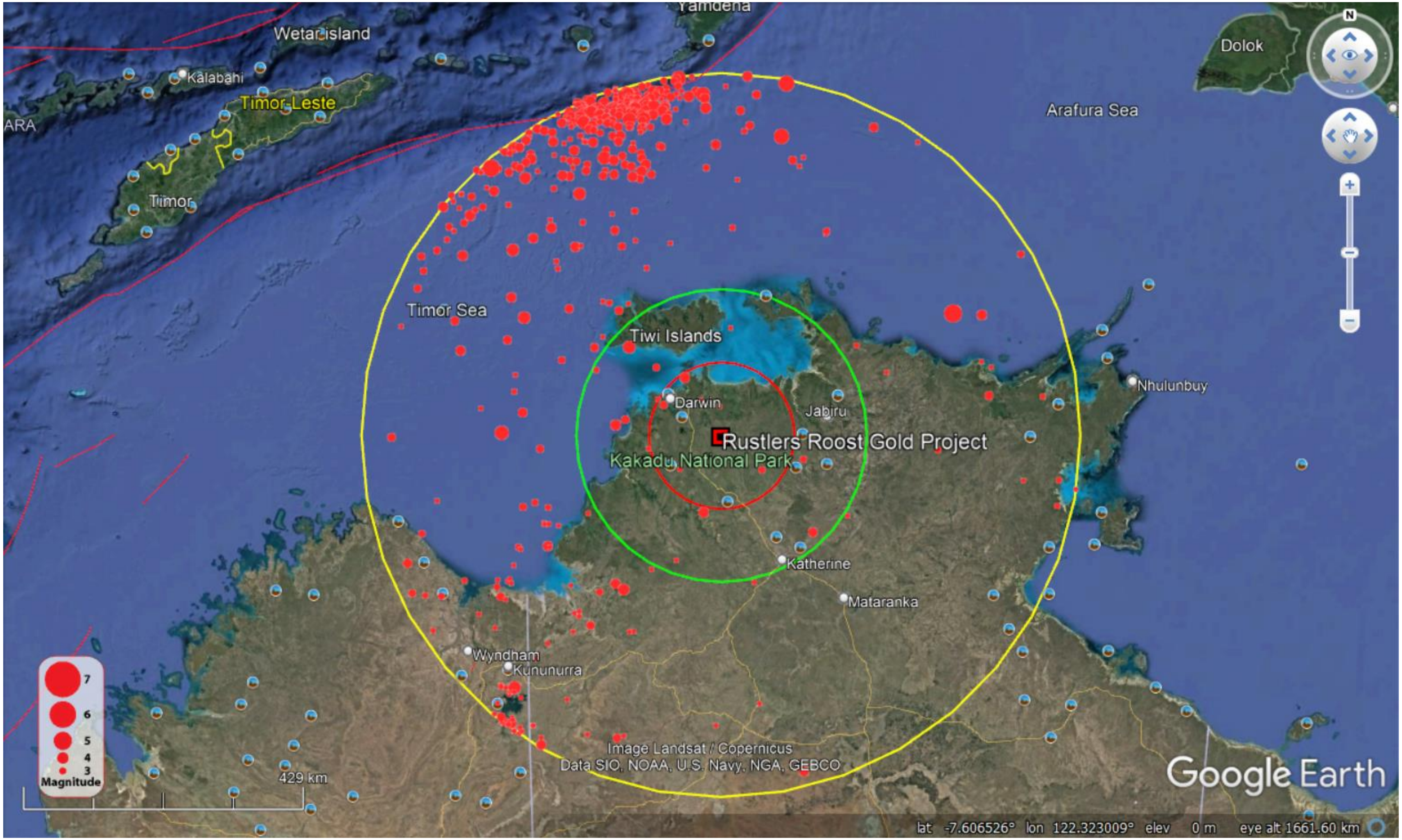
FIGURES


project site
North Australian Craton
Pine Creek
Halls Creek
Kimberley
King Leopold
McArthur
Murphy Inlier
Browns Range Dome
Tennant Creek
Mt Isa
Willowra Lineament
West Australian Craton
Central Australian Craton
Granites-Tanami
Arunta
Davenport
Pilbara
Edmund & Collier
Hamersley
Gascoyne
Officer
Paterson
Amadeus
Capricorn
Musgrave
Tasmanides
Bryah & Yerrida
Earaheedy
Stuart
Yilgarn
Gawler
Pinjarra (largely under cover)
Adelaide
South Australian Craton
Albany-Fraser
Phanerozoic
Proterozoic basin
Proterozoic orogen
Archean craton
Approximate edge of the Centralian Superbasin
Craton margin
500 km

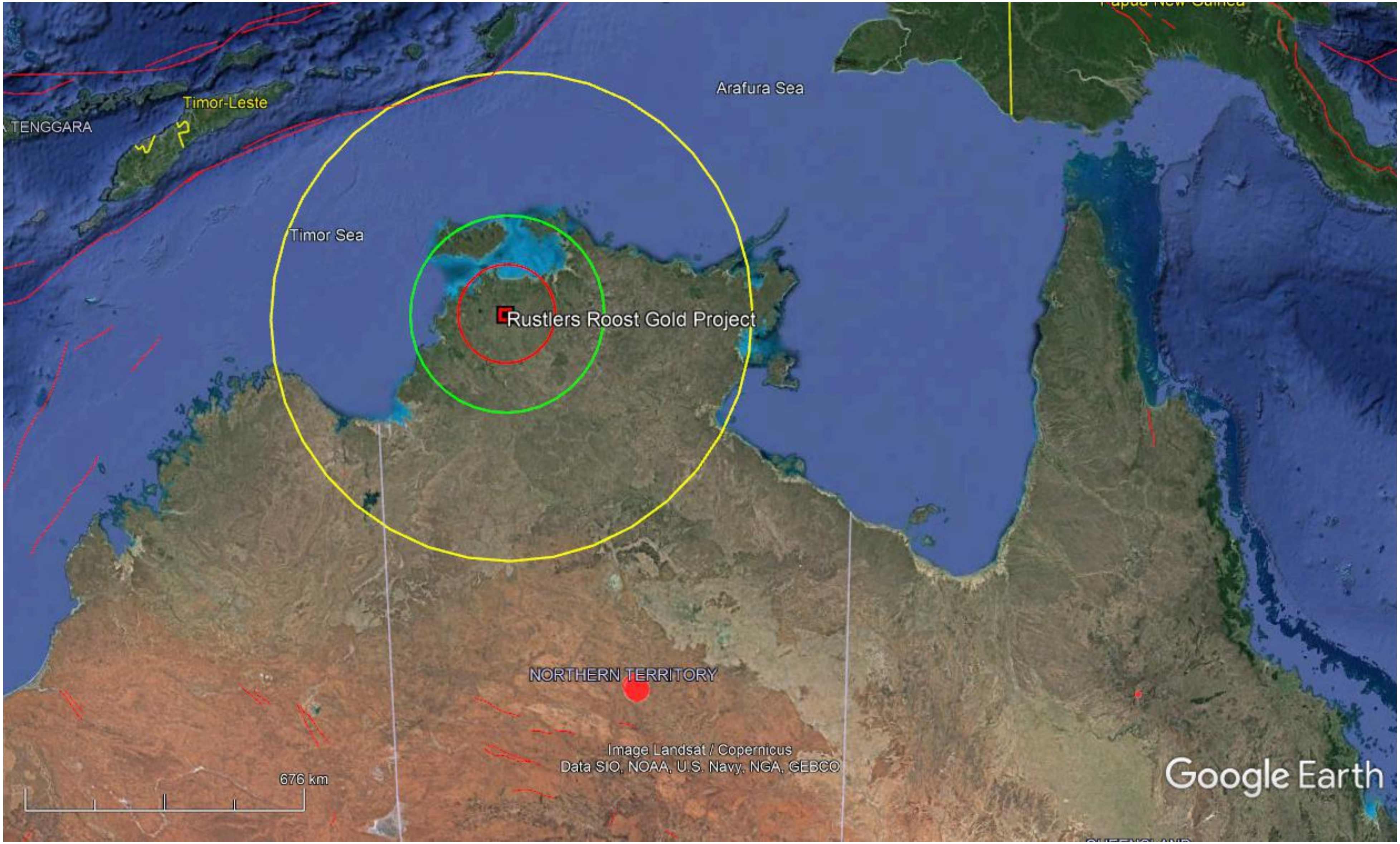


Strike-slip
Megathrust
Thrust
Normal
Focal Depth (km)
600
500
400
300
200
100
40
MW 6.0
MW 7.0
MW 8.0
0°
-5°
-10°
-15°
110°
115°
120°
125°
130°
Seram
Banda
Nusa Tenggara
Timor
project site

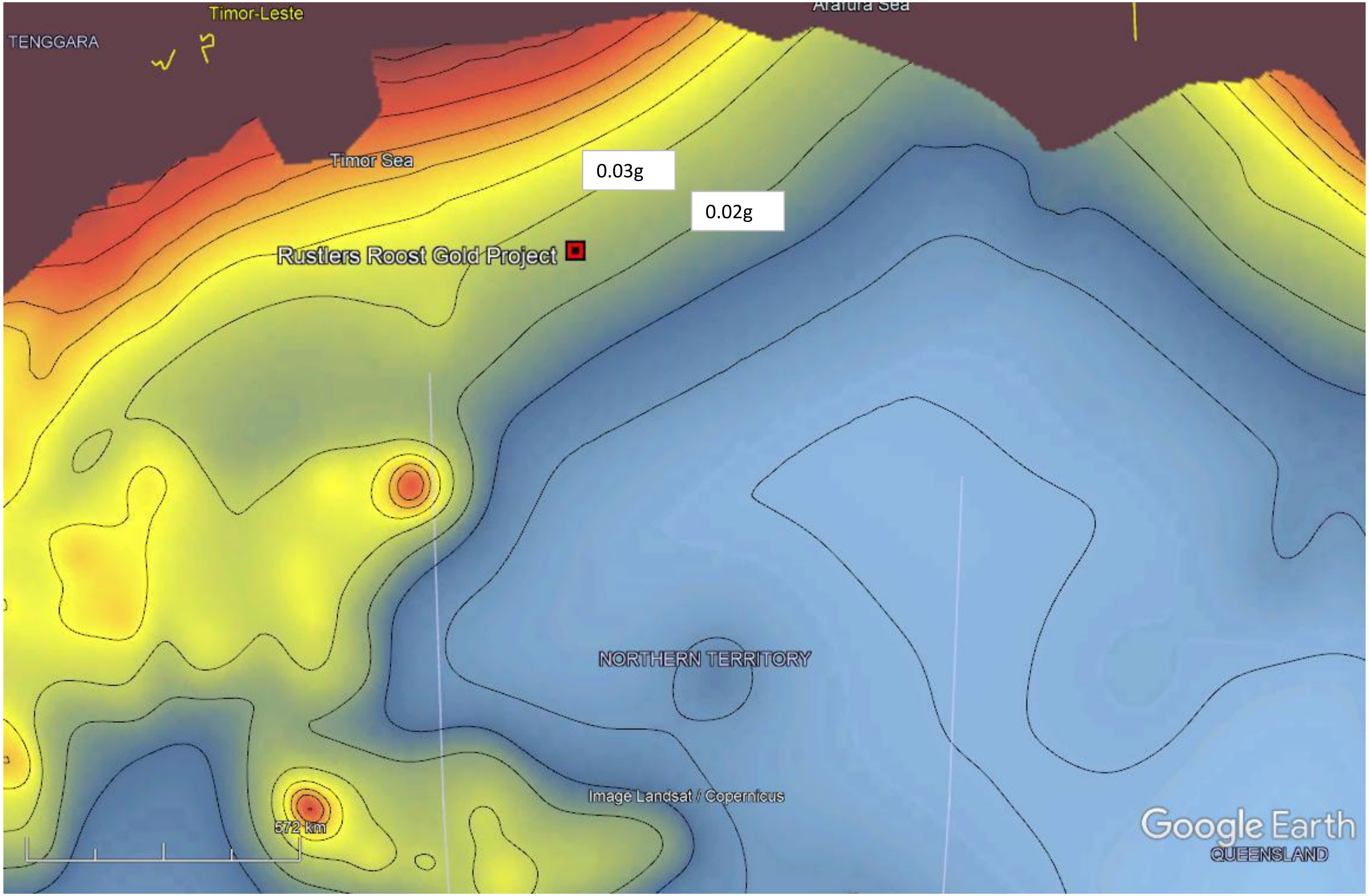

Megathrust And- Sumatra
Mw 9.2 ; a = 4.69 ; b = 0.826
Megathrust M1 Sumatra
Mw 8.7 ; a = 4.71 ; b = 0.878
Megathrust N. Sulawesi
Mw 8.2 ; a = 4.28 ; b = 0.914
Megathrust Philippine
Mw 8.2 ; a = 4.64 ; b = 0.873
Megathrust North Banda Sea
Mw 7.9 ; a = 6.86 ; b = 1.20
Megathrust M2 Sumatra
Mw 8.5 ; a = 5.35 ; b = 0.970
Megathrust S. Sumatra
Mw 8.2 ; a = 5.76 ; b = 1.05
Megathrust Java
Mw 8.1 ; a = 6.14 ; b = 1.10
Megathrust Sumba
Mw 7.8 ; a = 6.81 ; b = 1.20
Megathrust Timor
Mw 7.9 ; a = 9.09 ; b = 1.60
Megathrust South Banda Sea
Mw 7.4 ; a = 7.56 ; b = 1.34
Legend
Fault
Thrust/Back-arc
Subduction
10° N
6° N
2° N
2° S
6° S
10° S
14° S
92° E
96° E
100° E
104° E
108° E
112° E
116° E
120° E
124° E
128° E
132° E
136° E
140° E

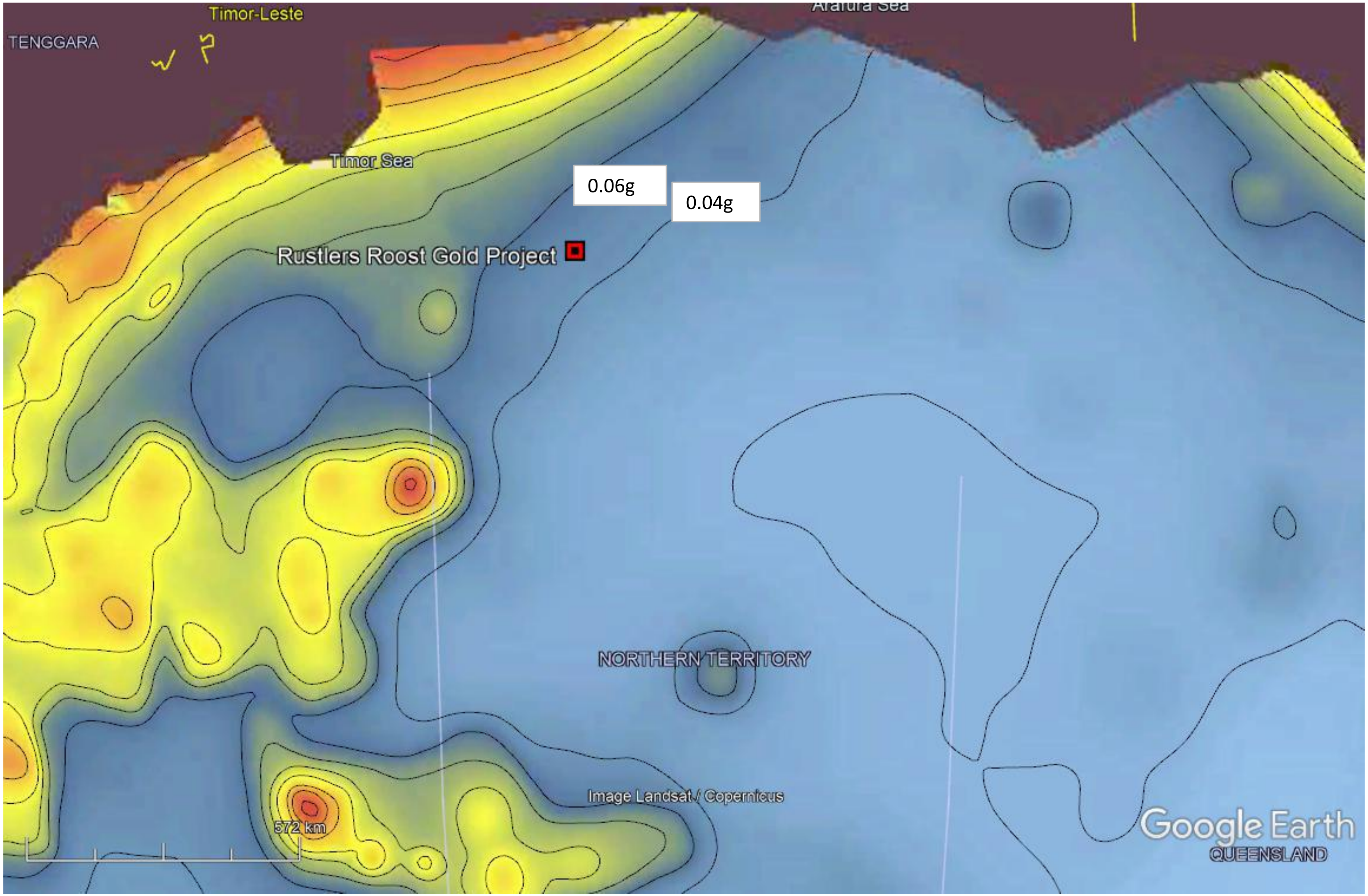


Red, green, yellow circles correspond to 100km, 200km and 500 km distance to the project site

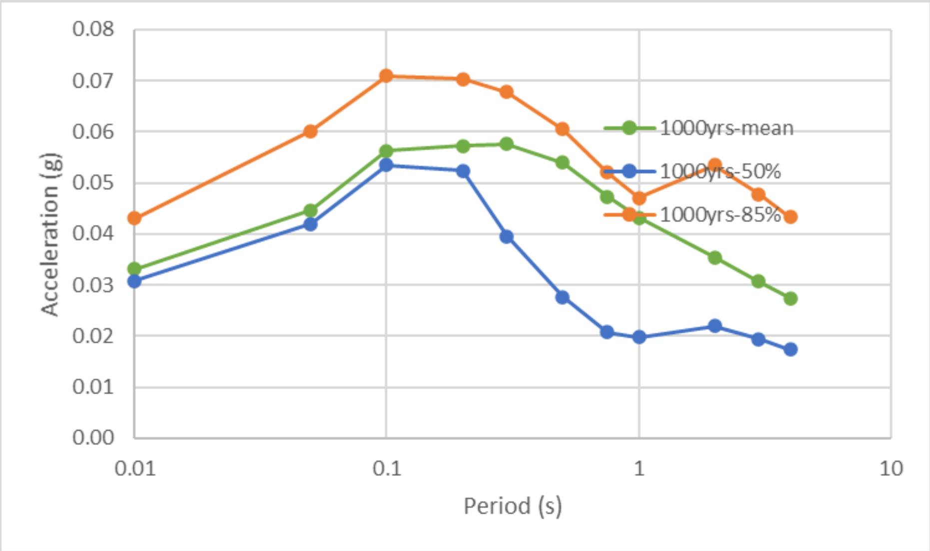


Red, green, yellow circles correspond to 100km, 200km and 500 km distance to the project site

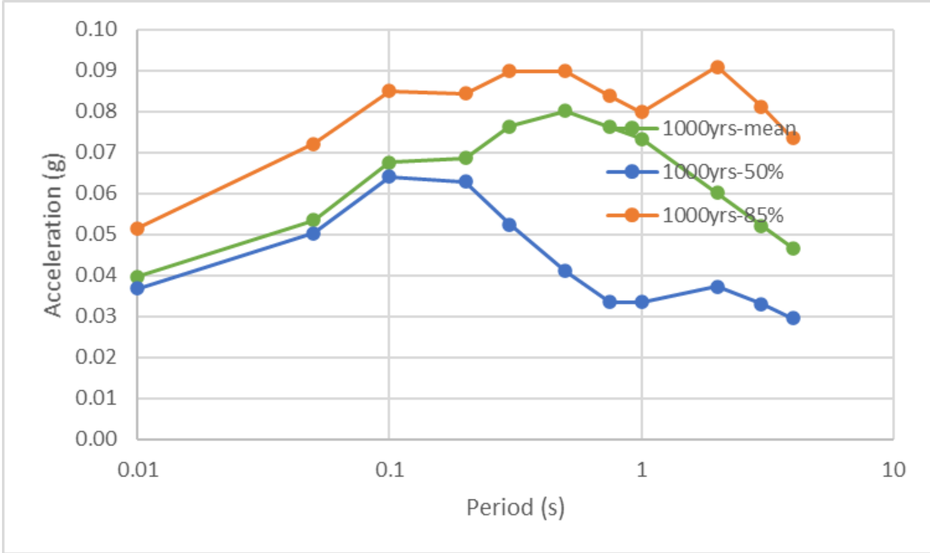

Timor-Leste
TENGGARA
Arafura Sea
Timor Sea
0.03g
0.02g
Rustlers Roost Gold Project
NORTHERN TERRITORY
Image Landsat / Copernicus
572 km
Google Earth
QUEENSLAND



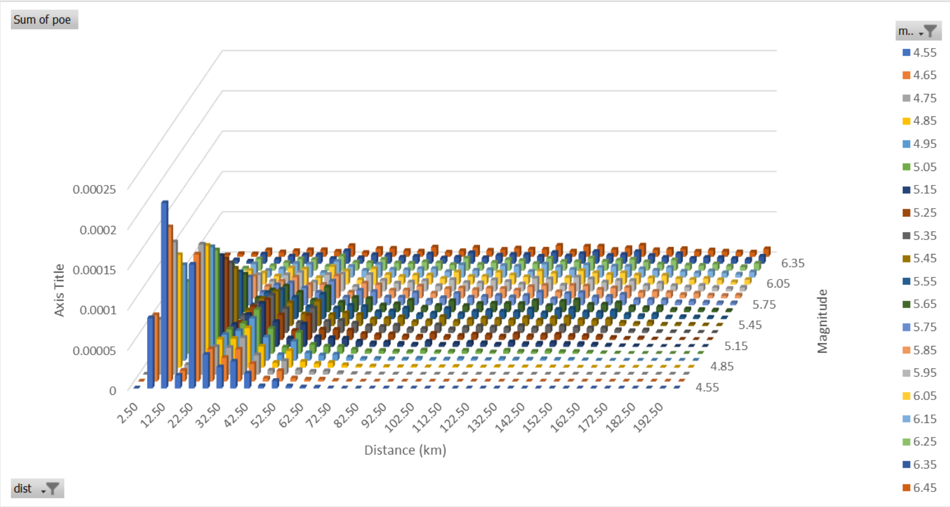
Timor-Leste
TENGGARA
Arafura Sea
Timor Sea
0.06g
0.04g
Rustlers Roost Gold Project
NORTHERN TERRITORY
Image Landsat / Copernicus
572 km
Google Earth
QUEENSLAND



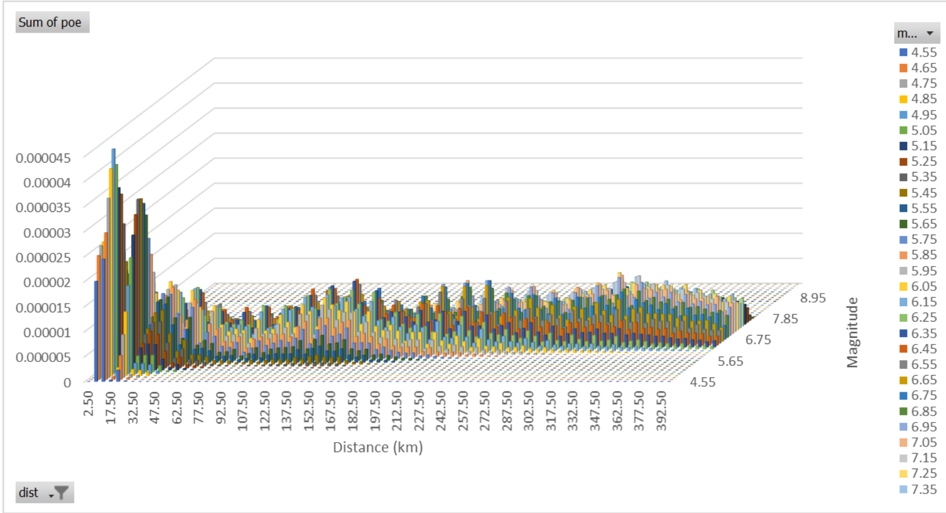
Site Class B



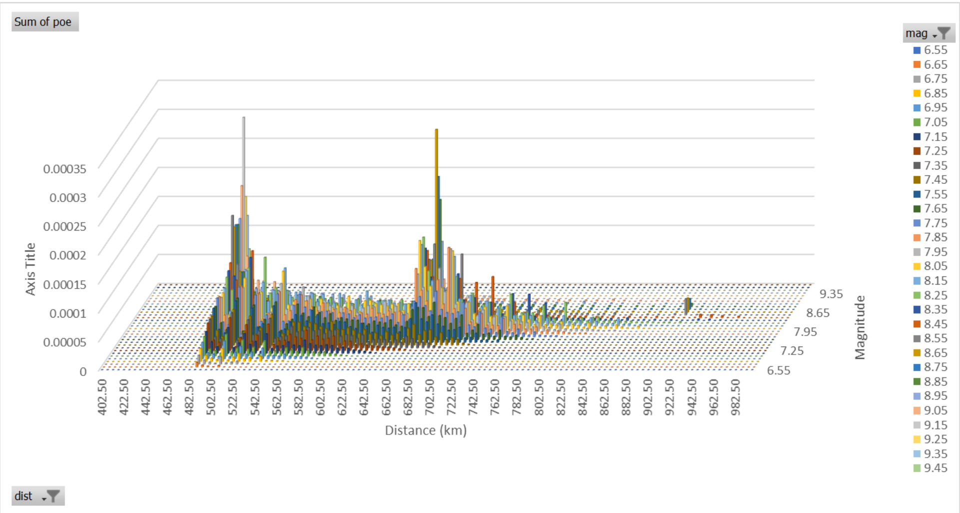
Site Class C



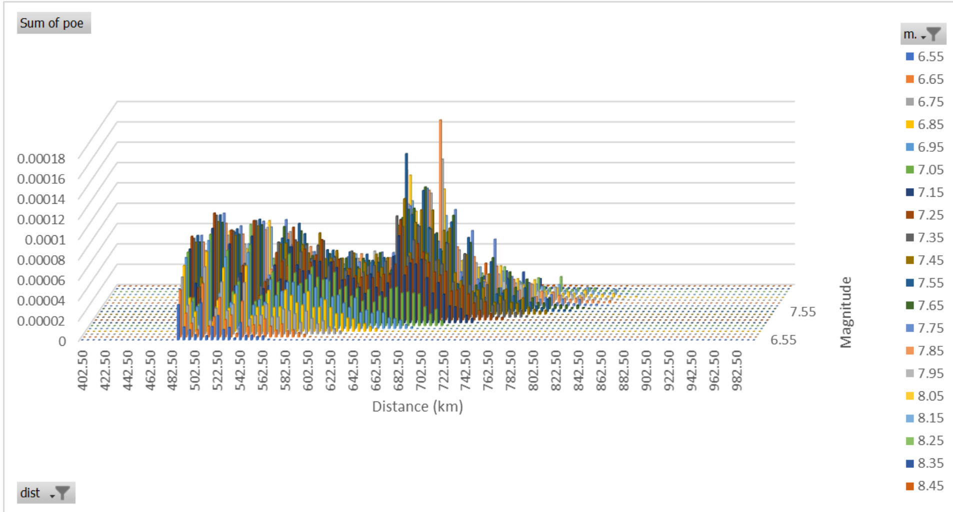
<400km



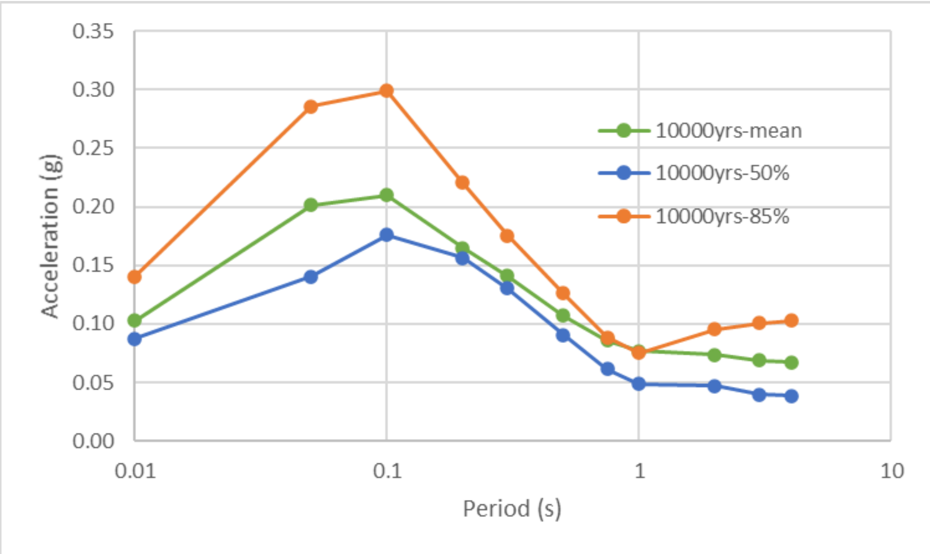
<400km

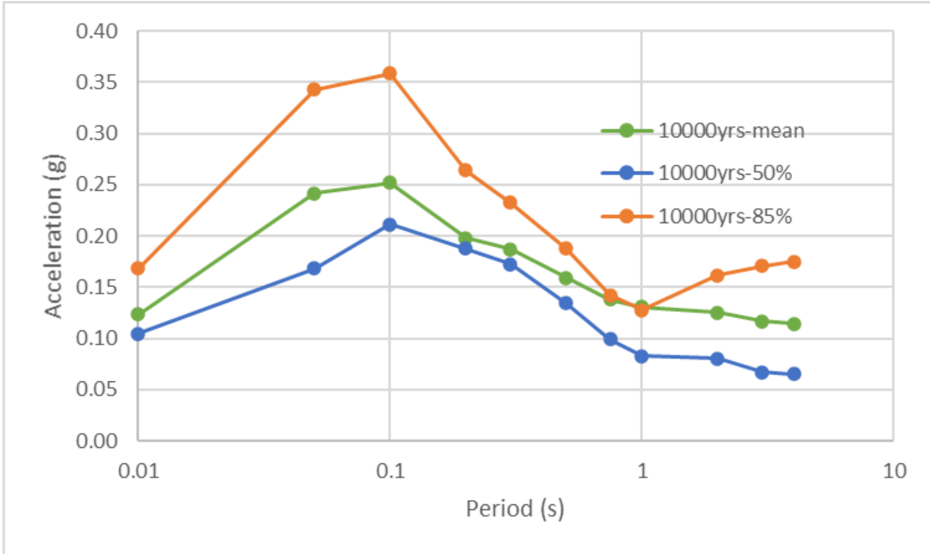


>400km
PGA



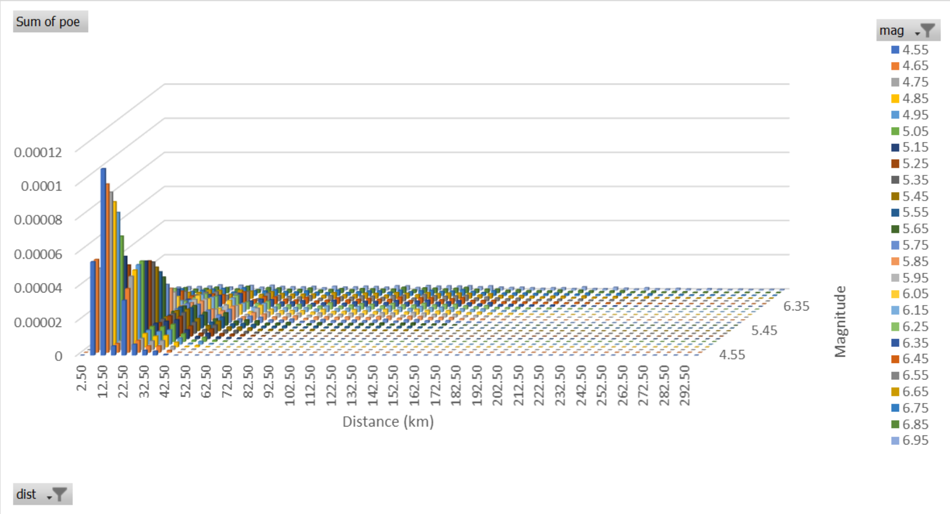
>400km
0.5 second

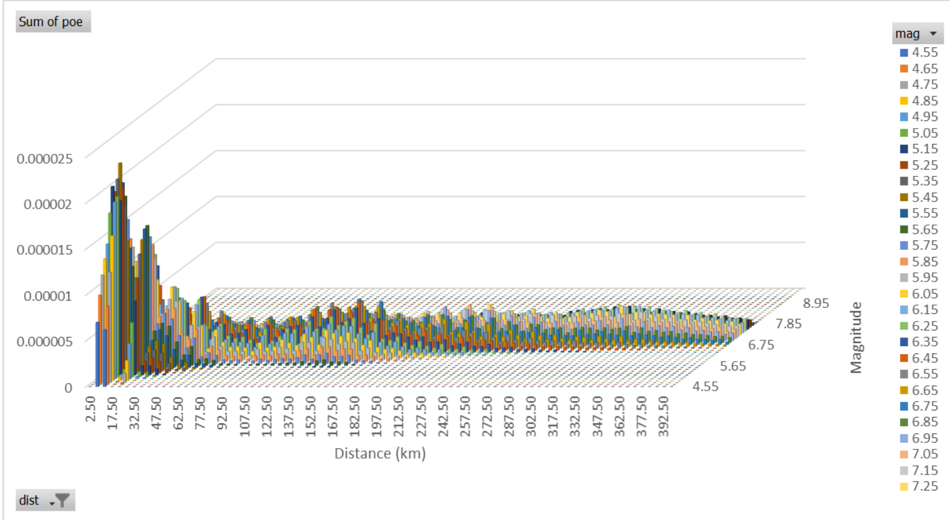




Site Class B

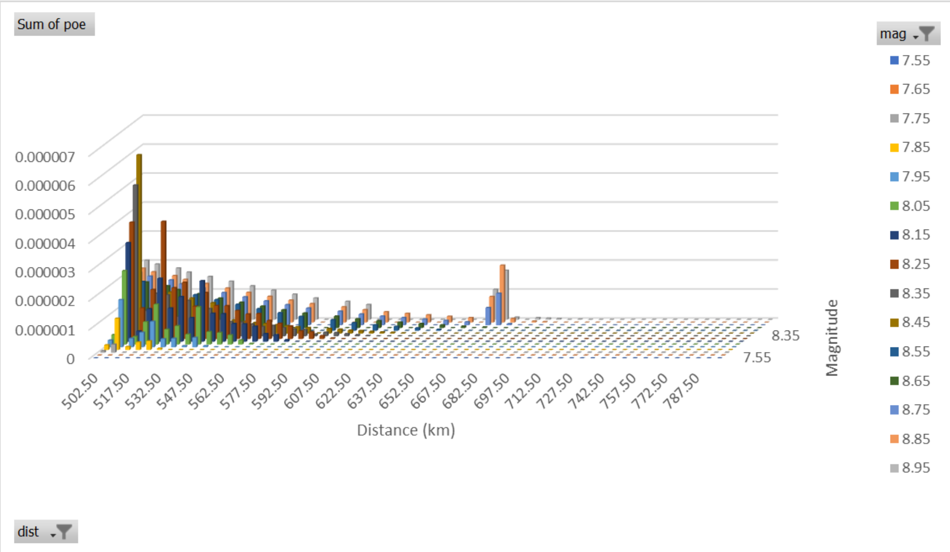
Site Class C

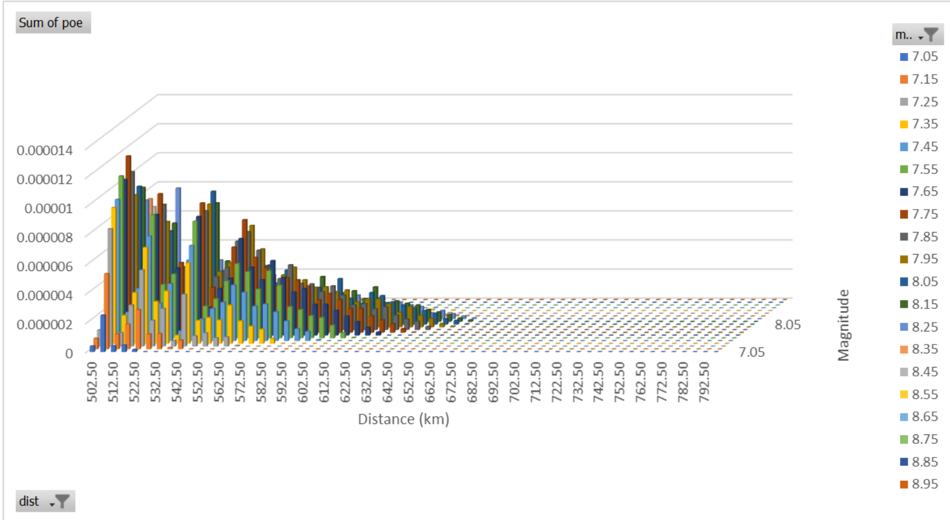




<400km

<400km



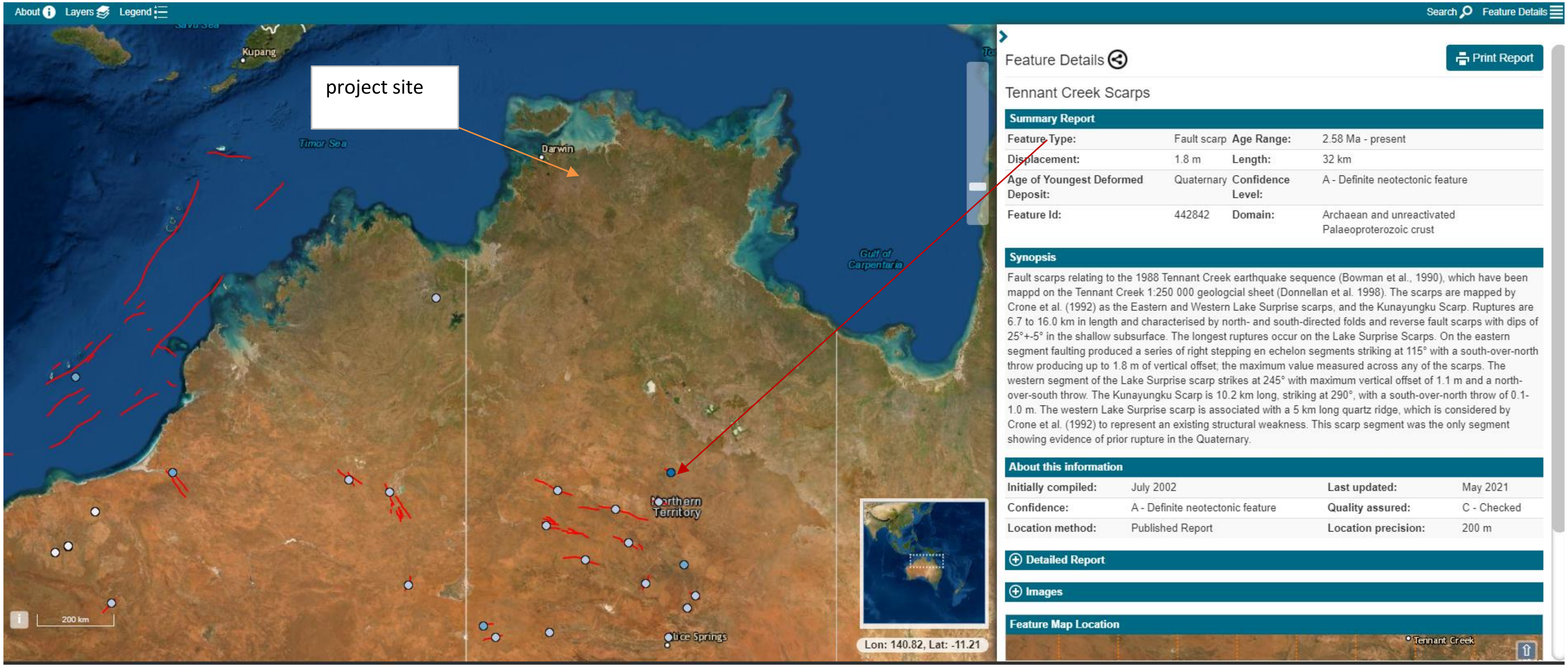


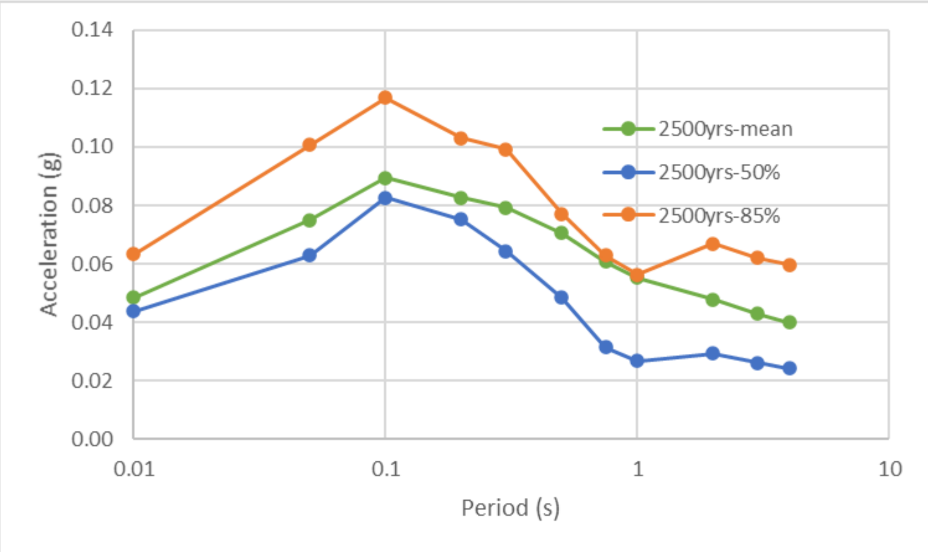
>400km

>400km

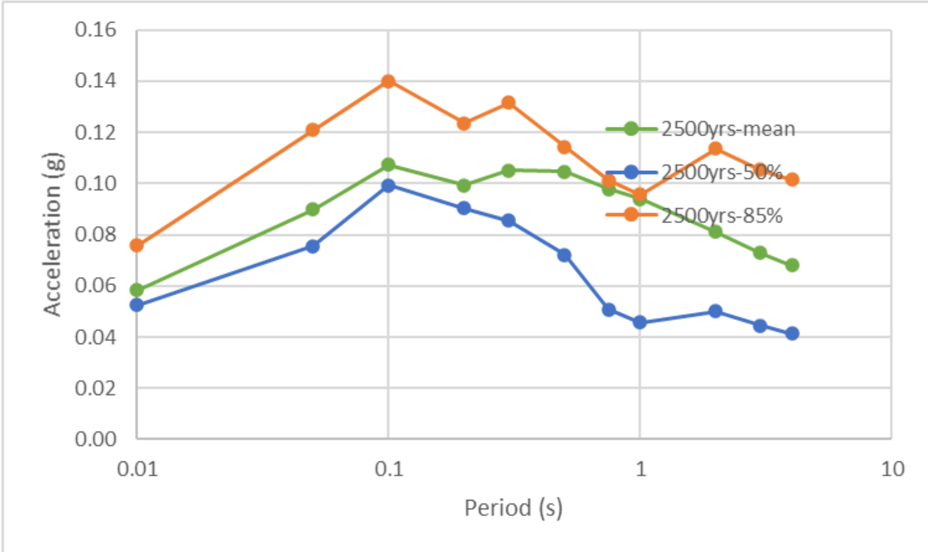
PGA

0.5 second

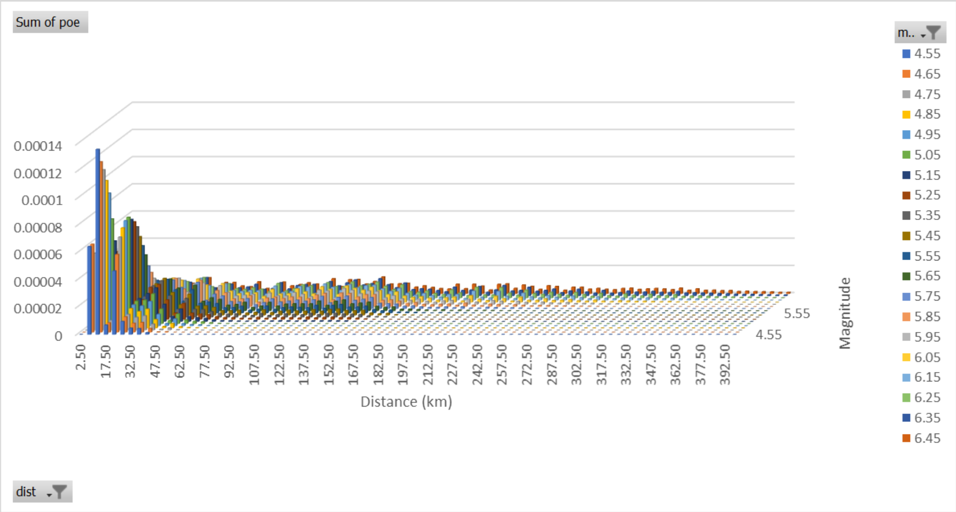

About
Layers
Legend
Search
Feature Details
project site
Kupang
Timor Sea
Darwin
Gulf of Carpentaria
Northern Territory
Alice Springs
200 km
Lon: 140.82, Lat: -11.21
Feature Details
Print Report
Tennant Creek Scarps
Summary Report
Feature Type: Fault scarp
Age Range: 2.58 Ma - present
Displacement: 1.8 m
Length: 32 km
Age of Youngest Deformed Deposit: Quaternary
Confidence Level: A - Definite neotectonic feature
Feature Id: 442842
Domain: Archaean and unreactivated Palaeoproterozoic crust
Synopsis
Fault scarps relating to the 1988 Tennant Creek earthquake sequence (Bowman et al., 1990), which have been mappd on the Tennant Creek 1:250 000 geologcial sheet (Donnellan et al. 1998). The scarps are mapped by Crone et al. (1992) as the Eastern and Western Lake Surprise scarps, and the Kunayungku Scarp. Ruptures are 6.7 to 16.0 km in length and characterised by north- and south-directed folds and reverse fault scarps with dips of 25°+-5° in the shallow subsurface. The longest ruptures occur on the Lake Surprise Scarps. On the eastern segment faulting produced a series of right stepping en echelon segments striking at 115° with a south-over-north throw producing up to 1.8 m of vertical offset; the maximum value measured across any of the scarps. The western segment of the Lake Surprise scarp strikes at 245° with maximum vertical offset of 1.1 m and a north-over-south throw. The Kunayungku Scarp is 10.2 km long, striking at 290°, with a south-over-north throw of 0.1-1.0 m. The western Lake Surprise scarp is associated with a 5 km long quartz ridge, which is considered by Crone et al. (1992) to represent an existing structural weakness. This scarp segment was the only segment showing evidence of prior rupture in the Quaternary.
About this information
Initially compiled: July 2002
Last updated: May 2021
Confidence: A - Definite neotectonic feature
Quality assured: C - Checked
Location method: Published Report
Location precision: 200 m
Detailed Report
Images
Feature Map Location
Tennant Creek

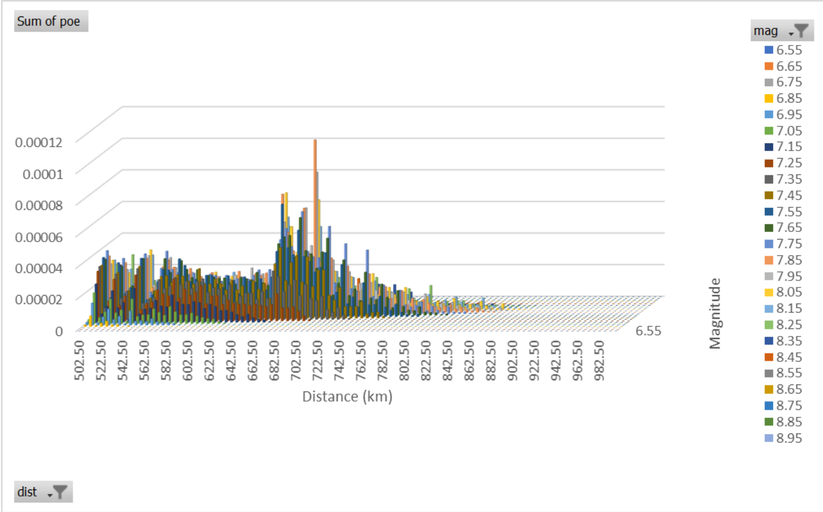


Site Class B



Site Class C





0.2 second

1 second

APPENDIX C

Memorandum Ref. PE21-01554, Rustlers Roost Gold Project –Tailings Physical Testing

MEMORANDUM

To:	Hanking Australia Investment	Date:	08 December 2021
Attn:	John Zimmerman	Our Ref:	PE21-01554
		KP File Ref.:	PE801-00102/04-A dss M21006
cc:	Charles Hastie	From:	Dean Sawyer/Dave Morgan

RE: RUSTLERS ROOST GOLD PROJECT – TAILINGS PHYSICAL TESTING

1. INTRODUCTION

Knight Piésold (KP) was engaged by Hanking Australia Investment (Hanking) to undertake the Pre-Feasibility study at the Rustlers Roost project in the northern region of the Northern Territory. The physical tests were performed to determine the rate of settling, achieved density and rate of water release of the tailings. In addition, air drying, permeability and consolidation tests were performed.

2. SAMPLE DESCRIPTION AND TEST WORK

Two 10 kg bags of dry tailings sample were sent to the KP laboratory in Perth from ALS Metallurgy in July 2021. The samples were both labelled "*Hanking RR Tailings Composite*" and were assume to be the same. A portion of each of the two bags were combined with Perth tap water as instructed by Hanking to achieve the target percent solids of 60% solids w/w. The sample is referred to as "*Rustlers Roost Composite*" for the purposes of this memorandum.

The following tests were carried out on the sample:

I. Classification tests to determine:
   - Particle size distribution of the tailings;
   - Supernatant liquor density and pH;
   - Tailings solids particle density;
   - Atterberg Limits of the tailings solids;

II. Undrained and drained sedimentation tests;

III. Air drying tests;

IV. Permeability tests; and

V. Consolidation tests.

During laboratory testing it is Knight Piésold's normal practice to duplicate each test as a means to verify the consistency of the test results. The results of each individual test are plotted on the corresponding figures. The interpreted mean values are given in the tables and text of the document. A brief description of the method employed in each test is also provided.

3. PHYSICAL TESTING

The following section presents the physical testing results for the tailings sample received. Predicted tailings behaviour is discussed in Section 4, however, it should be noted that tailings behaviour in the field is also dependent on the layout, height and design of the storage facility, climatic conditions on site and the operating parameters of the processing plant. Hence it is recommended that detailed water balance and associated density modelling is conducted as part of the design phase to enable a full assessment of the likely performance of the facility.

3.1 CLASSIFICATION TESTING

Classification testing for the sample was completed by Trilab in Perth. Where appropriate, classification tests were conducted in accordance with relevant Australian Standards. The results of the classification tests and relevant Australian Standards are summarised in Table 3.1. The Trilab laboratory test reports are presented in Appendix A.

Table 3.1: Classification testing – results and relevant standards

Test	Rustlers Roost Composite	AS1289
Solids Particle Density (t/m^3)	2.86	3.5.1
Supernatant Density (t/m^3)	1.000	(hydrometer)
Supernatant pH	7[1]	(pH meter)
Liquid Limit (%)	28	3.9.2
Plastic Limit (%)	19	3.2.1
Plasticity Index (%)	9	3.3.1
Linear Shrinkage (%)	3.5*	3.4.1

[1] Perth Tap Water
*Cracking Occurred

The particle size analysis for the tailings sample was completed in accordance with AS1289 3.6.3. The measured particle size distribution is presented in Table 3.2 and the grading curve for the sample is shown on Figure 3.1.

Table 3.2: Particle size distribution – Rustlers Roost Composite

Fraction	Particle Size (µm)	Percent Passing (%)
Sand	600	100
	200	99
Silt	75	81
	20	54
	6	36
Clay	2	15

The Rustlers Roost Composite sample consisted of 19% sand, 66% silt and 15% clay sized material. The testing indicates that the material is SILT with sand and clay and would be classified as CL with low to medium plasticity according to Geotechnical Site Investigation Standard AS1726-2017. The sample's P_{80} is approximately 70 µm.

The grading curve indicates that the majority of the sample falls inside the boundary of potentially liquefiable soil and therefore liquefaction of tailings should be considered in design.

3.2 SEDIMENTATION TESTING

Drained and undrained sedimentation tests were carried out to determine the settling rate, volume of supernatant, and settled dry density of the tailings.

In the undrained sedimentation test, tailings slurry is allowed to settle in a measuring cylinder. This is equivalent to the deposition of tailings under water. The results indicate the expected rate and quantity of supernatant release and enable the minimum expected dry density of the tailings to be determined.

In the drained sedimentation test, tailings slurry is allowed to settle and drain in a cylinder with a fine sand filter drain at the base. This simulates the deposition of tailings where both settling and free drainage can occur. The results indicate the relative quantities of supernatant and underdrainage released by the settling slurry and enables the dry density of the drained tailings to be determined. The underdrainage values are maximum values, as the drainage layer is free-draining without back pressure and the tailings is deposited directly over the drainage medium.

The results of the sedimentation tests are presented in figures 3.2 and 3.3. Table 3.3 presents a summary of the measured sedimentation data.

Table 3.3: Sedimentation test results

Sample	Test	Initial Solids (%)	Supernatant (% of initial water volume)	Underdrainage (% of initial water volume)	Time to Achieve Final Density (Days)	Final Dry Density (t/m^3)	Final Void Ratio	Figure
Rustlers Roost Composite	Undrained	58	37	-	3	1.24	1.31	3.2
	Drained	59	26	22	3.5	1.39	1.06	3.3

The undrained test indicated that the Rustlers Roost Composite sample is moderate settling, taking approximately three days to complete the majority of the settlement and water release. There is approximately a 12% increase in the settled density with drainage.

The sample released approximately 37% of water in slurry to supernatant in the undrained test, reducing to 26% in the drained test. The sample achieved a reasonable dry density from settlement before air drying or consolidation, approaching a void ratio of 1.0.

3.3 AIR DRYING TESTS

Air drying tests were carried out on slurry samples to determine the effect of natural drying of the tailings after initial settling and removal of supernatant liquor, thereby simulating conditions expected following sub-aerial deposition. Continuous monitoring of the weight and volume of each specimen was carried out in order to quantify the relationship between dry density, moisture content, volumetric change and the degree of saturation of the tailings against a measured evaporation rate.

A direct relationship exists between dry density and moisture content up to a breakaway point, at which the degree of saturation falls below 100%. At this point, negative pore water pressures are developed, which further consolidates the tailings.

Drying below a limiting saturation produces no further consolidation, and the density at this point represents the maximum that can be achieved via air drying of the tailings. The results of air drying tests are presented in figures 3.4 and 3.5 and are summarised in Table 3.4.

Table 3.4: Results of air drying tests

Sample	Moisture Content at Breakaway Point (%MC)	Dry Density at Breakaway Point (t/m^3)	Limiting Saturation Value (%Sat)	Final Dry Density (t/m^3)	Final Void Ratio	Figures
Rustlers Roost Composite	25	1.60	90	1.69	0.70	3.4, 3.5

The Rustlers Roost Composite sample achieved a final dry density of 1.69 t/m^3 after around 5 days of air drying at an evaporation rate of about 8.4 mm/day (total evaporation of about 40 mm). There is 22% improvement in density over the drained results with the void ratio reducing to around 0.70.

3.4 CONSOLIDATION TEST

The consolidation of the tailings can be quantified in terms of the compression index C_C and the coefficient of consolidation C_v. The compression index relates the void ratio or tailings density to the effective stress of the tailings sample. The larger the value of C_C, the more compressible the tailings are. The coefficient of consolidation defines the rate of excess pore water dissipation, and hence the rate of increase in effective stress within the tailings. Higher values of C_V indicate more rapid consolidation of the sample.

The settlement with respect to time for the test is presented in Figure 3.6 and the results of the consolidation tests are summarised in Table 3.5.

Table 3.5: Consolidation test results

Test	Dry Density (t/m^3)	Void Ratio	Stress Range (kPa)	Coeff. of Consolidation C_V (m^2/y)	Coeff. of Volume Decrease M_V (m^2/kN)	Comp. Index C_C
Rustlers Roost Composite	1.20 - 1.29	1.39 – 1.22	2.06 - 5.51	27.8	0.020	0.393

These results indicate the Rustlers Roost Composite sample has medium compressibility and will consolidate moderately quickly under the self-weight of additional deposition.

3.5 PERMEABILITY TEST

Falling head permeability tests were completed on saturated tailings samples with drainage through the drained sedimentation sample being measured. In addition, permeability values were derived from the results of consolidation tests. Measured permeability data are summarised in Table 3.6.

Table 3.6: Permeability test results

Sample	Test Type	Dry Density (t/m^3)	Permeability (m/s)
Rustlers Roost Composite	Falling Head Test	1.39	1×10^{-7}
	Consolidation Test	1.21	2×10^{-6}
		1.25	3×10^{-7}
		1.29	1×10^{-7}

These results represent the vertical permeability of saturated tailings prior to additional consolidation due to additional deposition loading or negative suction due to air-drying. In the range of expected settled densities, the vertical permeability of the Rustlers Roost Composite sample is approximately 1×10^{-7} m/s. As the tailings consolidate, it is anticipated that the permeability may reduce by about another order of magnitude.

4. INTERPRETATION OF RESULTS

Based on the physical testing of the samples the behaviour of the tailings can be predicted. The testing has been undertaken at 60% solids w/w which is the design target for the operation. It should be noted that field results are also dependent on the processing plant operation, the layout and design of the TSF and site climatic conditions. The interpretation of the test results provided herein is based on these percentage solids.

4.1 WATER PRODUCTION

The release of water following deposition of the tailings can be estimated from the results of undrained and drained sedimentation tests. The rate of release will determine the amount of liquor available in the decant pond for collection and return to the process plant or release from the facility. The testing indicated that the rate of supernatant release is moderate, taking 3 days day to complete.

The expected supernatant release would be in the range of 26 to 37% of the water in slurry, not accounting for rainfall and evaporation but incorporating the loss of water to re-saturate lower tailings layers for the operating tailings.

The quantity of underdrainage release in the field would be expected to be lower than the values indicated by the test work, due to the thickness of the deposited tailings, further consolidation of the tailings, drying of intermediate layers and underdrainage collection efficiency. A recovery rate between 5% and 10% could be expected depending on the arrangement of underdrainage collection and basin treatment or lining.

4.2 TAILINGS DENSITY

The settled dry density of tailings deposited into the facility can be predicted from laboratory testing and an appropriate site water balance taking into account climatic conditions. It has been observed over a number of years that densities achieved in the field are generally lower that those obtained in the laboratory. In addition, field densities achieved are dependent on the area available for drying, site evaporation and the thickness of deposited layers.

Tests provided dry densities as follows:

- Undrained test 1.24 t/m^3;
- Drained test 1.39 t/m^3; and
- Air drying test 1.69 t/m^3.

The test results indicated that the tailings sample achieved high densities (void ratio around 1.2) from settling alone. There was an additional improvement due to air drying and potential consolidation, reaching a void ratio of 0.70.

With suitable underdrainage, a small supernatant pond and air drying of a large tailings beach, dry densities of 1.35 t/m^3 to 1.50 t/m^3 are expected over the life of the facility. Final densities are dependent on a number of aspects of TSF design such as rate of rise and water management. It is therefore recommended that water balance and density modelling are undertaken as part of the TSF design.

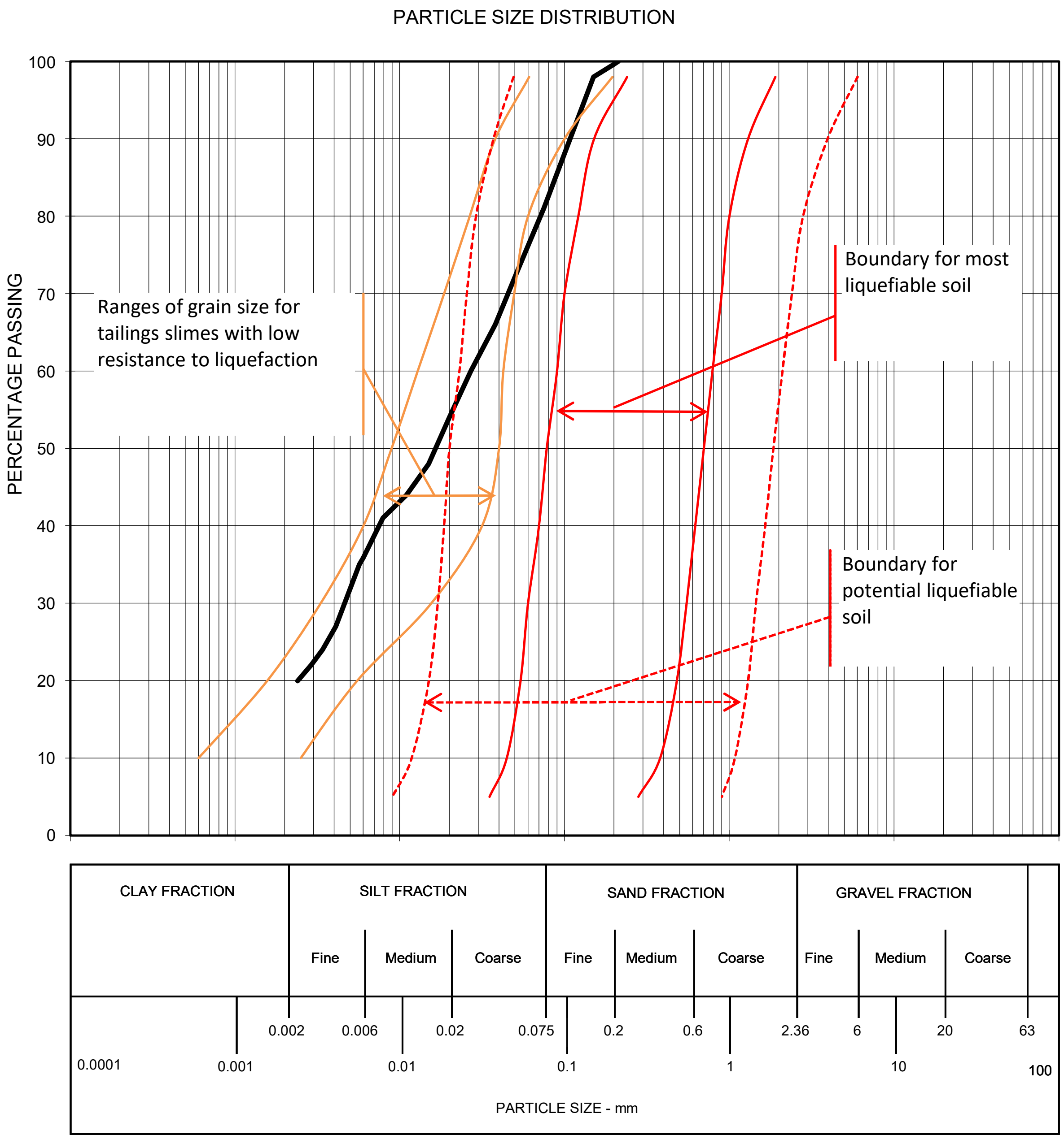
We trust this is sufficient information for your current requirements, however please contact us if you have any questions.

Yours faithfully
KNIGHT PIÉSOLD PTY LTD

DEAN SAWYER
Senior Engineer

DAVID MORGAN
Managing Director

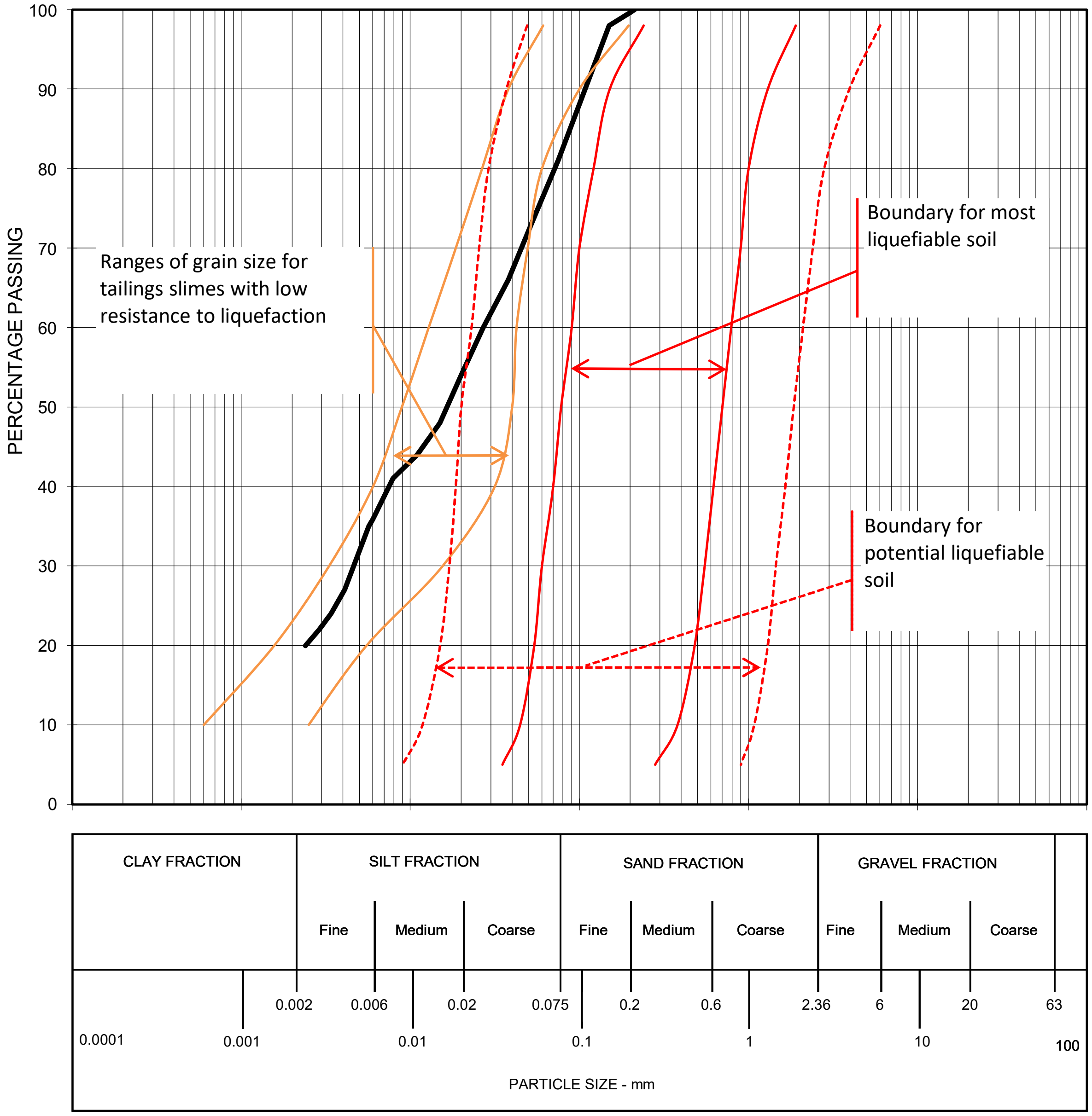
FIGURES



SYMBOL	SOIL DESCRIPTION	LIQUID LIMIT	PLASTICITY INDEX	SPECIFIC GRAVITY
▬▬	RUSTLERS ROOST COMPOSITE SAMPLE	28.00	9.00	2.86

Note:
Liquefiable and potentially liquefiable soil limits suggested by USNRC 1985.
Data obtained from Figure 12.15 and 12.16 of Geotechnical Engineering of Dams, Fell 2005

PARTICLE SIZE DISTRIBUTION

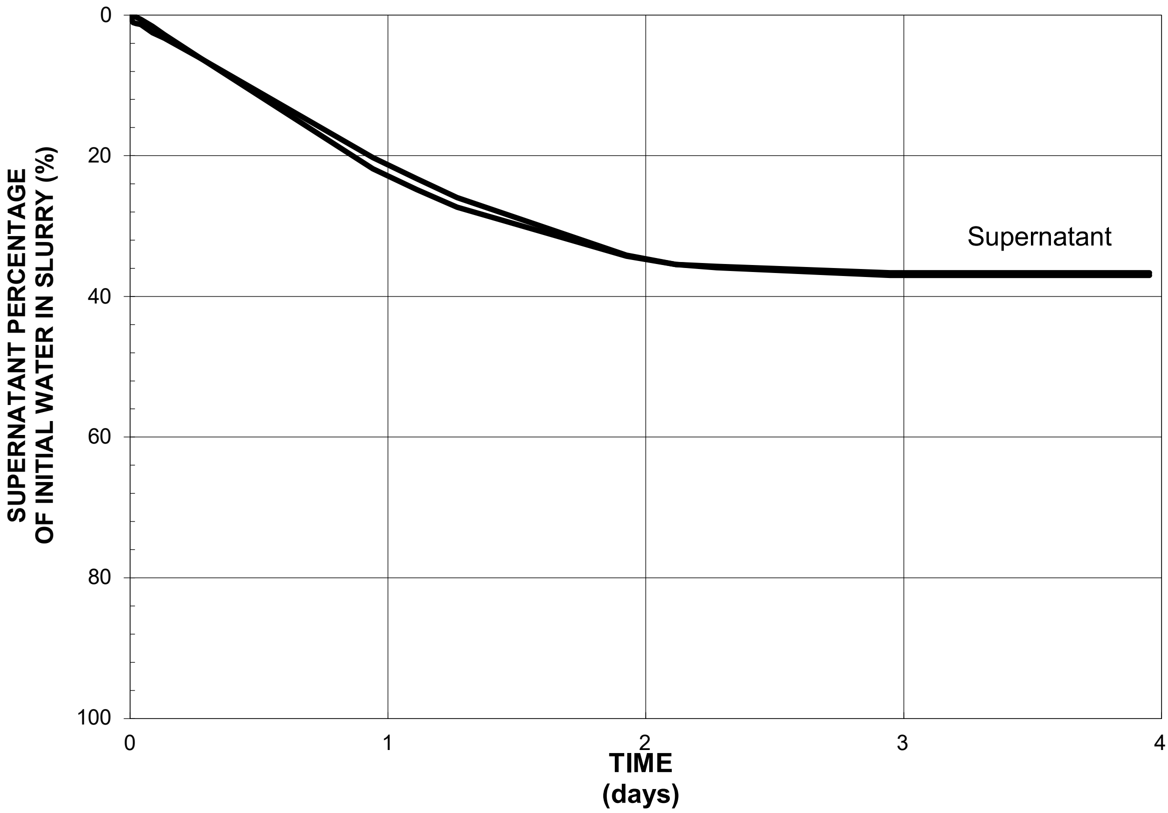


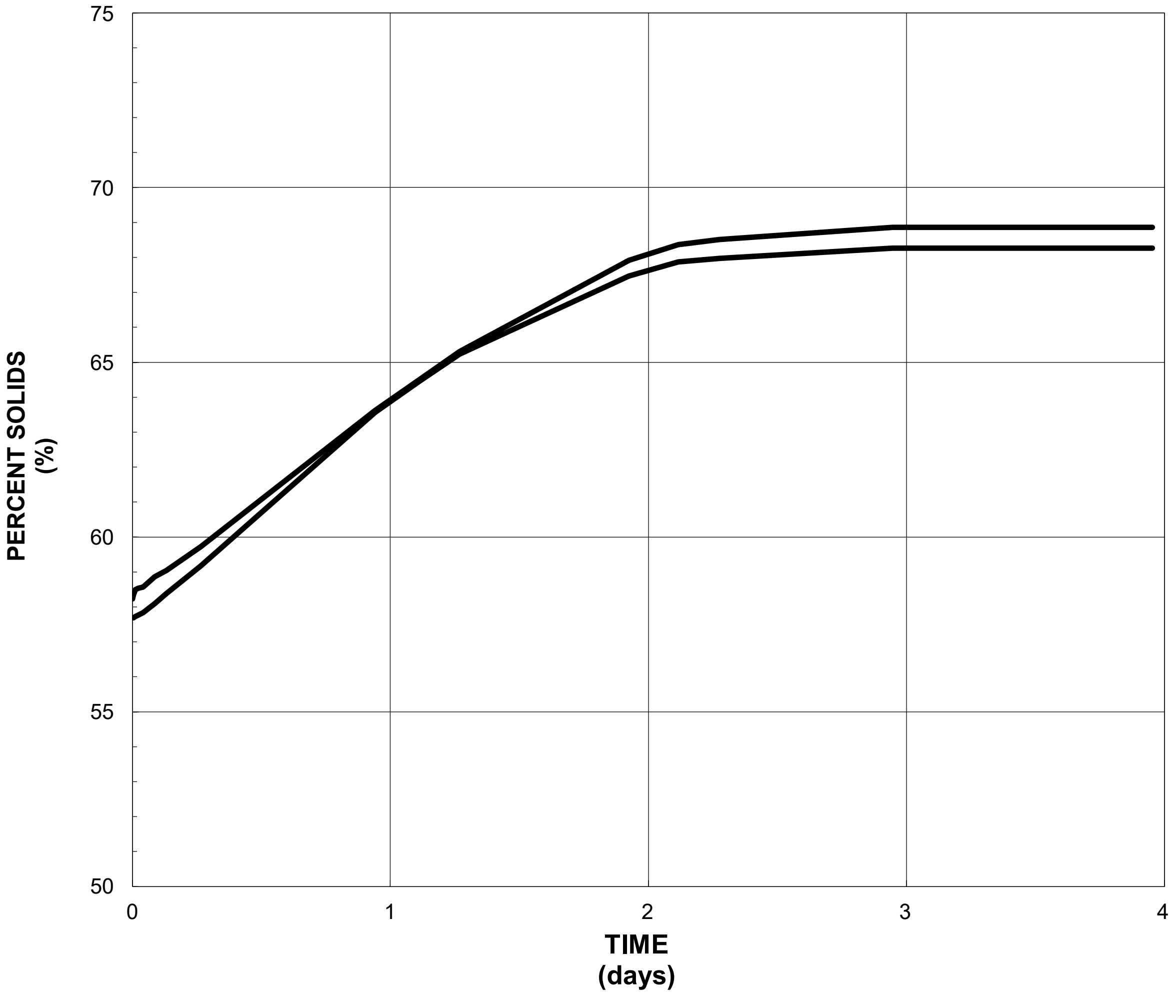
SYMBOL	SOIL DESCRIPTION	LIQUID LIMIT	PLASTICITY INDEX	SPECIFIC GRAVITY
▬▬	RUSTLERS ROOST COMPOSITE SAMPLE	28.00	9.00	2.86

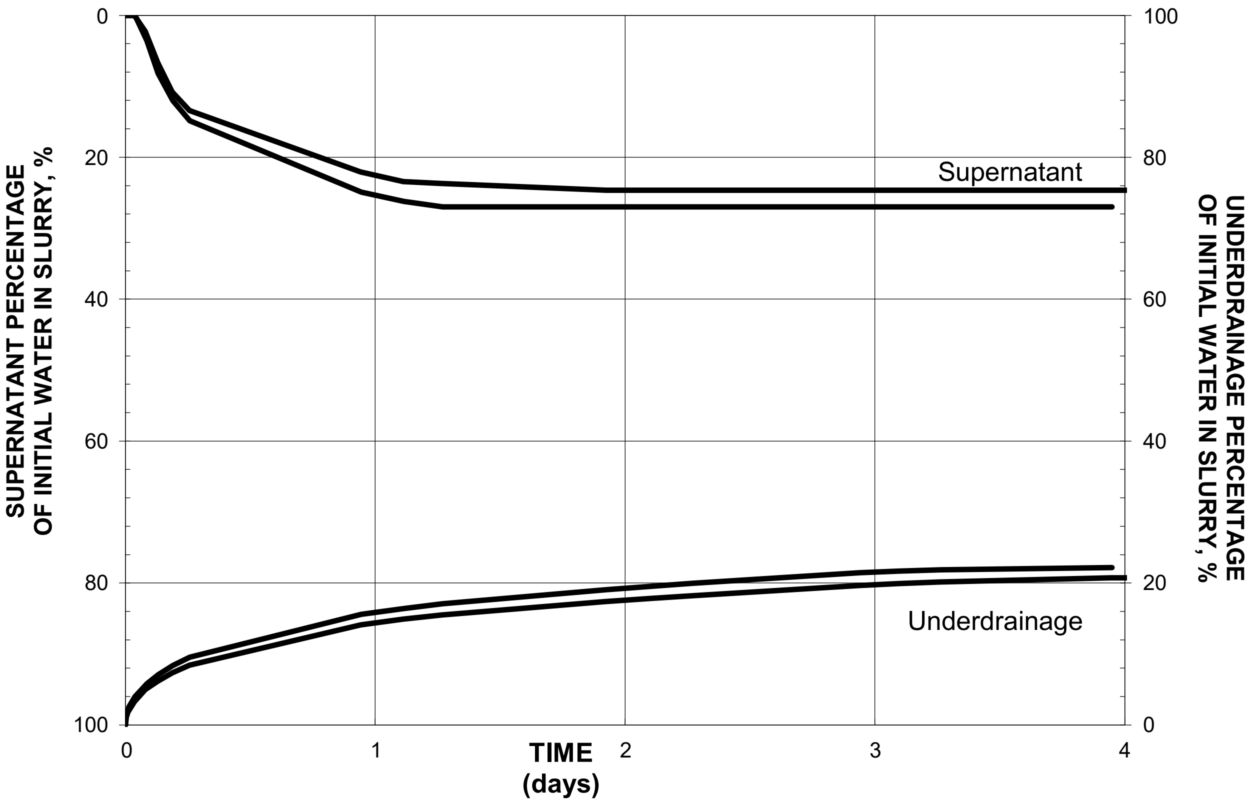
Note:

Liquefiable and potentially liquefiable soil limits suggested by USNRC 1985.

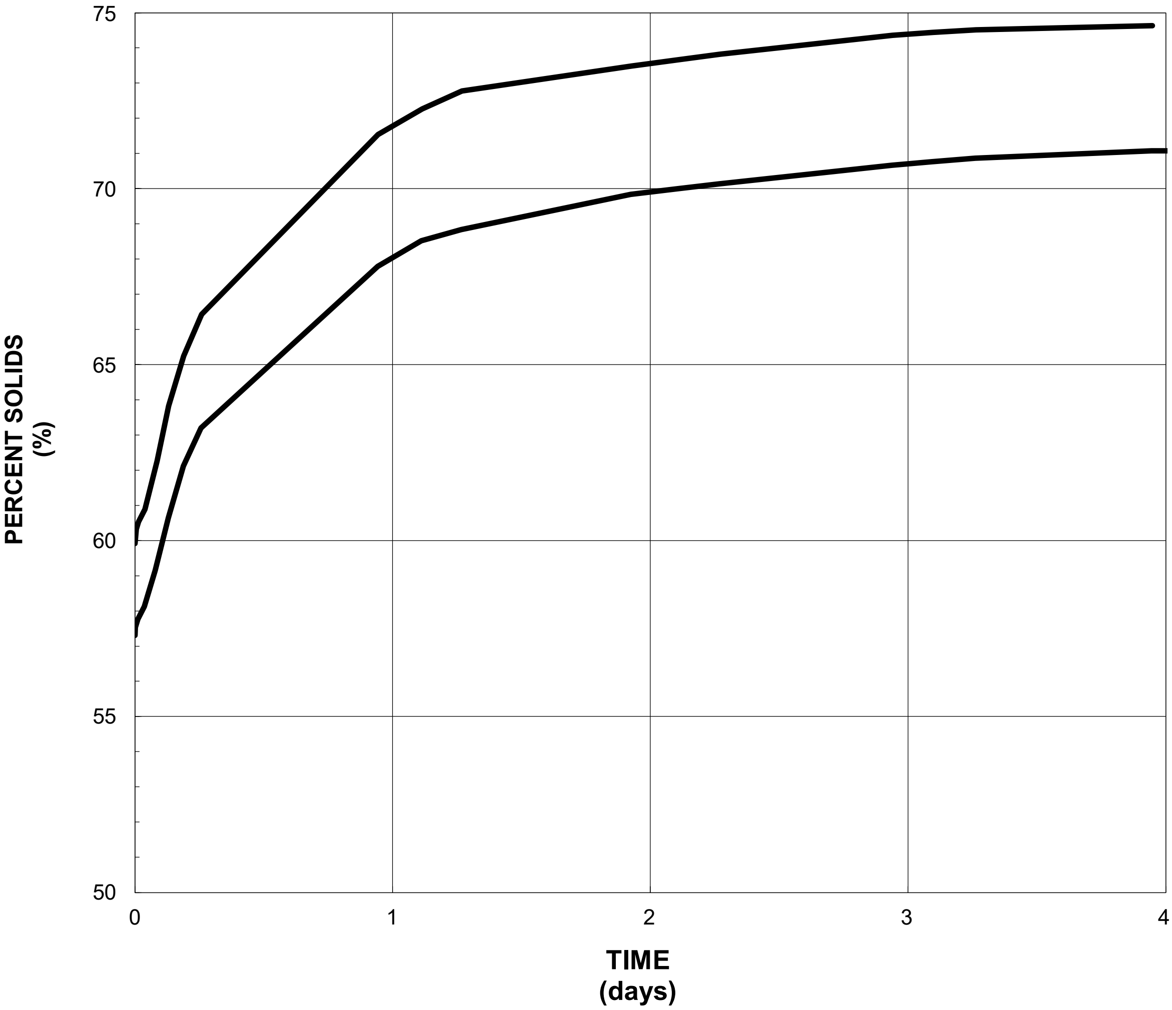
Data obtained from Figure 12.15 and 12.16 of Geotechnical Engineering of Dams, Fell 2005


0
20
40
60
80
100
SUPERNATANT PERCENTAGE
OF INITIAL WATER IN SLURRY (%)
Supernatant
0
1
2
3
4
TIME
(days)

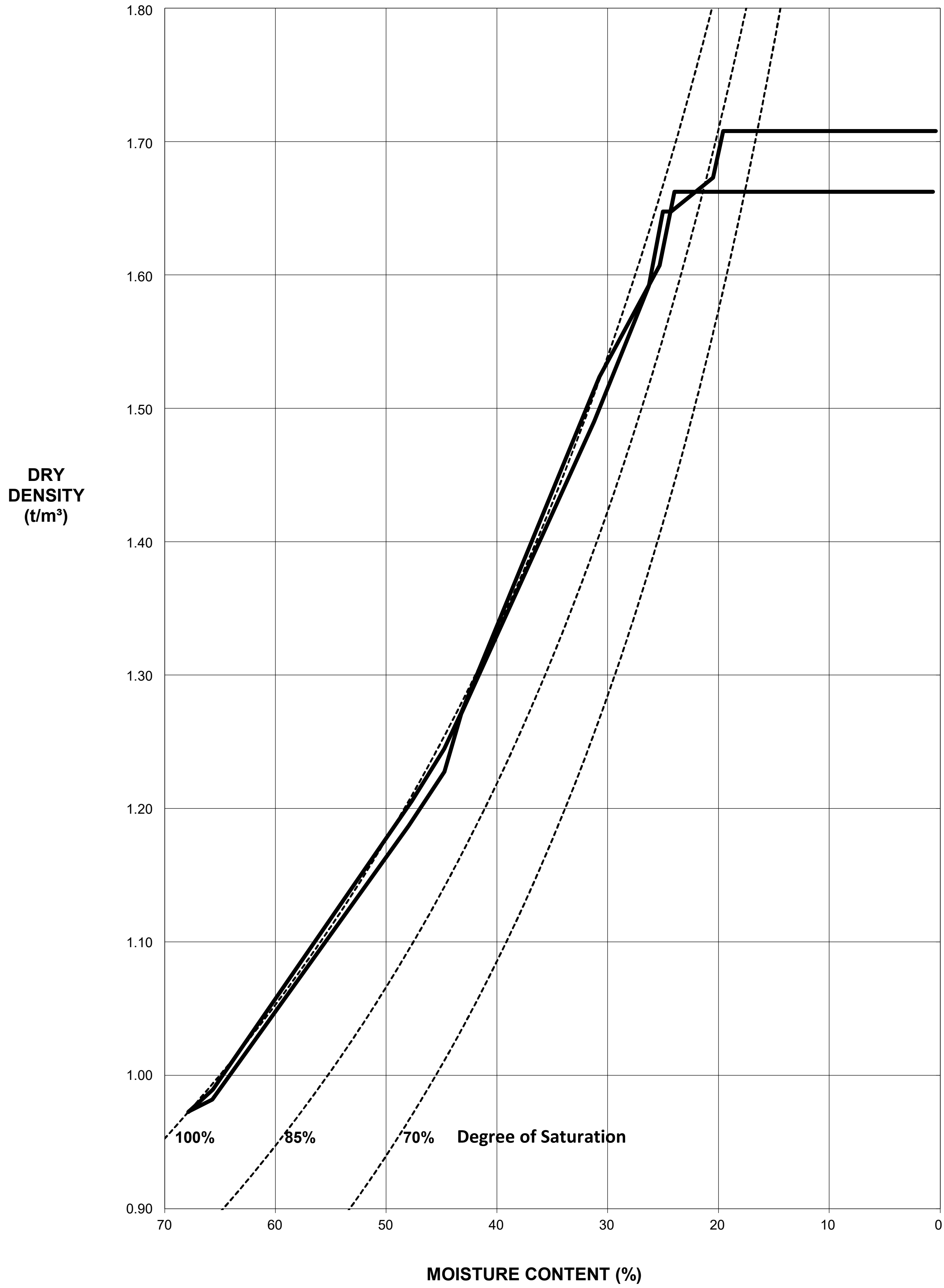

75
70
65
60
55
50
PERCENT SOLIDS
(%)
0
1
2
3
4
TIME
(days)

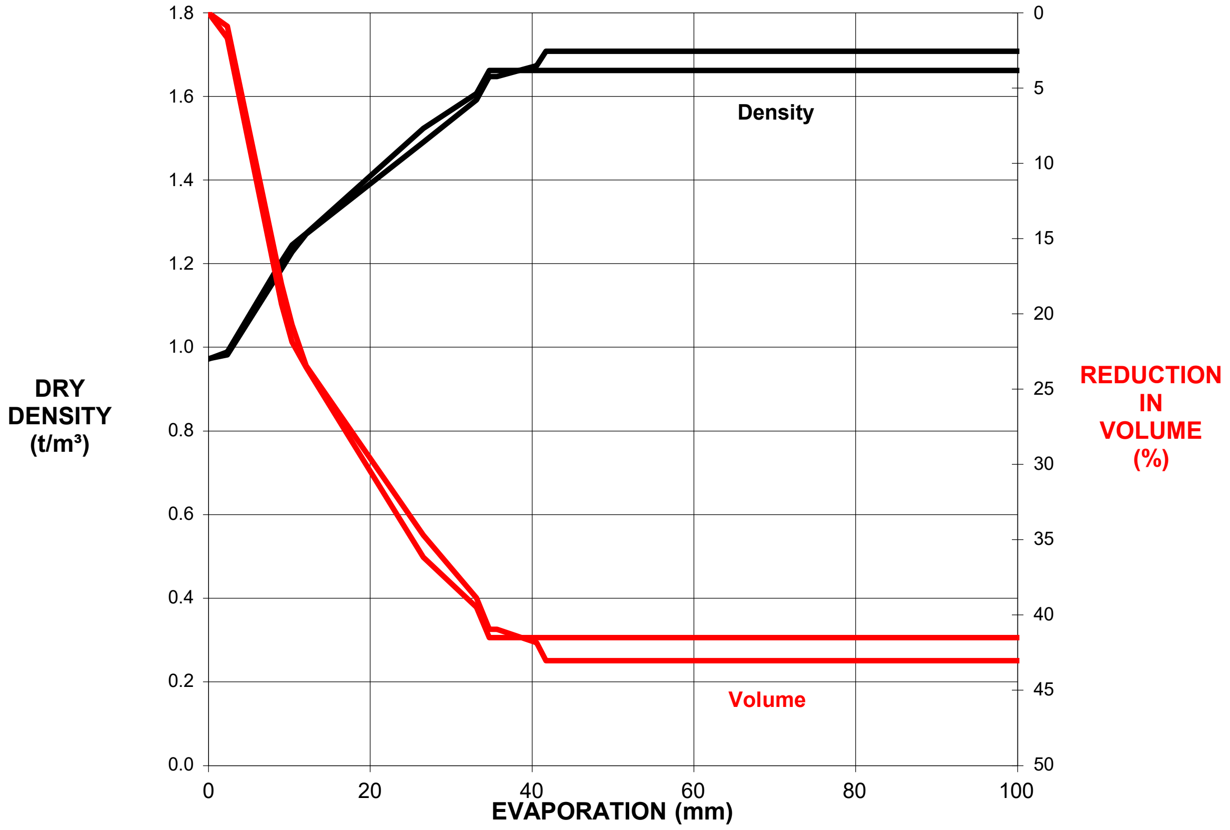


SUPERNATANT PERCENTAGE OF INITIAL WATER IN SLURRY, %
UNDERDRAINAGE PERCENTAGE OF INITIAL WATER IN SLURRY, %
Supernatant
Underdrainage
TIME (days)
0
20
40
60
80
100
0
1
2
3
4

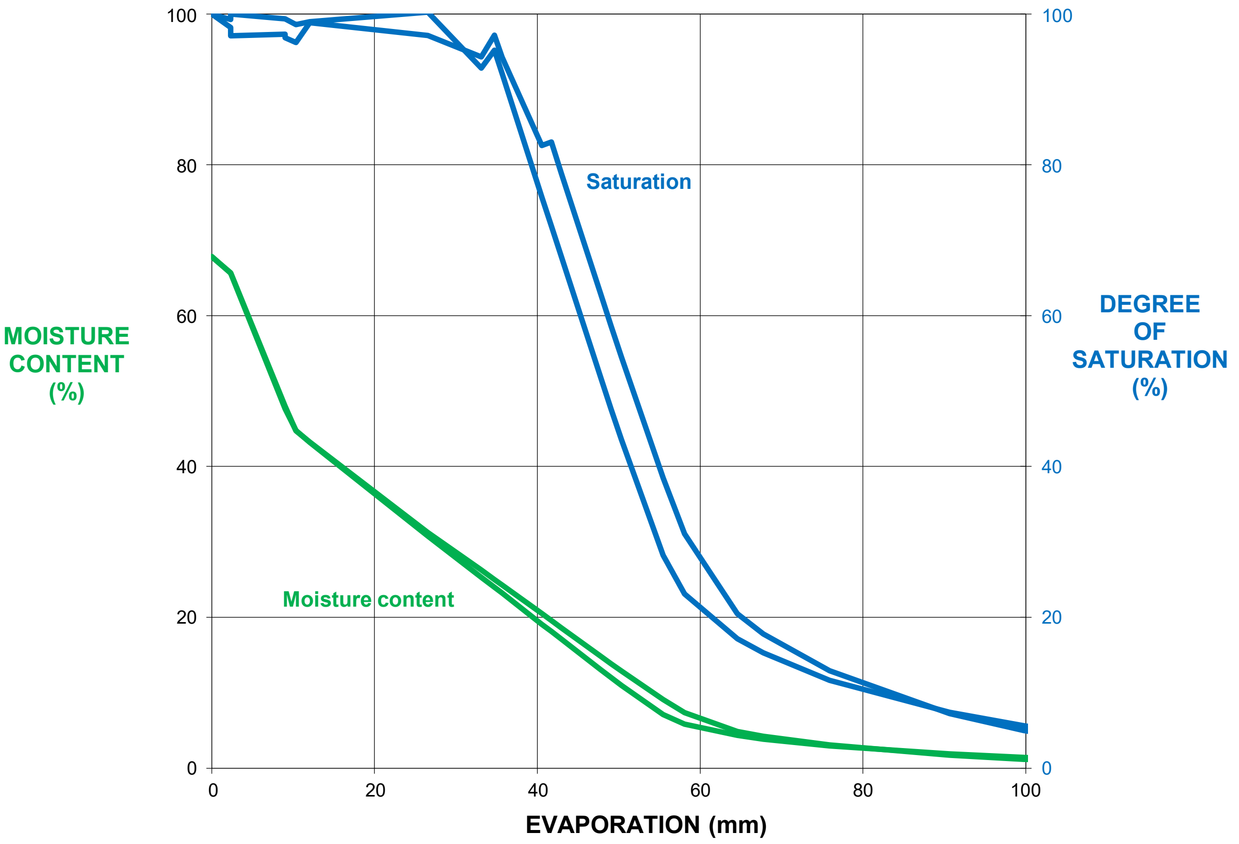


PERCENT SOLIDS (%)
TIME (days)
50
55
60
65
70
75
0
1
2
3
4

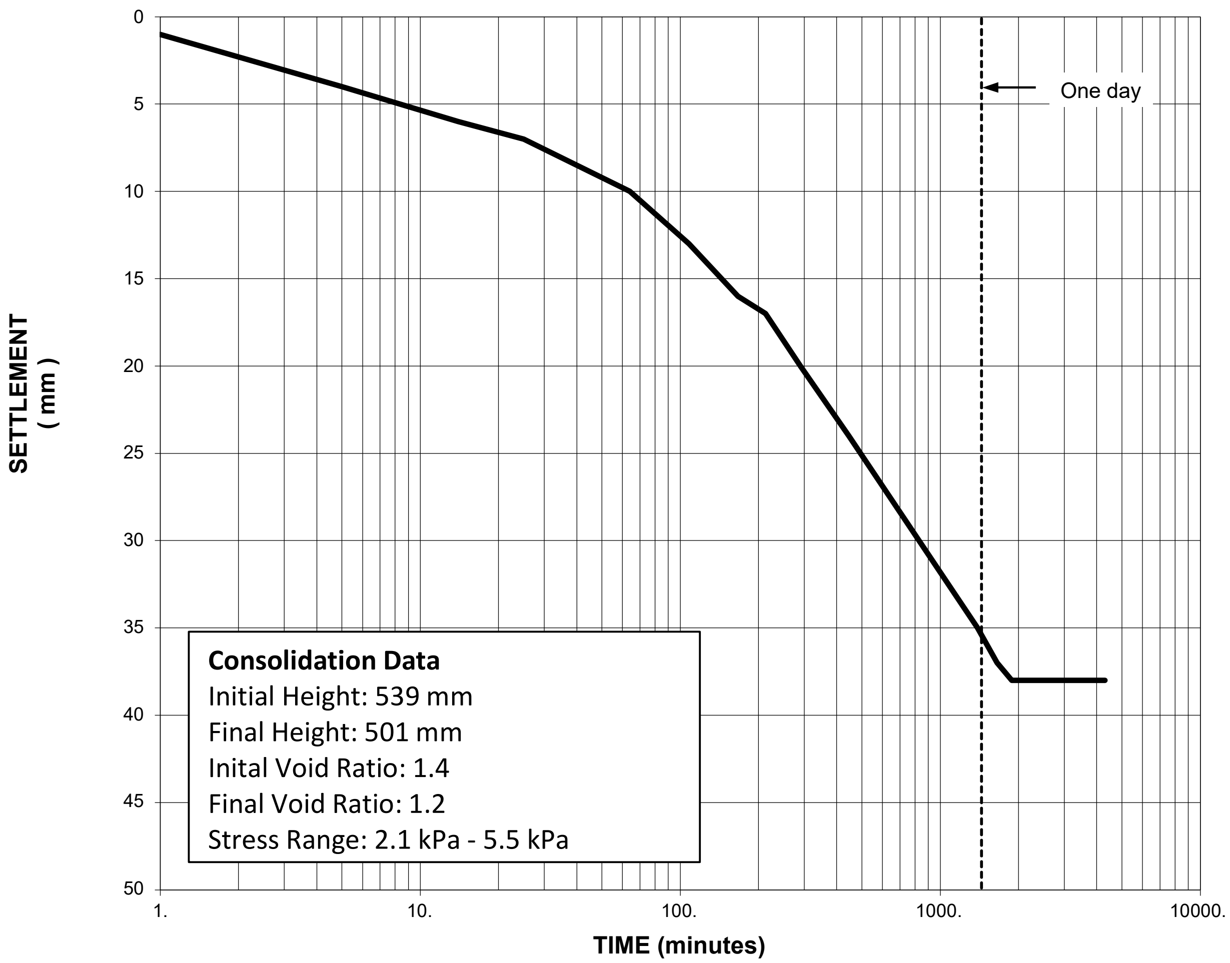

1.80
1.70
1.60
1.50
1.40
1.30
1.20
1.10
1.00
0.90
DRY DENSITY (t/m³)
100%
85%
70%
Degree of Saturation
70
60
50
40
30
20
10
0
MOISTURE CONTENT (%)

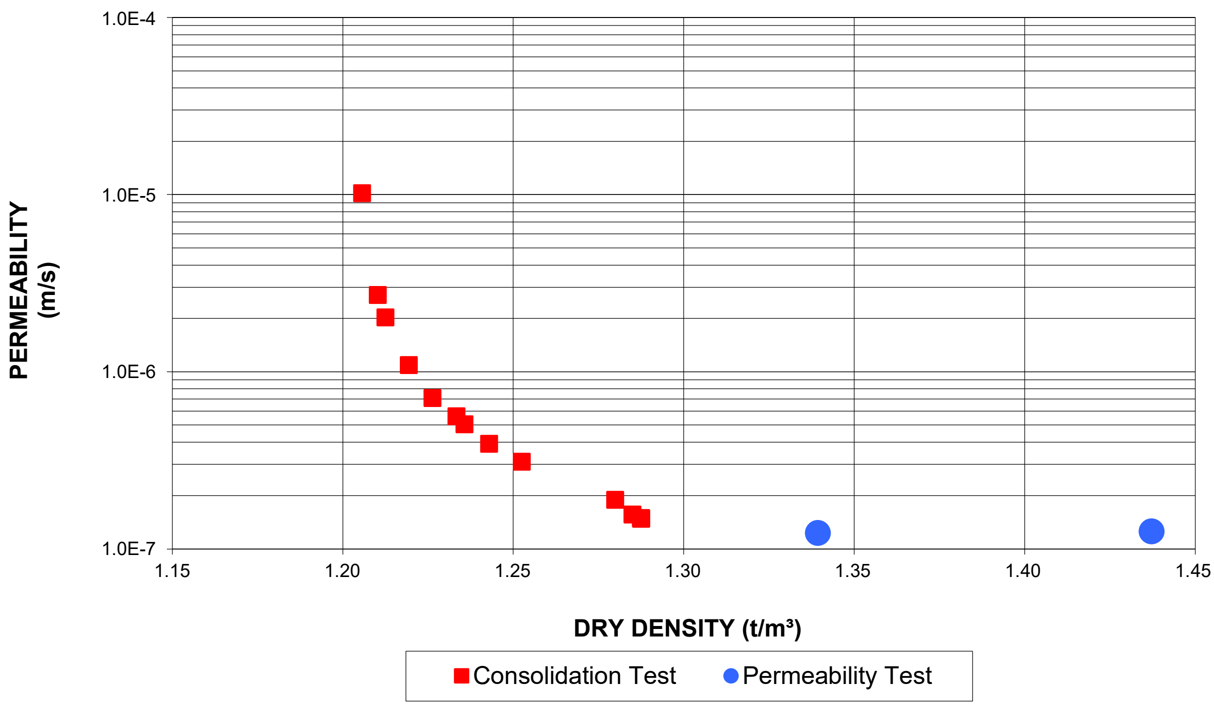


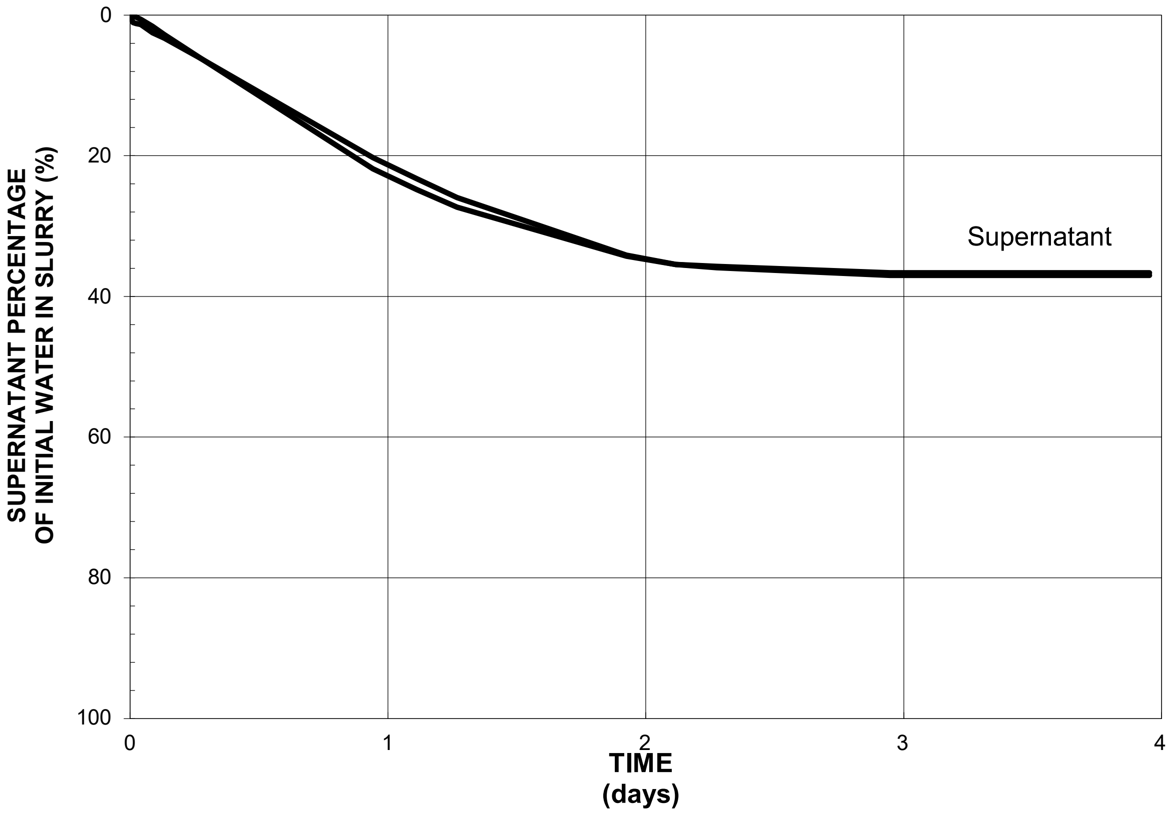
DRY DENSITY (t/m³)
REDUCTION IN VOLUME (%)
Density
Volume
EVAPORATION (mm)
1.8
1.6
1.4
1.2
1.0
0.8
0.6
0.4
0.2
0.0
0
5
10
15
20
25
30
35
40
45
50
0
20
40
60
80
100

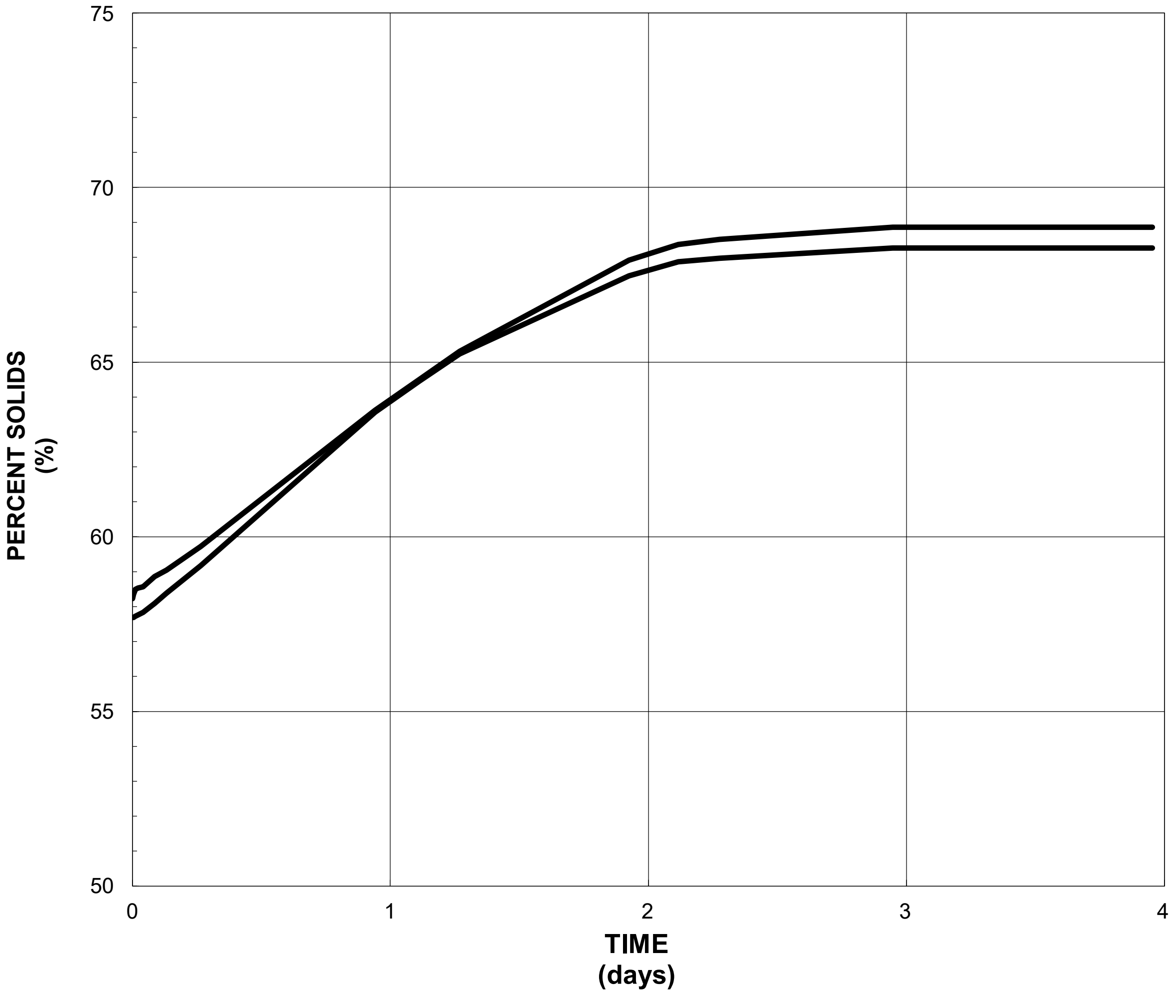


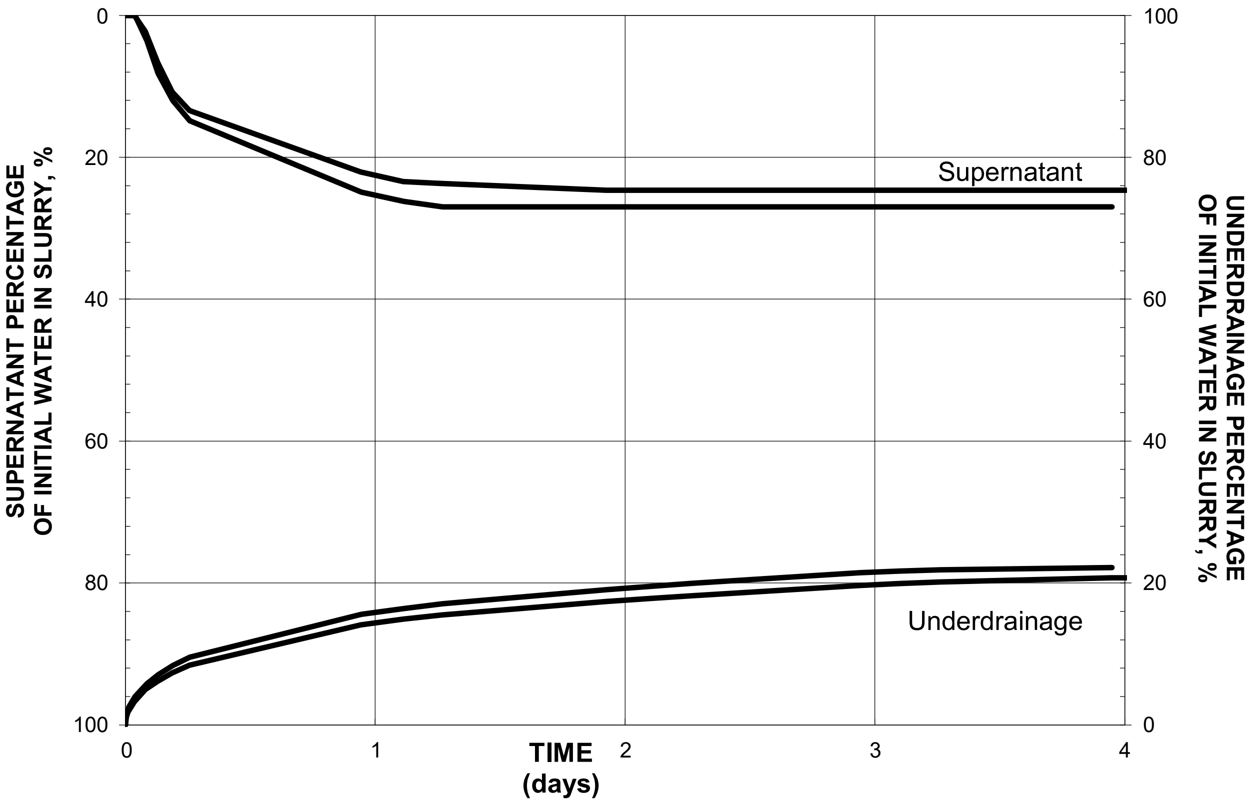
MOISTURE CONTENT (%)
DEGREE OF SATURATION (%)
Saturation
Moisture content
EVAPORATION (mm)
100
80
60
40
20
0
0
20
40
60
80
100


SETTLEMENT (mm)
0
5
10
15
20
25
30
35
40
45
50
1.
10.
100.
1000.
10000.
TIME (minutes)
One day
Consolidation Data
Initial Height: 539 mm
Final Height: 501 mm
Inital Void Ratio: 1.4
Final Void Ratio: 1.2
Stress Range: 2.1 kPa - 5.5 kPa

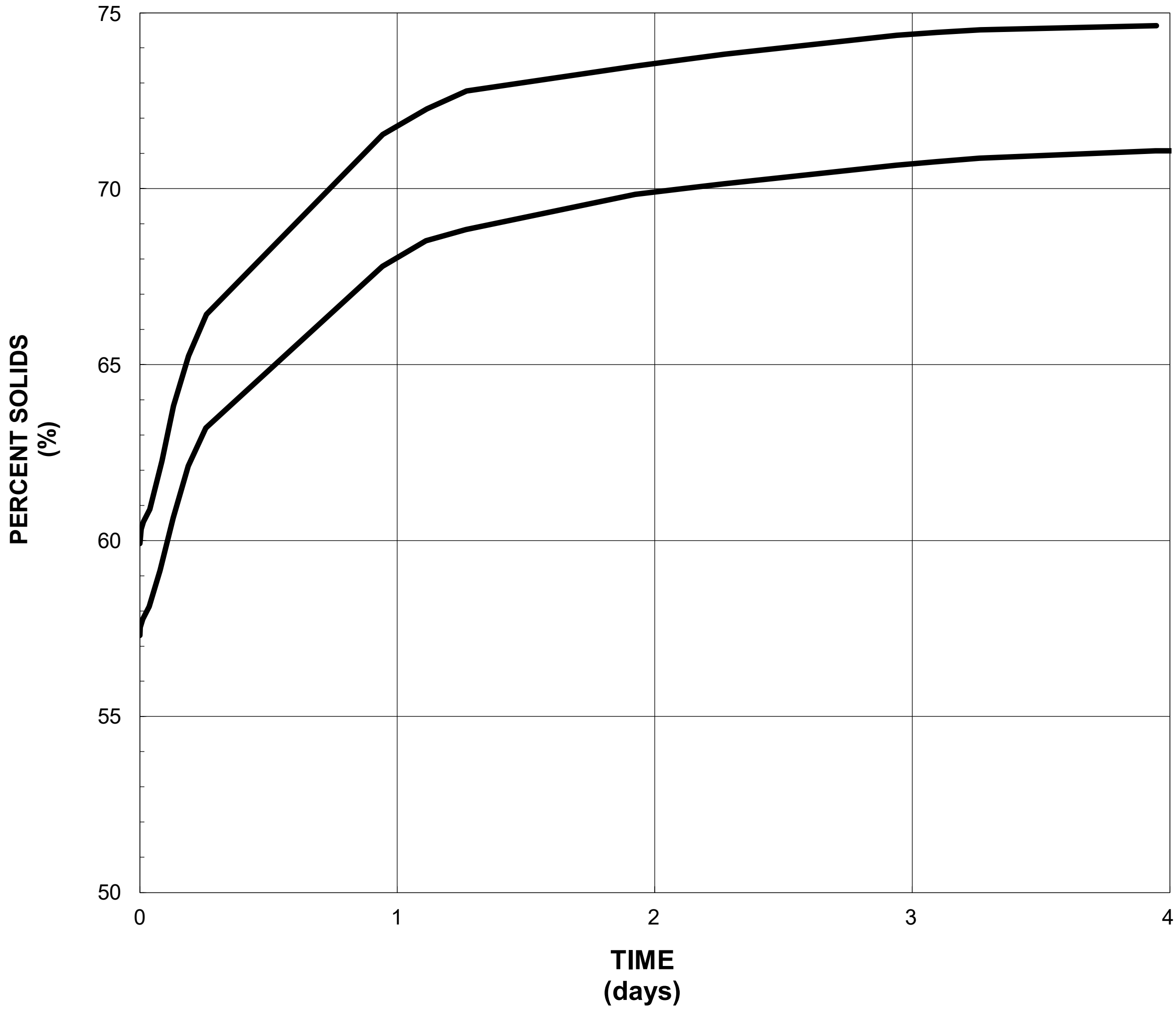

PERMEABILITY (m/s)
1.0E-4
1.0E-5
1.0E-6
1.0E-7
1.15
1.20
1.25
1.30
1.35
1.40
1.45
DRY DENSITY (t/m³)
Consolidation Test
Permeability Test


SUPERNATANT PERCENTAGE
OF INITIAL WATER IN SLURRY (%)
0
20
40
60
80
100
0
1
2
3
4
TIME
(days)
Supernatant

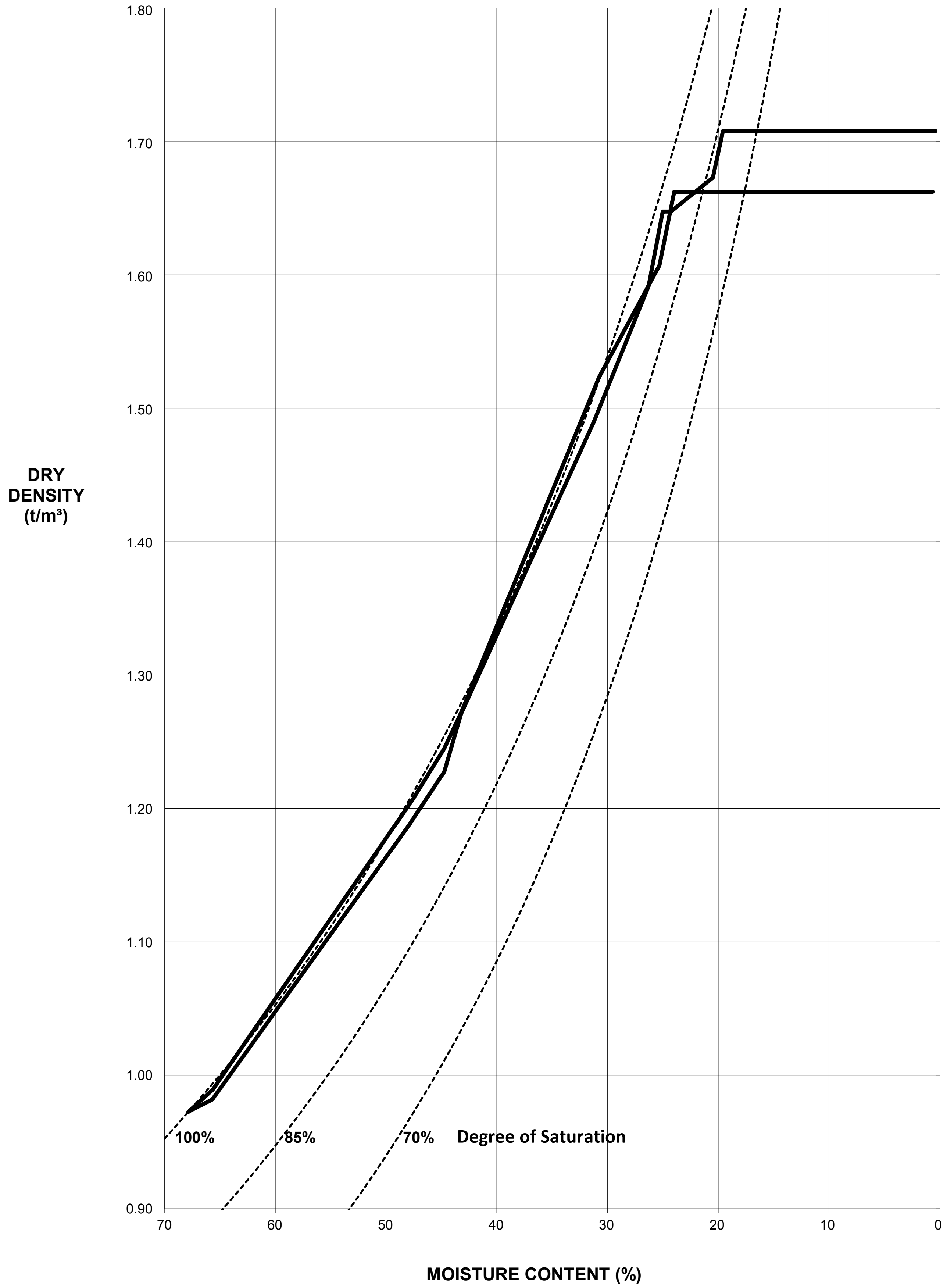

PERCENT SOLIDS
(%)
75
70
65
60
55
50
0
1
2
3
4
TIME
(days)

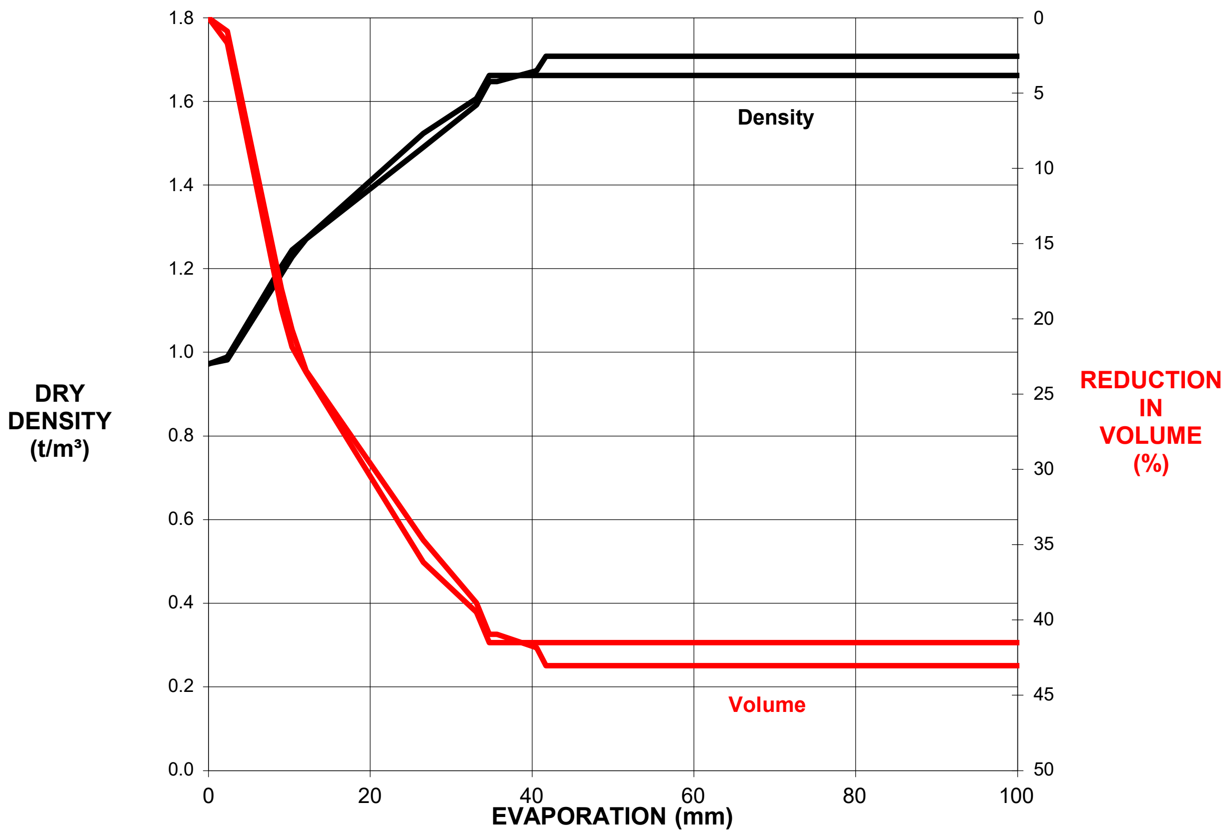


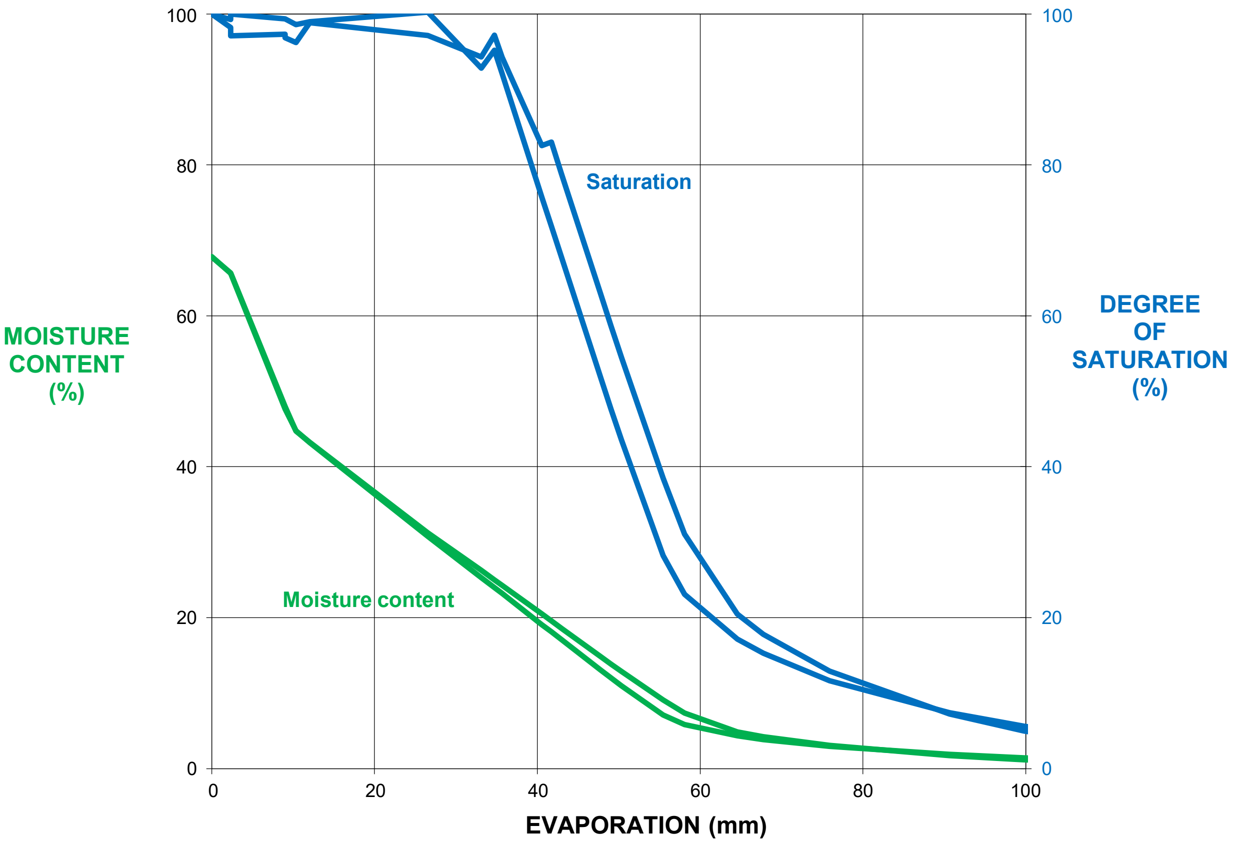
SUPERNATANT PERCENTAGE
OF INITIAL WATER IN SLURRY, %
UNDERDRAINAGE PERCENTAGE
OF INITIAL WATER IN SLURRY, %
0
20
40
60
80
100
100
80
60
40
20
0
Supernatant
Underdrainage
0
1
2
3
4
TIME
(days)



PERCENT SOLIDS
(%)
75
70
65
60
55
50
0
1
2
3
4
TIME
(days)

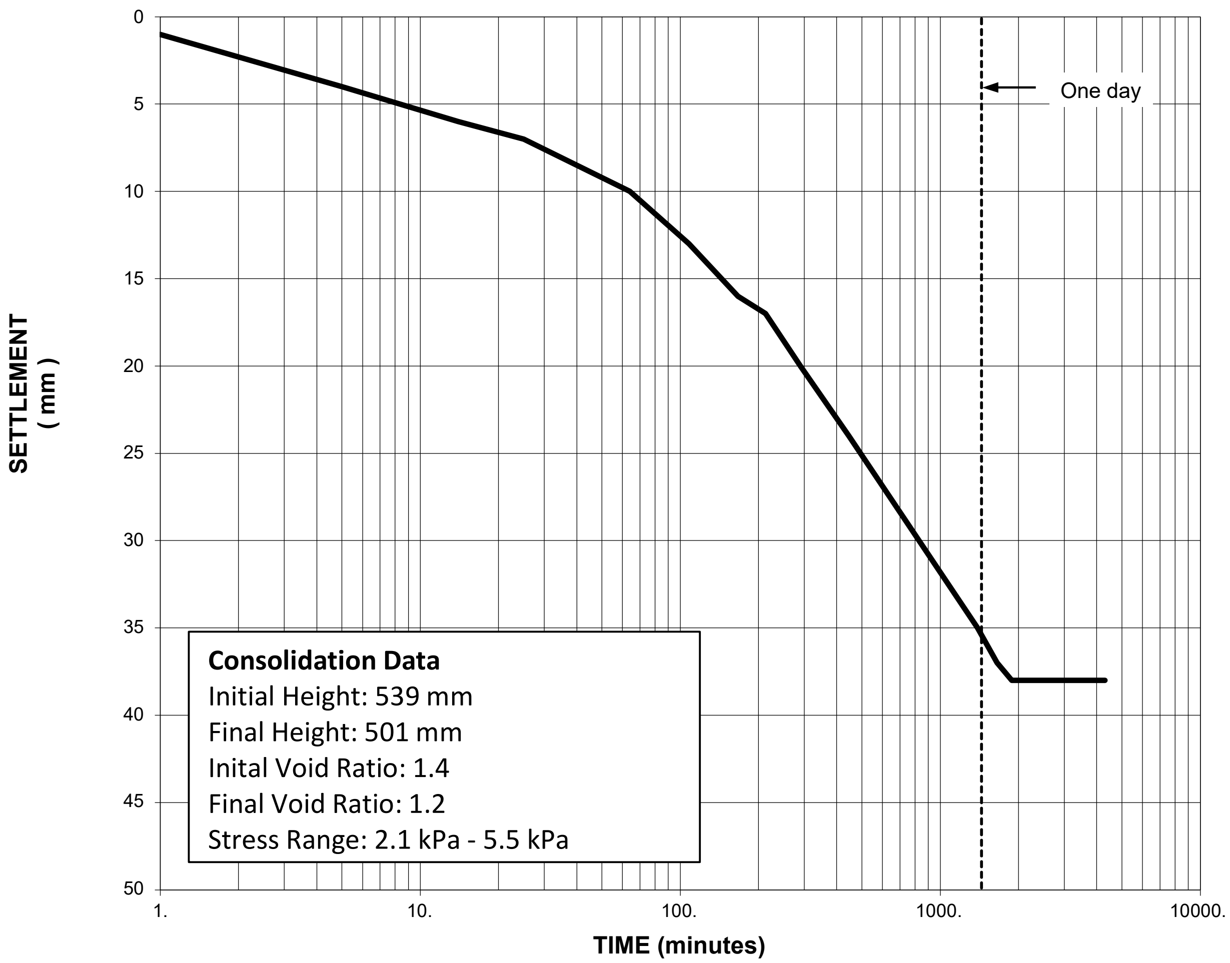

1.80
1.70
1.60
1.50
1.40
1.30
1.20
1.10
1.00
0.90
DRY DENSITY (t/m³)
100%
85%
70%
Degree of Saturation
70
60
50
40
30
20
10
0
MOISTURE CONTENT (%)



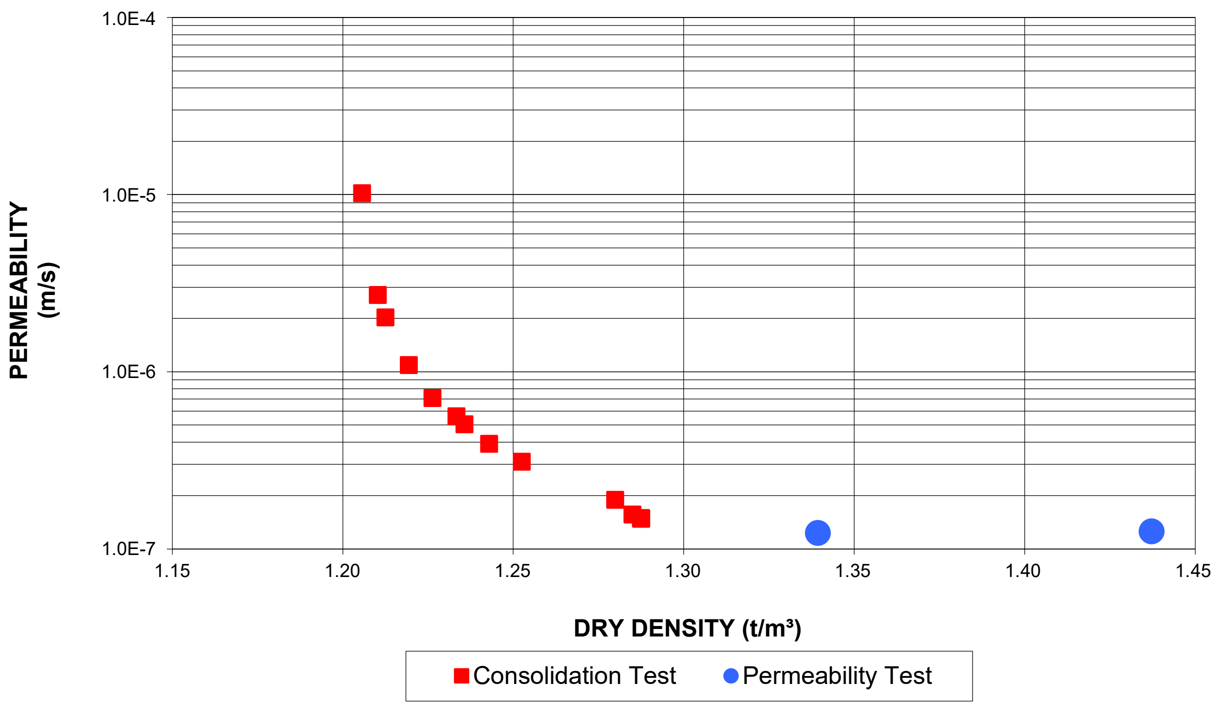


RUSTLERS ROOST GOLD PROJECT
AIR DRYING TESTS - EVAPORATION
RUSTLERS ROOST COMPOSITE SAMPLE

PE801-00102 M21006
Figure 3.5



SETTLEMENT (mm)
0
5
10
15
20
25
30
35
40
45
50
1.
10.
100.
1000.
10000.
TIME (minutes)
One day
Consolidation Data
Initial Height: 539 mm
Final Height: 501 mm
Inital Void Ratio: 1.4
Final Void Ratio: 1.2
Stress Range: 2.1 kPa - 5.5 kPa



PERMEABILITY (m/s)
1.0E-4
1.0E-5
1.0E-6
1.0E-7
1.15
1.20
1.25
1.30
1.35
1.40
1.45
DRY DENSITY (t/m³)
Consolidation Test
Permeability Test

APPENDIX A
Laboratory Test Results

Trilab
Soil Rock Calibration

Brisbane
346A Bilsen Road,
Geebung
QLD 4034
Ph: +61 7 3265 5656

Perth
2 Kimmer Place,
Queens Park
WA 6107
Ph: +61 8 9258 8323

ATTERBERG LIMITS TEST REPORT

Test Method: AS 1289 2.1.1, 3.9.2, 3.2.1, 3.3.1, 3.4.1

Client	Knight Piesold Pty Ltd	**Report No.**	P21080027-AL
		Workorder No.	0020462
Address	Level 1 184 Adelaide Terrace, EAST PERTH WA 6004	**Report Date**	13/08/2021
Project	Rustlers Roost		

Sample No.	21080027					
Liquid Limit Determination	AS 1289.3.9.1					
Test Date	06/08/2021					
Client ID	Hanking PR Tailings Composite					
Depth (m)	Not Supplied					
Liquid Limit (%)	28					
Plastic Limit (%)	19					
Plasticity Index (%)	9					
Linear Shrinkage (%)	3.5 *					
Moisture Content (%)	0.3					

Sample No.						
Liquid Limit Determination						
Test Date						
Client ID						
Depth (m)						
Liquid Limit (%)						
Plastic Limit (%)						
Plasticity Index (%)						
Linear Shrinkage (%)						
Moisture Content (%)						

NOTES/REMARKS: The samples were tested oven dried, dry sieved and in a 125-250mm mould.

Sample/s supplied by the client * Cracking occurred + Curling occurred

Accredited for compliance with ISO/IEC 17025 - Testing.
The results of the tests, calibrations, and/or measurements included in this document are traceable to Australian/National Standards.

Tested at Trilab Perth Laboratory

Authorised Signatory

G. Creely

NATA ACCREDITED FOR TECHNICAL COMPETENCE

Laboratory No. 9926

Trilab
Soil Rock Calibration

Brisbane
346A Bilsen Road,
Geebung
QLD 4034
Ph: +61 7 3265 5656

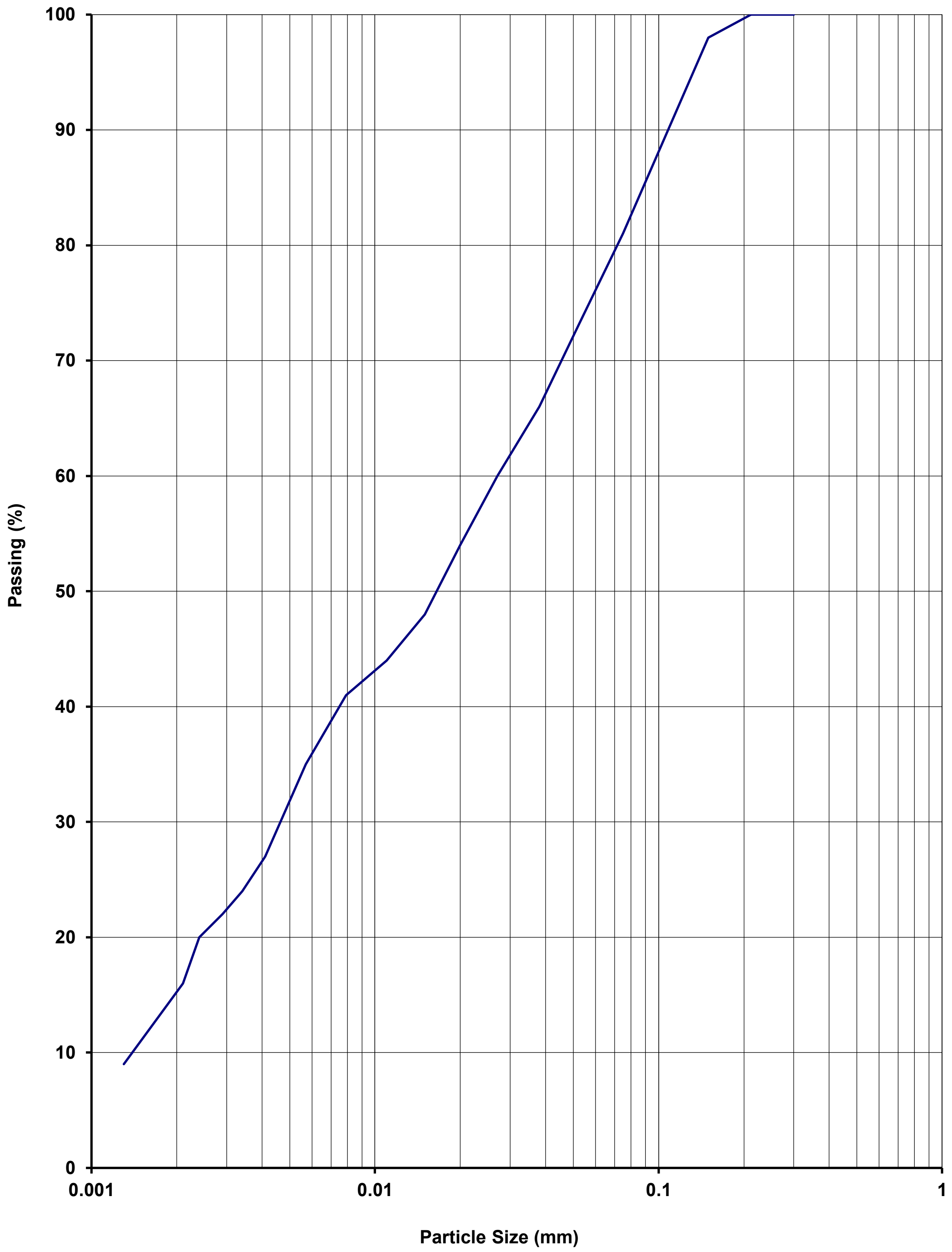
Perth
2 Kimmer Place,
Queens Park
WA 6107
Ph: +61 8 9258 8323

PARTICLE SIZE DISTRIBUTION TEST REPORT

Test Method: AS 1289 3.6.3, 3.5.1 & 2.1.1

Client	Knight Piesold Pty Ltd	**Report No.**	P21080027-G
		Workorder No.	0020462
Address	Level 1 184 Adelaide Terrace, EAST PERTH WA 6004	**Test Date**	5/8/2021
		Report Date	13/8/2021
Project	Rustlers Roost		
Client ID	Hanking PR Tailings Composite	**Depth (m)**	Not Supplied

Sieve Size (mm)	Passing %
150.0	
75.0	
63.0	
53.0	
37.5	
26.5	
19.0	
13.2	
9.5	
6.7	
4.75	
2.36	
1.18	
0.600	
0.425	
0.300	
0.212	100
0.150	98
0.075	81
0.052	73
0.038	66
0.027	60
0.02	54
0.015	48
0.011	44
0.0079	41
0.0057	35
0.0041	27
0.0034	24
0.0029	22
0.0024	20
0.0021	16
0.0013	9



NOTES/REMARKS: -

Moisture Content 0.3%   -2.36mm Soil Particle Density(t/m^3) 2.86

Sample/s supplied by the client

Accredited for compliance with ISO/IEC 17025 - Testing.
The results of the tests, calibrations, and/or measurements included in this document are traceable to Australian/National Standards.

Tested at Trilab Perth Laboratory

Authorised Signatory

G. Creely

NATA
ACCREDITED FOR TECHNICAL COMPETENCE

Laboratory No. 9926

APPENDIX D

Memorandum Ref. PE21-21010 Rustlers Roost Gold Project –Water Balance Modelling

MEMORANDUM

To:	Hanking Australia Investment	Date:	20 December 2021
Attn:	John Zimmerman	Our Ref:	PE21-01614
		KP File Ref.:	PE801-00102/06-A djtm M21010
cc:	Charles Hastie	From:	Dave Morgan

RE: RUSTLERS ROOST GOLD PROJECT – WATER BALANCE MODELLING

EXECUTIVE SUMMARY

Site water balance modelling was completed for the Rustlers Roost Gold Project Plant Site and Tailings Storage Facility (TSF), for average and design wet and dry climatic conditions as part of the Feasibility Study design. The modelling was completed for a tailings percent solids of 55%.

Key findings from the modelling are as follows:

- The TSF is designed to hold the tailings plus the design rainfall conditions, and thus has sufficient storm water storage capacity for all design storm events and rainfall sequences.
- The supernatant pond should be removed (and treated if necessary) as soon as practicable after decommissioning.
- Process water shortfall is expected to occur under average and design dry climatic conditions. Peak shortfalls occur in 2025, primarily due to lower runoff in the initial stages of operation and an increasing tailings beach, resulting in lower recovery of supernatant from the TSF.
- Make up water up to 1.86 Mm^3/year will be required to cover shortfalls from the TSF with an additional 0.57 Mm^3/year to cover the additional site water requirements. This water will be sourced from either pit dewatering and/or bore field supply.

1. INTRODUCTION

Please find herein the site-wide water balance modelling for the Rustlers Roost Gold Project Plant Site and Tailings Storage Facility (TSF). The water balance modelling was completed as part of the Feasibility Design.

2. MODELLING PARAMETERS

2.1 OBJECTIVES

The primary objectives of the tailings deposition and water balance modelling are summarised below:

- Establish the filling rate for tailings solids within the TSF.
- Estimate the in-situ tailings density within the TSF, taking into consideration tailings properties from laboratory testing and the TSF basin storage parameters.

- Determine supernatant pond volumes within the TSF under average climatic conditions throughout operation.
- Determine supernatant pond volumes within the TSF for design wet rainfall sequences and storm events, check TSF storm water storage capacity and confirm the suitability of the current TSF design philosophy.
- Determine staged embankment crest elevations, to ensure containment of tailings and design supernatant pond volumes.
- Determine the likelihood of recycle water shortfalls during average conditions and design dry rainfall sequences.
- Determine the required make-up water supply rate to provide additional make-up water for shortfalls.
- Assess risk factors for water balance modelling.

2.2 MODEL PARAMETERS AND ASSUMPTIONS

2.2.1 General

The water balance was completed using a site-wide water balance model, utilising a monthly time step. Tailings behaviour parameters were incorporated into the model using KP's in-house tailings density modelling software. The model allows for the build-up of tailings solids and area, then calculates the tailings densities, supernatant release, and water losses from within the TSF due to evaporation, seepage etc.

2.2.2 Tailings Storage Facility (TSF)

For the water management model, the following inputs into the TSF were included:

- Water in the slurry from the process plant.
- Rainfall runoff from the tailings and pond surface and surrounding catchments.

Outputs from the TSF include:

- Evaporation losses.
- Seepage losses.
- Water returned to the plant as process make-up water (via decant system).

2.3 WATER USAGE PRIORITY

For the water management modelling the following water requirements and priorities were assumed to supply the required process plant water demand:

- The water present in the ore was allowed for.
- Water collected from Plant Site runoff was not allowed for.
- The minimum raw water requirement (for use in the process plant) is sourced from the bore field and pit dewatering.
- The additional raw water requirement for dust suppression and wash down (external to the process) is sourced from the bore field and pit dewatering.
- After allowing for the bulleted items above, the model tries to supply the remaining process water requirement from the TSF supernatant pond.
- After including the process water return from the TSF, any further water requirements (make-up water) are provided from the external make-up supply i.e., pit dewatering and/or bore field supply.
- If the process water requirement is not met, the model then determines any shortfall volume.

2.4 MODELLING SCENARIOS

For water management modelling, various design rainfall conditions were modelled for selected operational years. The following rainfall sequences were modelled:

- Average conditions.
- 1 in 100 year recurrence interval, 1 year dry rainfall sequence.
- 1 in 100 year recurrence interval, 1 year wet rainfall sequence.
- 1 in 100 year recurrence interval, 72 hour duration storm event superimposed over 1 in 10 year recurrence interval wet season (90 days duration), with 100% runoff and no evaporation.

When design rainfall sequences were modelled in a selected year, it was assumed that average climatic conditions occurred prior to, and subsequent to, the design sequence.

Average and design wet / dry monthly rainfall and evaporation values were used in the modelling, as documented in the Baseline Climatology Assessment completed by KP (Memorandum Ref. PE21-01333 (October 2021). Monthly rainfall and evaporation values are provided in Table 2.1.

Table 2.1: Design rainfall and evaporation annual sequences

Month	Average Rainfall (mm)	1 in 100 ARI Wet Annual Rainfall (mm)	1 in 100 ARI Dry Annual Rainfall (mm)	Average Pan Evap. (mm)	Average Lake Evap. (mm)
January	241	427	222	170	123
February	410	641	266	150	109
March	252	438	131	170	123
April	56	380	65	177	127
May	0	0	23	190	137
June	3	0	0	181	130
July	0	2	0	193	139
August	0	0	0	212	151
September	26	33	3	222	157
October	62	161	0	236	165
November	93	94	232	204	145
December	338	343	66	186	134
TOTAL	1,481	2,519	1,010	2,290	1,641

Design storm events from the baseline climatology assessment are provided in Table 2.2. The design storm event of a 1 in 100-year recurrence interval, 72 hour duration storm event has a rainfall depth of 441.1 mm.

Table 2.2: Storm Events (24 hour and 72 hour duration)

Duration (hours)	100-year ARI	1,000-year ARI	PMP
24	293.2	457.0	1200
72	441.1	647.6	2,180

The design wet season was modelled as a 90-day duration rainfall sequence (the period in which 70% of the annual rainfall occurs on average, as defined by ANCOLD (Ref. 1)). This sequence is summarised in Table 2.3.

Table 2.3: Design wet season rainfall sequence

Month	Wet Season Scenarios (mm)				
	5-yr ARI	10-yr ARI	20-yr ARI	50-yr ARI	100-yr ARI
Jan	308	336	355	372	380
Feb	504	550	581	609	623
Mar	466	508	537	562	576
TOTAL	1,278	1,394	1,473	1,543	1,579

2.5 CATCHMENT AND RUNOFF ESTIMATION

Catchment and runoff characteristics used in the water balance modelling are shown in Table 2.4. Runoff coefficients shown in Table 2.4 were assumed based on similar regional project experience.

Table 2.4: Catchment characteristics

Parameter	Value
TSF Catchment Area	191 Ha
TSF Storage Curve	Refer Figure 2.1
Runoff Coefficients (Long-term) - Undisturbed (external to basins) - Compacted Soil Liner - Tailings beach (Drying) - Tailings beach (Active) - Ponds	 20% 40% 85% 100% 100%
Runoff Coefficients (Short-term event*) - Undisturbed (external to basins) - Cleared areas - Compacted Soil Liner - Tailings beach	 100% 100% 100% 100%

*1 in 100 year average recurrence interval, 72 hour duration storm event

2.6 TAILINGS CHARACTERISTICS

Modelling parameters for the tailings were adopted based on tailings physical testing completed by KP on two bags of dry tailings in July 2021 (Hanking RR Tailings Composite). Tailings testing results and interpretation are presented in Memorandum Ref. PE20-01554 (8 December 2021). Tailings parameters determined from the July 2021 tailings testing have been adjusted from 60% solids to 55% percent solids as advised by Hanking via email on 26 October 2021.

Ore blends throughout operation were assumed to be uniform throughout operation and have been adopted in the model based on the behaviour observed during tailings testing. Tailings parameters used in the water balance modelling are summarised in Table 2.5.

Table 2.5: Tailings characteristics

Parameter	Value
Tailings % Solids	55%
In Situ Density (minimum)*	0.99
In Situ Density (maximum)*	1.50
Water Released - Supernatant - Underdrainage	 37% 10%

* Outcomes of tailings density/consolidation modelling

2.7 PIT DEWATERING AND BORE FIELD SUPPLY

Pit dewatering and external bore field supply were not considered as direct inputs into the model. The model determines any process supply shortfalls from the TSF and additional raw water demand to estimate the required make up water supply from external sources (Pit dewatering and Bore Field supply).

2.8 PROCESS PARAMETERS

Process parameters used in the water balance modelling and additional water requirements (both provided by Hanking) are summarised in tables 2.6 and 2.7 respectively.

Table 2.6: Process parameters

Parameter	Average Flow (% of water in slurry)
Water in Slurry (Plant Tailings)	100%
Water in Ore [1]	4.0%
Minimum Raw Water Required (e.g. reagents, gland)	17.2%
Maximum Allowable TSF Recycle [2]	82.8%

[1] Provided by Hanking Australia Investment.
[2] Obtained from CDM Smith Investigation and Modelling Report (Ref. 2)

Table 2.7: Additional Water Requirements

Parameter	Average Flow (m^3/h)
Mine Services and Mine Dust Suppression	21.9 [1,2]
Potable Water	49.3 [1,2,3]

[1] Based on 8,000 hours of operation per year.
[2] Obtained from CDM Smith Investigation and Modelling Report (Ref. 2)
[3] No seasonal variation or day/night factors have been included.

2.9 ADDITIONAL PARAMETERS

Additional modelling parameters used are listed in Table 2.8.

Table 2.8: Additional modelling parameters

Parameter	Value
TSF	
Commissioning date (storage)	01 September 2023
Commissioning date (operation)	01 October 2023
Tailings beach slope [1]	150 H:1V
Minimum operating pond volume	50,000 m^3
Tailings permeability [1]	1 x 10^{-7} m/s
Plant Site Parameters	
Commissioning Date	01 October 2023
Tailings Production Data	
Tailings % Solids	55%
Throughput (dry tonnes)	2.5 – 4.5 Mtpa
Total tonnage	48 Mt
General Parameters	
Evaporation pan factor (pond) [1]	0.70
Evaporation pan factor (beach) [1]	0.90

[1] Indicates assumed parameters

3. WATER BALANCE MODELLING RESULTS

3.1 AVERAGE CONDITIONS

Water management modelling using average climatic conditions allows general trends in the behaviour of the water management system to be determined.

Modelling results for the TSF under average climatic conditions are plotted on Figure 3.1. Plant Site water usage for average conditions is plotted on Figure 3.2.

Observations from the average conditions modelling are summarised below:

Tailings Storage Facility - General

- Based on the storage commissioning date (September 2023) adopted for the modelling, the TSF will accumulate rainfall runoff prior to process commissioning (October 2023).
- TSF recycle rates for each year of operation are provided in Table 3.1 for each year. The recycle rates are expressed as a percentage of the water in slurry reporting to the TSF.

Table 3.1: Average conditions – TSF recycle rates

Year	Total Annual Water in Slurry Volume (m^3/year)	Total Recycle Volume (m^3/year)	Average Monthly Recycle Rate (%)	Maximum Monthly Recycle Rate (%)	Minimum Monthly Recycle Rate (%)
2023*	711,000	311,000	44%	79%	0%
2024	2,640,000	1,514,000	57%	79%	19%
2025	3,654,000	2,124,000	58%	79%	22%
2026	3,654,000	2,346,000	64%	79%	22%
2027	3,658,000	2,450,000	67%	79%	32%
2028	3,661,000	2,497,000	68%	79%	33%
2029	3,654,000	2,530,000	69%	79%	33%
2030	3,654,000	2,563,000	70%	79%	33%
2031	3,658,000	2,595,000	71%	79%	33%
2032	3,661,000	2,622,000	72%	79%	34%
2033	3,655,000	2,642,000	72%	79%	40%
2034*	3,017,000	2,209,000	73%	79%	46%

* 2023 operation only September to December; 2034 operation comprises January to October only.

Tailings Storage Facility

- The supernatant pond volume peaks in March each year (at the end of the wet season), before returning to the minimum operating pond volume during the subsequent dry season.
- Prior to commissioning, the stored volume in the TSF in September 2023 is negligible.
- During operation, the peak volume increases from 142,000 m^3 in December 2023 to 1,140,000 m^3 in March 2034.
- The facility ceases operating in October 2034, with 50,000 m^3 in the supernatant pond, and the water balance remains positive after decommissioning (increasing pond volume). The supernatant pond volume in the final 12 months of operation ranges from 50,000 m^3 (in November 2034) to 1,140,000 m^3 (in March 2034), and accumulates water thereafter. Therefore, the supernatant pond should be removed (and treated if necessary) as soon as practicable after decommissioning and the TSF should be rehabilitated as a water shedding structure.

Plant Site and Water Supply

- Process water shortfall is expected to occur under average climatic conditions, summarised in Table 3.2. The peak annual shortfall of 1,530,000 m^3/year occurs in 2025, primarily due to lower effective runoff from the TSF catchment (due to the smaller tailings beach area resulting in lower recovery of supernatant from the TSF) and an increasing throughput to the TSF.
- The peak monthly process water shortfall of 236,000 m^3/month occurs in August 2025.

Table 3.2: Average conditions – Process Water Shortfall (prior to make up supply)

Year	Process Water Shortfall Volume (m^3/year)	Process Water Shortfall (% of water in slurry)	Total Raw Water Required (m^3/year)
2023*	400,000	56%	970,000
2024	1,126,000	43%	1,696,000
2025	1,530,000	42%	2,100,000
2026	1,308,000	36%	1,878,000
2027	1,208,000	33%	1,778,000
2028	1,164,000	32%	1,734,000
2029	1,124,000	31%	1,694,000
2030	1,091,000	30%	1,661,000
2031	1,063,000	29%	1,633,000
2032	1,039,000	28%	1,609,000
2033	1,013,000	28%	1,583,000
2034*	808,000	27%	1,378,000

* 2023 operation only September to December; 2034 operation comprises January to October only.

3.2 WET CONDITIONS

Modelling of design wet rainfall sequences and storm events were completed for each year of operation. The design cases considered were:

- 1 in 100 year recurrence interval, 12 month wet rainfall sequence.
- 1 in 100 year recurrence interval, 72 hour storm event (magnitude 440 mm) superimposed on the 1 in 10 year recurrence interval, wet season runoff (90 days duration), with 100% runoff and no evaporation.

For both situations, average climatic conditions were modelled prior to and following the design events. In addition, the supernatant pond volume prior to design events is based on the assumption that the TSF is managed appropriately and the accumulation of excess water reduced by recycling water to the plant to the maximum extent practicable, as assumed in the overall design concept.

Wet conditions modelling results for the TSF are presented in Figure 3.3.

TSF recycle rates for a 1 in 100 wet year for each year of operation are provided in Table 3.3. The maximum and minimum rates show the range of monthly values throughout each year. The recycle rates are expressed as a percentage of water in slurry reporting to the TSF. Average and design wet conditions supernatant pond volumes are summarised in Table 3.4. It is noted that the values shown are the highest recycle rates for design wet conditions, generally occurring in the same year as the 1 in 100 wet year.

Table 3.3: Wet conditions – TSF recycle rates

Year	Total Annual Water in Slurry Volume (m^3/year)	Total Recycle Volume (m^3/year)	Average Monthly Recycle Rate (%)	Maximum Monthly Recycle Rate (%)	Minimum Monthly Recycle Rate (%)
2023*	711,000	424,000	60%	79%	2%
2024	2,640,000	2,080,000	79%	79%	79%
2025 to 2033	3,656,000	2,879,000	79%	79%	79%
2034*	3,017,000	2,377,000	79%	79%	79%

* 2023 operation only September to December; 2034 operation comprises January to October only.

Table 3.4: Wet conditions – Peak supernatant pond volumes

Year (in which event occurs)	Peak Supernatant Pond Volume (m^3)		
	Average Conditions	1 in 100 Year, 12 month wet sequence	1 in 100 Year, 72 hour storm event + 1 in 10 Year, Wet Season Runoff*2
2023*1	142,000	401,000	**650,000**
2024	394,000	2,164,000	**3,205,000**
2025	466,000	2,037,000	**3,125,000**
2026	729,000	2,225,000	**3,171,000**
2027	862,000	2,334,000	**3,211,000**
2028	921,000	2,065,000	**3,231,000**
2029	967,000	2,130,000	**3,250,000**
2030	1,007,000	2,182,000	**3,268,000**
2031	1,045,000	2,196,000	**3,285,000**
2032	1,075,000	2,207,000	**3,300,000**
2033	1,108,000	2,078,000	**3,315,000**
2034*1	1,140,000	**2,069,000**	1,706,000

*1 2023 operation only September to December; 2034 operation comprises January to October only.
*2 No evaporation and 100% runoff.

Key observations from the wet conditions modelling are as follows:

Tailings Storage Facility

- The TSF is designed to hold the tailings plus the design rainfall conditions, and thus has sufficient storm water storage capacity for all design storm events and rainfall sequences.
- The design storm event results in the largest supernatant pond volume for all years of operation.
- The TSF has a minimum stormwater storage capacity of 3.3 Mm^3 (Stage 1) to 3.0 Mm^3 (final). This occurs in December each year. It should be noted that the peak storm events occur in March, nine months prior to this time, when greater stormwater storage capacity is available (i.e., 5.0 Mm^3 stormwater storage capacity in March 2024).
- The critical design rainfall event in terms of pond elevation is the design storm event occurring in the last month of each stage of operation, when the TSF storm water capacity is at its minimum. There is sufficient stormwater storage capacity in the TSF for this situation.

3.3 DRY CONDITIONS

Modelling of design dry rainfall sequences was completed for each year of operation. The design case considered was a 1 in 100 year recurrence interval, 12 month dry rainfall sequence. Average climatic conditions were modelled prior to and following the design event.

TSF recycle rates for each year of operation under design dry conditions are provided in Table 3.5. The maximum and minimum rates show the range of monthly values throughout each year. The recycle rates are expressed as a percentage of the water in slurry reporting to the TSF. Plant Site water usages for design dry conditions are plotted on Figure 3.4. The values shown are the lowest recycle rates for design dry conditions, which occur during the year in which the 1 in 100 dry year sequence occurs.

Table 3.5: Dry conditions – TSF recycle rates

Year	Total Annual Water in Slurry Volume (m^3/year)	Total Recycle Volume (m^3/year)	Average Monthly Recycle Rate (%)	Maximum Monthly Recycle Rate (%)	Minimum Monthly Recycle Rate (%)
2023	711,000	280,000	39%	79%	27%
2024	2,640,000	1,248,000	47%	79%	19%
2025	3,654,000	1,794,000	49%	79%	21%
2026	3,654,000	1,943,000	53%	79%	21%
2027	3,658,000	2,014,000	55%	79%	21%
2028	3,661,000	2,049,000	56%	79%	21%
2029	3,654,000	2,071,000	57%	79%	21%
2030	3,654,000	2,093,000	57%	79%	21%
2031	3,658,000	2,116,000	58%	79%	21%
2032	3,661,000	2,133,000	58%	79%	21%
2033	3,655,000	2,149,000	59%	79%	21%
2034	3,017,000	1,679,000	56%	79%	20%

* 2023 operation only September to December; 2034 operation comprises January to October only.

Key observations from dry conditions modelling are as follows:

Plant Site and Water Supply

- Process water shortfall is expected to occur under dry climatic conditions, summarised in Table 3.6. Peak shortfalls occur in 2025, primarily due to lower runoff in the initial stages of operation and an increasing tailings beach, resulting in lower recovery of supernatant from the TSF.
- The peak monthly shortfall under design dry conditions is 243,400 m^3/month in October 2031.

Table 3.6: Dry conditions – Process water shortfall (prior to make up supply)

Year	Average Conditions			Design Dry Conditions		
	Process Water Shortfall Volume (m^3/year)	Process Water Shortfall (% of water in slurry)	Total Raw Water Required (m^3/year)	Process Water Shortfall Volume (m^3/year)	Process Water Shortfall (% of water in slurry)	Total Raw Water Required (m^3/year)
2023	400,000	56%	590,000	431,000	61%	621,000
2024	1,126,000	43%	1,696,000	1,392,000	53%	1,962,000
2025	1,530,000	42%	2,100,000	1,860,000	51%	2,430,000
2026	1,308,000	36%	1,878,000	1,711,000	47%	2,281,000
2027	1,208,000	33%	1,778,000	1,644,000	45%	2,214,000
2028	1,164,000	32%	1,734,000	1,612,000	44%	2,182,000
2029	1,124,000	31%	1,694,000	1,583,000	43%	2,153,000
2030	1,091,000	30%	1,661,000	1,561,000	43%	2,131,000
2031	1,063,000	29%	1,633,000	1,542,000	42%	2,112,000
2032	1,039,000	28%	1,609,000	1,528,000	42%	2,098,000
2033	1,013,000	28%	1,583,000	1,506,000	41%	2,076,000
2034*	808,000	27%	1,283,000	1,338,000	44%	1,813,000

* 2023 operation only September to December; 2034 operation comprises January to October only.

4. CONCLUSIONS

Water balance modelling results are summarised in Table 4.1.

Table 4.1: Water balance modelling summary

	Design Conditions			
	Average Rainfall Conditions	100 Year ARI Dry Rainfall Conditions	100 Year Wet Rainfall Conditions	Average Conditions + 100 year ARI 72-hour storm + 1 in 10 year wet season
Tailings Storage Facility				
Peak Supernatant Pond Volume (m^3) (2023) *1	142,000	78,000	401,000	650,000
Peak Supernatant Pond Volume (m^3) (2029)	967,000	410,000	2,130,000	3,250,000
Peak Supernatant Pond Volume (m^3) (Final Year) *1	1,140,000	488,000	2,069,000	1,706,000
Average TSF Recycle Rate – 2023 (% of water in slurry) *2	44%	39%	60%	-
Minimum Monthly TSF Recycle Rate – 2023 (% of water in slurry) *2	0%	27%	2%	-
Maximum Monthly TSF Recycle Rate – 2023 (% of water in slurry) *2	79%	79%	79%	-
Average TSF Recycle Rate – 2029 (% of water in slurry) *2	69%	57%	79%	-
Minimum Monthly TSF Recycle Rate – 2029 (% of water in slurry) *2	33%	21%	79%	-
Maximum Monthly TSF Recycle Rate – 2029 (% of water in slurry) *2	79%	79%	79%	-
Average TSF Recycle Rate – Final Year (% of water in slurry) *2	73%	56%	79%	-
Minimum Monthly TSF Recycle Rate – Final Year*1 (% of water in slurry) *2	46%	20%	79%	-
Maximum Monthly TSF Recycle Rate – Final Year*1 (% of water in slurry) *2	79%	79%	79%	-
Plant Site and Water Supply				
Maximum Annual Process Water Shortfall (m^3/year) *1	1,530,000	1,860,000	-	-
Maximum Annual Process Water Shortfall (% of water in slurry) *2	56%	51%	-	-
Maximum Annual Raw Water Demand (m^3/year)	2,100,000	2,430,000	-	-

*1 2023 operation only September to December; 2034 operation comprises January to October only.
*2 Percentage of total annual volume of water required.

Based on the modelling undertaken, the following conclusions can be drawn:

Tailings Storage Facility

- The TSF is designed to hold the tailings plus the design rainfall conditions, and thus has sufficient storm water storage capacity for all design storm events and rainfall sequences.
- The supernatant pond volume peaks in March each year (at the end of the wet season), before returning to the minimum operating pond volume during the subsequent dry season.
- Prior to commissioning, the stored volume in the TSF in September 2023 is negligible.
- During operation, assuming average conditions, the peak volume increases from 142,000 m^3 in December 2023 to 1,140,000 m^3 in March 2034.
- The facility ceases operating in October 2034, with 50,000 m^3 within the supernatant pond, and the water balance remains positive after decommissioning (increasing pond volume).
- The supernatant pond volume in the final 12 months of operation ranges from 1,140,000 m^3 (in March 2034) to 50,000 m^3 (in November 2034), and accumulates water thereafter. Therefore, the supernatant pond should be removed (and treated if necessary) as soon as practicable after decommissioning and the TSF should be rehabilitated as a water shedding structure.

Plant Site and Water Supply

- Process water shortfall is expected to occur under average and design dry climatic conditions. Peak shortfalls occur in 2025, primarily due to lower runoff in the initial stages of operation and an increasing tailings beach, resulting in lower recovery of supernatant from the TSF.
- Make up water up to 1.86 Mm^3/year will be required to cover shortfalls from the TSF with an additional 0.57 Mm^3/year to cover the additional site water requirements as noted in Table 2.7.

We trust this is sufficient information for your current requirements. If you have any questions please contact us.

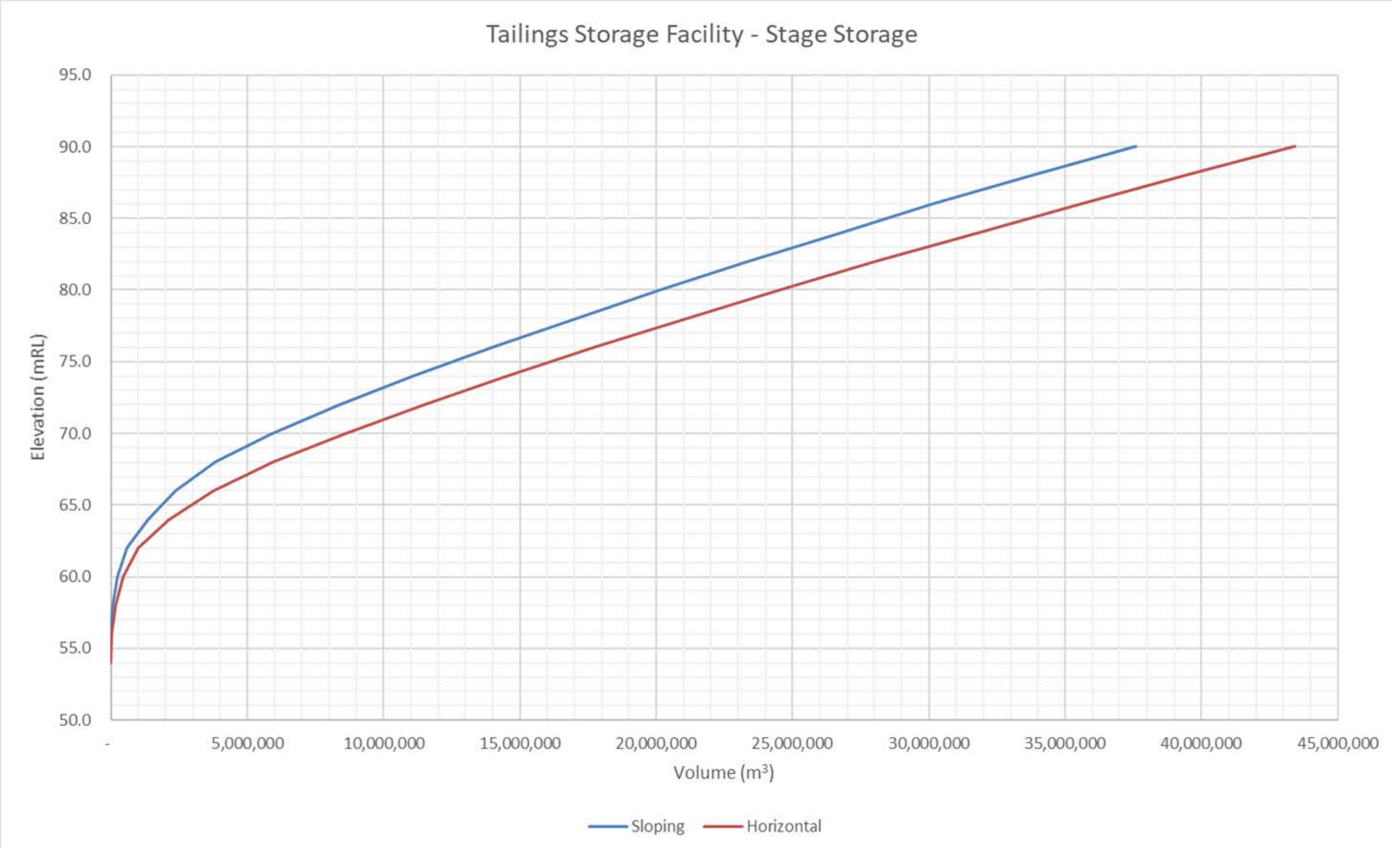
Yours faithfully
KNIGHT PIÉSOLD PTY LTD

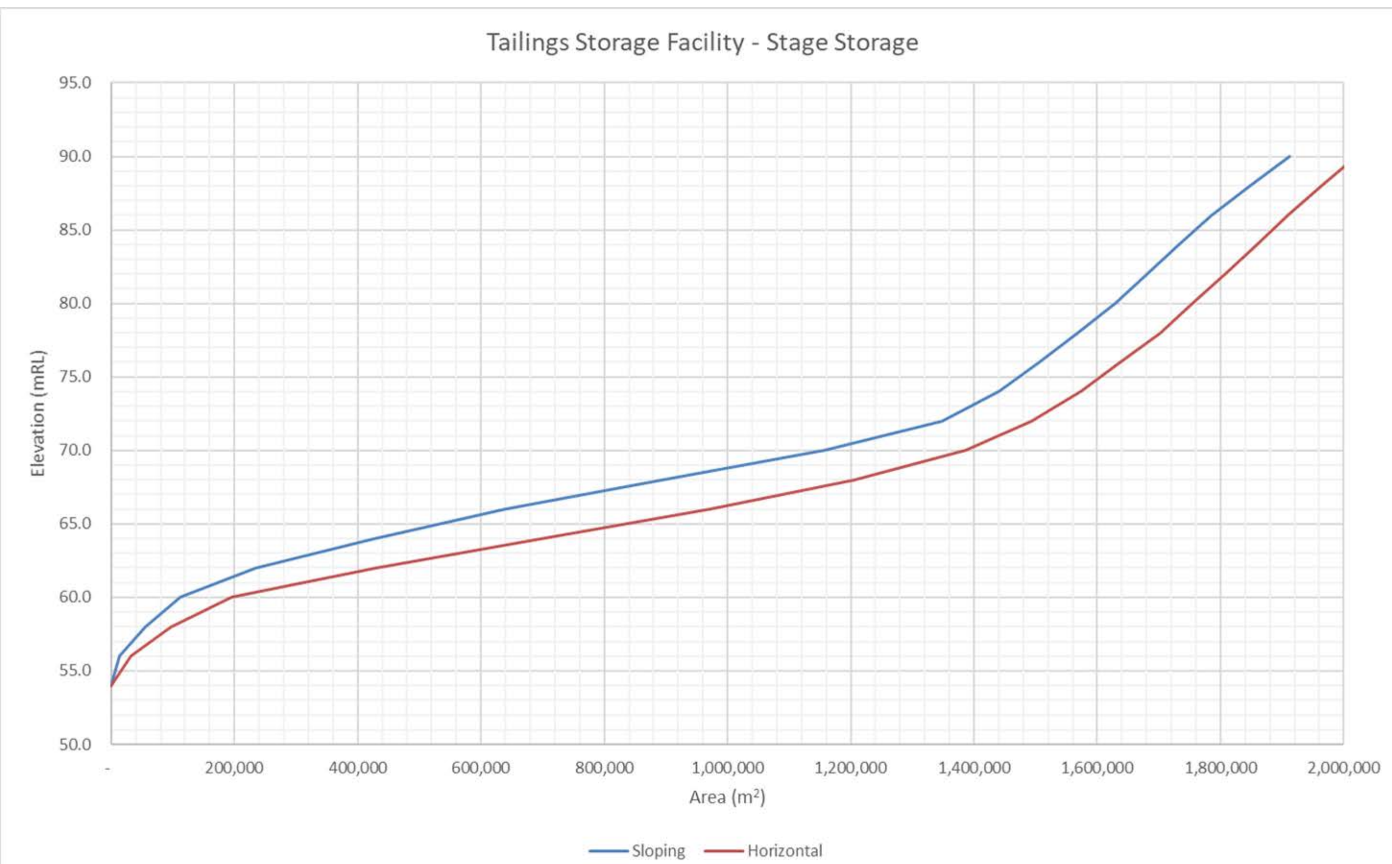
ANDREW MOLLAN	**DAVID MORGAN**
Project Engineer	Managing Director

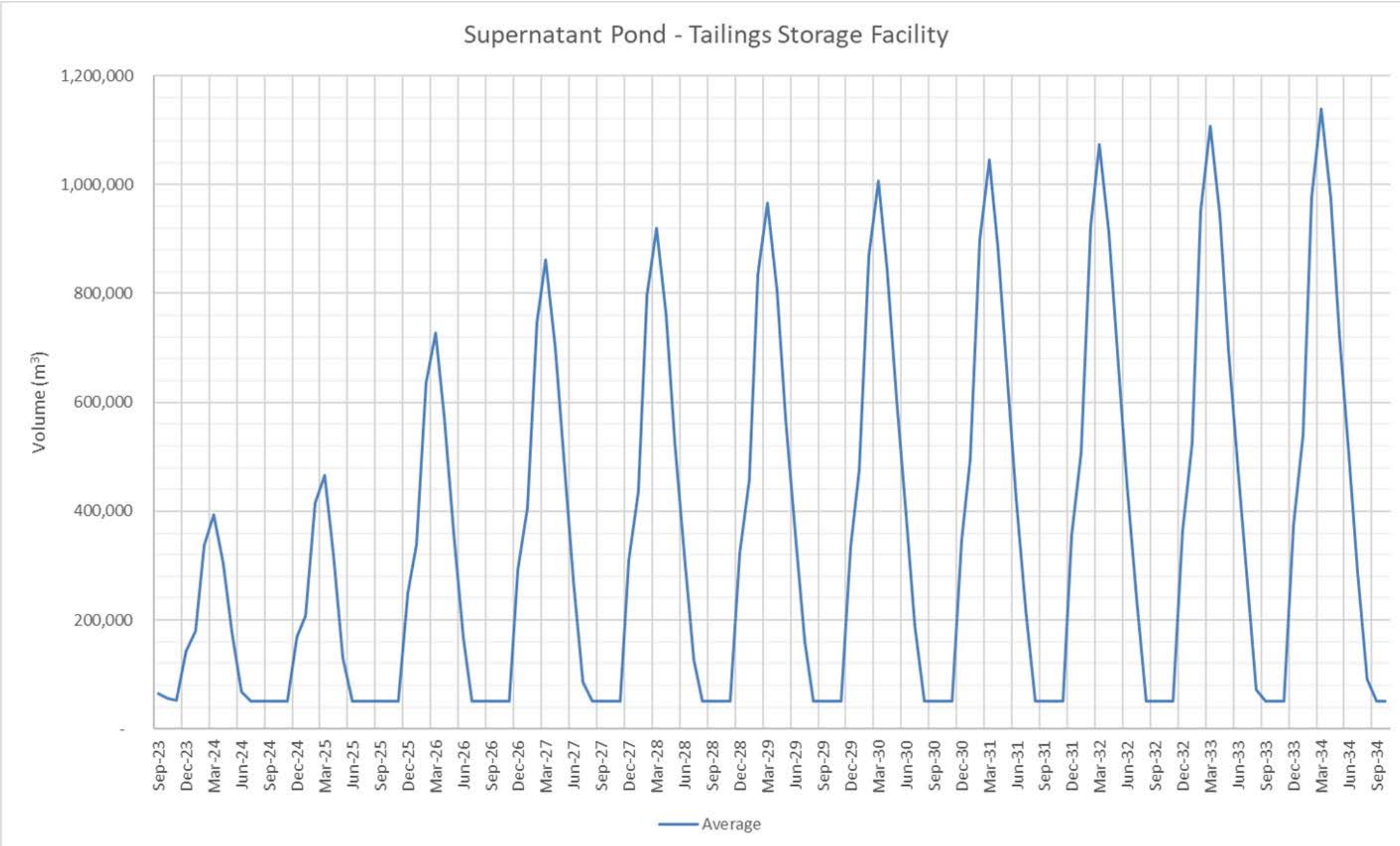
REFERENCES

1. ANCOLD "*Guidelines on Tailings Dams, Planning, Design, Construction, Operation and Closure*", July 2019.
2. CDM Smith "*Appendix H - Groundwater Investigation and Modelling Report_Rev.0*", September 2021.

FIGURES

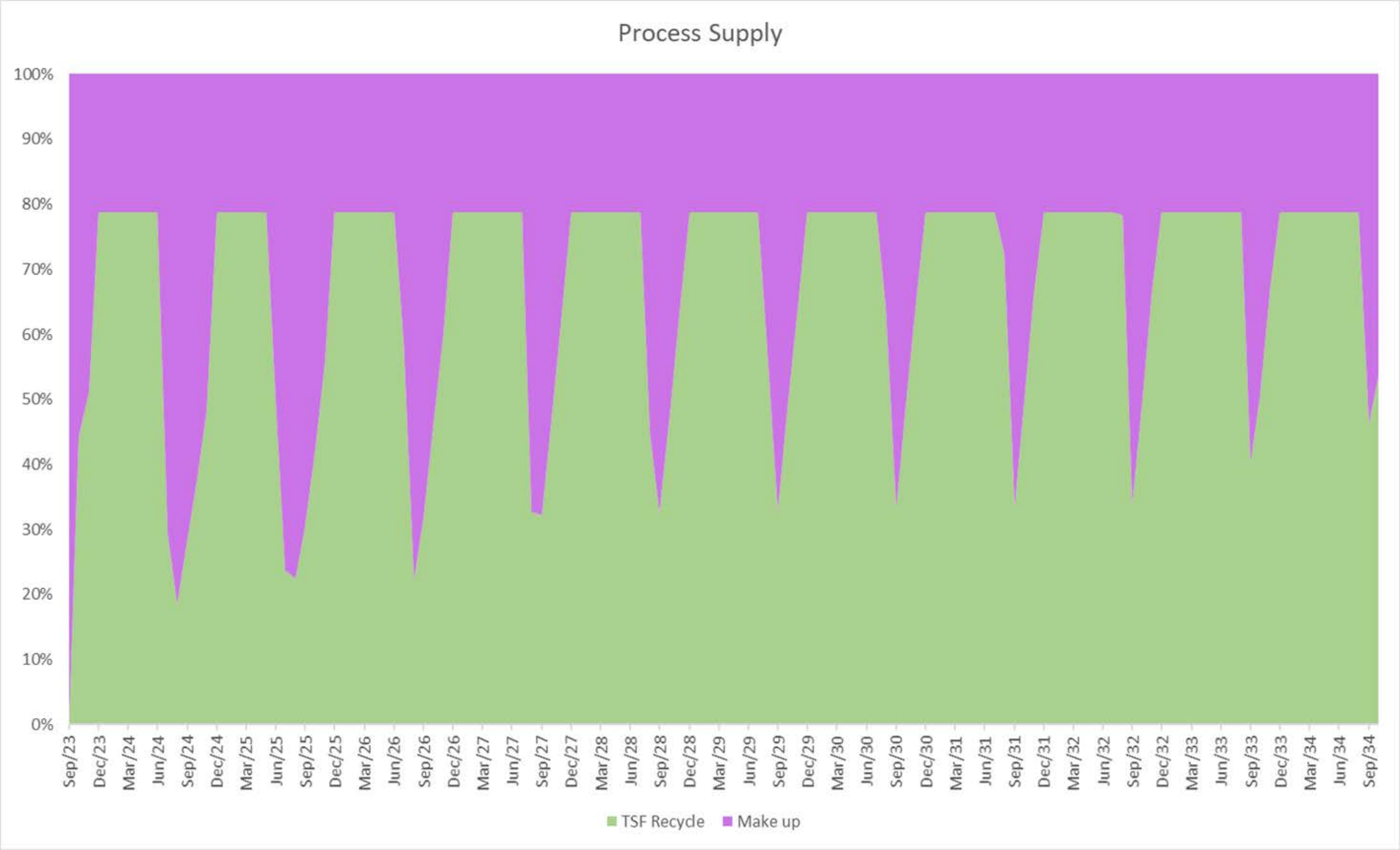

Tailings Storage Facility - Stage Storage
Elevation (mRL)
Volume (m³)
95.0
90.0
85.0
80.0
75.0
70.0
65.0
60.0
55.0
50.0
-
5,000,000
10,000,000
15,000,000
20,000,000
25,000,000
30,000,000
35,000,000
40,000,000
45,000,000
Sloping
Horizontal

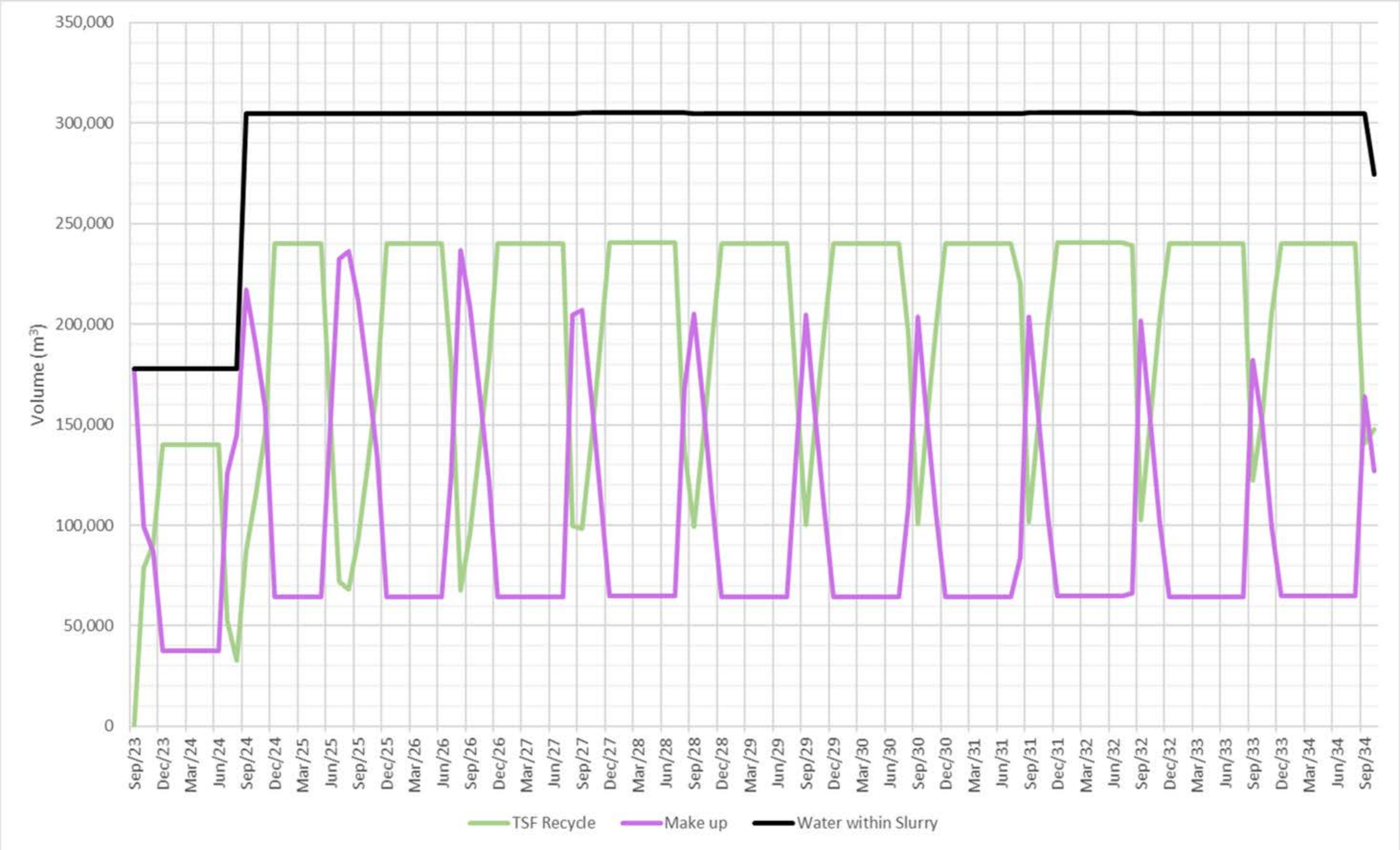

Tailings Storage Facility - Stage Storage
Elevation (mRL)
Area (m²)
95.0
90.0
85.0
80.0
75.0
70.0
65.0
60.0
55.0
50.0
-
200,000
400,000
600,000
800,000
1,000,000
1,200,000
1,400,000
1,600,000
1,800,000
2,000,000
Sloping
Horizontal

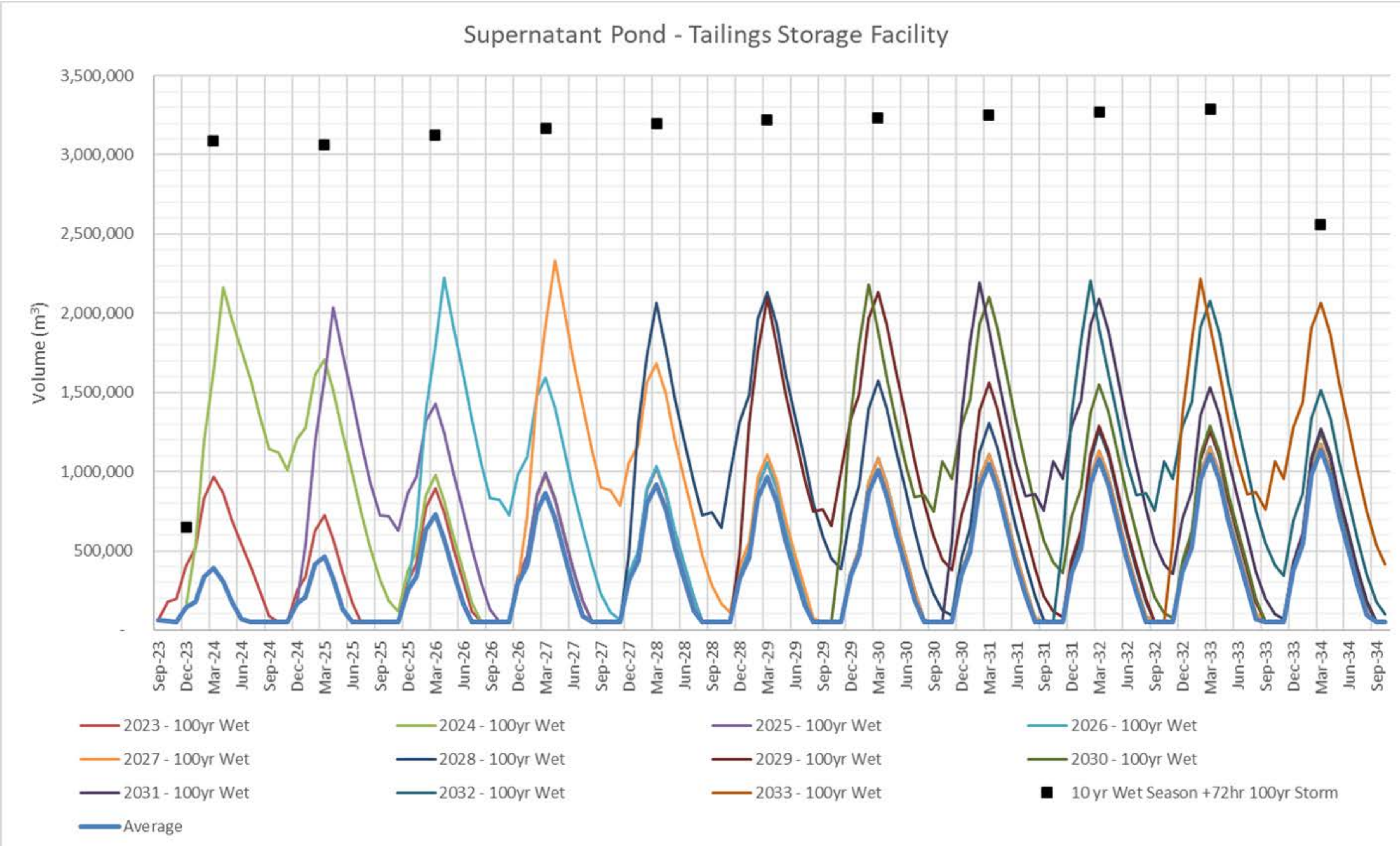


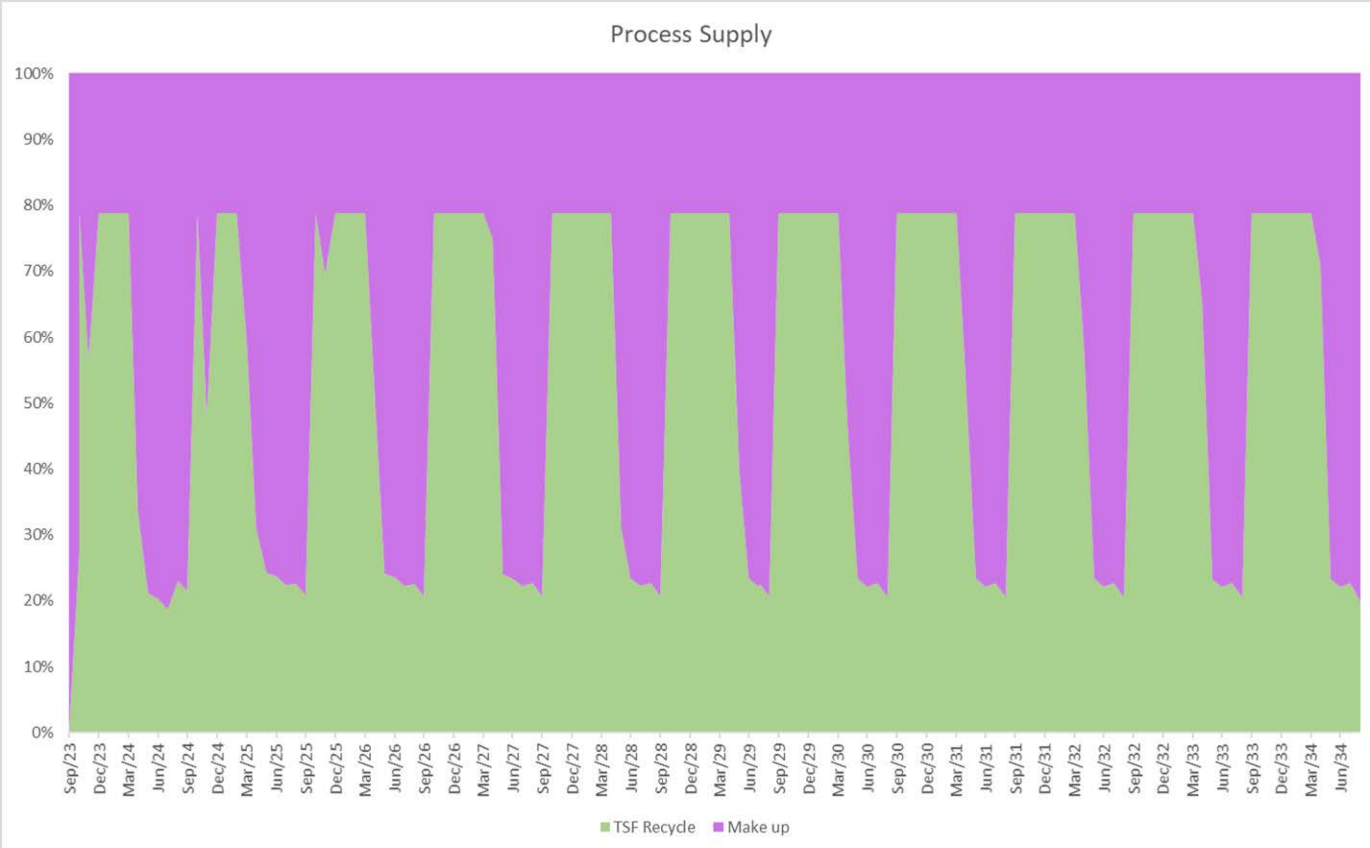
Supernatant Pond - Tailings Storage Facility
Volume (m³)
1,200,000
1,000,000
800,000
600,000
400,000
200,000
-
Sep-23
Dec-23
Mar-24
Jun-24
Sep-24
Dec-24
Mar-25
Jun-25
Sep-25
Dec-25
Mar-26
Jun-26
Sep-26
Dec-26
Mar-27
Jun-27
Sep-27
Dec-27
Mar-28
Jun-28
Sep-28
Dec-28
Mar-29
Jun-29
Sep-29
Dec-29
Mar-30
Jun-30
Sep-30
Dec-30
Mar-31
Jun-31
Sep-31
Dec-31
Mar-32
Jun-32
Sep-32
Dec-32
Mar-33
Jun-33
Sep-33
Dec-33
Mar-34
Jun-34
Sep-34
Average

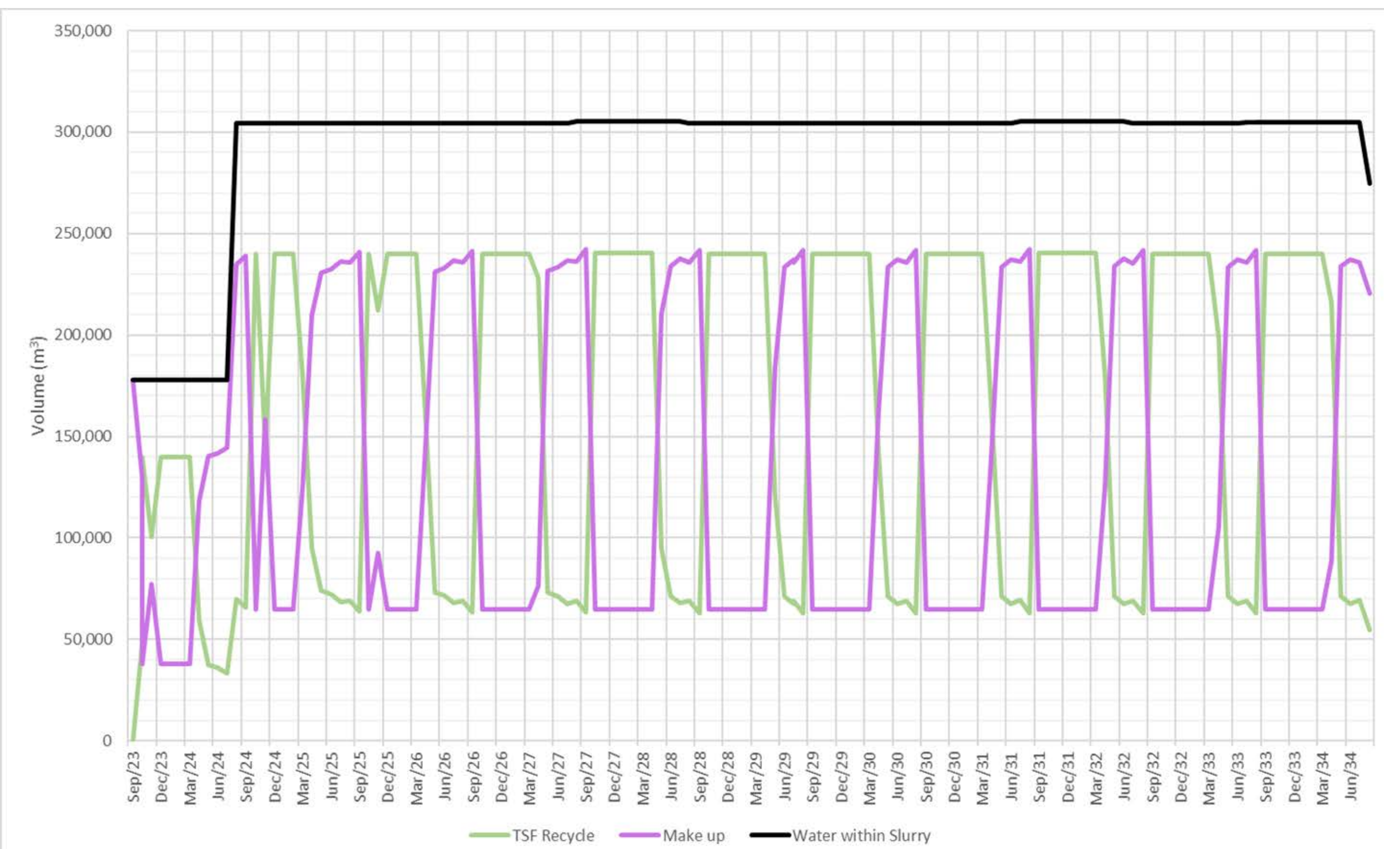
Figure 3.1


Process Supply
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Sep/23
Dec/23
Mar/24
Jun/24
Sep/24
Dec/24
Mar/25
Jun/25
Sep/25
Dec/25
Mar/26
Jun/26
Sep/26
Dec/26
Mar/27
Jun/27
Sep/27
Dec/27
Mar/28
Jun/28
Sep/28
Dec/28
Mar/29
Jun/29
Sep/29
Dec/29
Mar/30
Jun/30
Sep/30
Dec/30
Mar/31
Jun/31
Sep/31
Dec/31
Mar/32
Jun/32
Sep/32
Dec/32
Mar/33
Jun/33
Sep/33
Dec/33
Mar/34
Jun/34
Sep/34
TSF Recycle
Make up


Volume (m³)
350,000
300,000
250,000
200,000
150,000
100,000
50,000
0
Sep/23
Dec/23
Mar/24
Jun/24
Sep/24
Dec/24
Mar/25
Jun/25
Sep/25
Dec/25
Mar/26
Jun/26
Sep/26
Dec/26
Mar/27
Jun/27
Sep/27
Dec/27
Mar/28
Jun/28
Sep/28
Dec/28
Mar/29
Jun/29
Sep/29
Dec/29
Mar/30
Jun/30
Sep/30
Dec/30
Mar/31
Jun/31
Sep/31
Dec/31
Mar/32
Jun/32
Sep/32
Dec/32
Mar/33
Jun/33
Sep/33
Dec/33
Mar/34
Jun/34
Sep/34
TSF Recycle
Make up
Water within Slurry


Supernatant Pond - Tailings Storage Facility
Volume (m³)
2023 - 100yr Wet
2024 - 100yr Wet
2025 - 100yr Wet
2026 - 100yr Wet
2027 - 100yr Wet
2028 - 100yr Wet
2029 - 100yr Wet
2030 - 100yr Wet
2031 - 100yr Wet
2032 - 100yr Wet
2033 - 100yr Wet
10 yr Wet Season +72hr 100yr Storm
Average


Process Supply
100%
90%
80%
70%
60%
50%
40%
30%
20%
10%
0%
Sep/23
Dec/23
Mar/24
Jun/24
Sep/24
Dec/24
Mar/25
Jun/25
Sep/25
Dec/25
Mar/26
Jun/26
Sep/26
Dec/26
Mar/27
Jun/27
Sep/27
Dec/27
Mar/28
Jun/28
Sep/28
Dec/28
Mar/29
Jun/29
Sep/29
Dec/29
Mar/30
Jun/30
Sep/30
Dec/30
Mar/31
Jun/31
Sep/31
Dec/31
Mar/32
Jun/32
Sep/32
Dec/32
Mar/33
Jun/33
Sep/33
Dec/33
Mar/34
Jun/34
TSF Recycle
Make up


350,000
300,000
250,000
200,000
150,000
100,000
50,000
0
Volume (m³)
Sep/23
Dec/23
Mar/24
Jun/24
Sep/24
Dec/24
Mar/25
Jun/25
Sep/25
Dec/25
Mar/26
Jun/26
Sep/26
Dec/26
Mar/27
Jun/27
Sep/27
Dec/27
Mar/28
Jun/28
Sep/28
Dec/28
Mar/29
Jun/29
Sep/29
Dec/29
Mar/30
Jun/30
Sep/30
Dec/30
Mar/31
Jun/31
Sep/31
Dec/31
Mar/32
Jun/32
Sep/32
Dec/32
Mar/33
Jun/33
Sep/33
Dec/33
Mar/34
Jun/34
TSF Recycle
Make up
Water within Slurry

APPENDIX E

Turret Brochure

THE TURRET

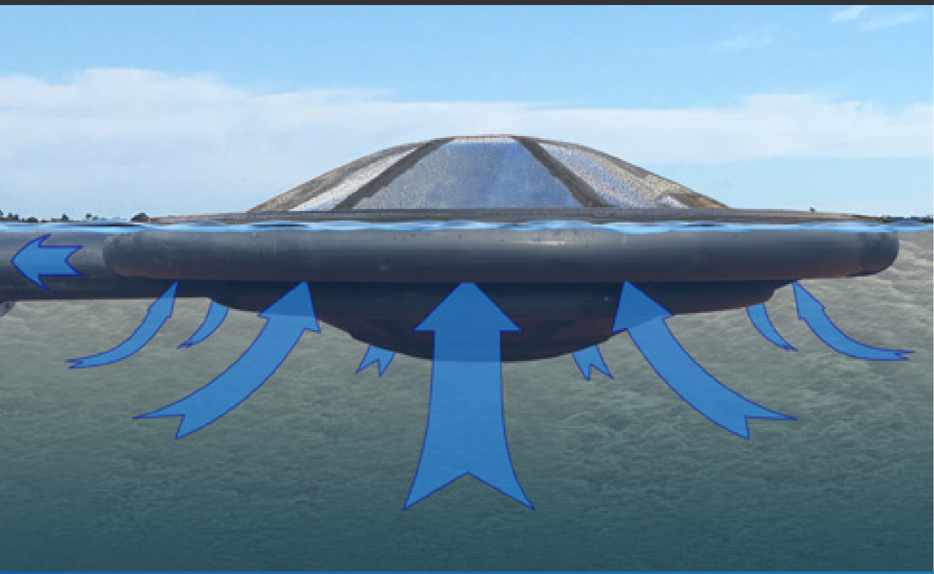
SHALLOW WATER? NO PROBLEM.

A floating water intake device
for shallow water.

PRODUCT DESCRIPTION

RBH ENGINEERING


RBH
ENGINEERING



INTRODUCING THE TURRET

The Turret is a floating water intake device that solves the problems of vortexing, pump cavitation and drawing from shallow water with its patented radial geometric design.

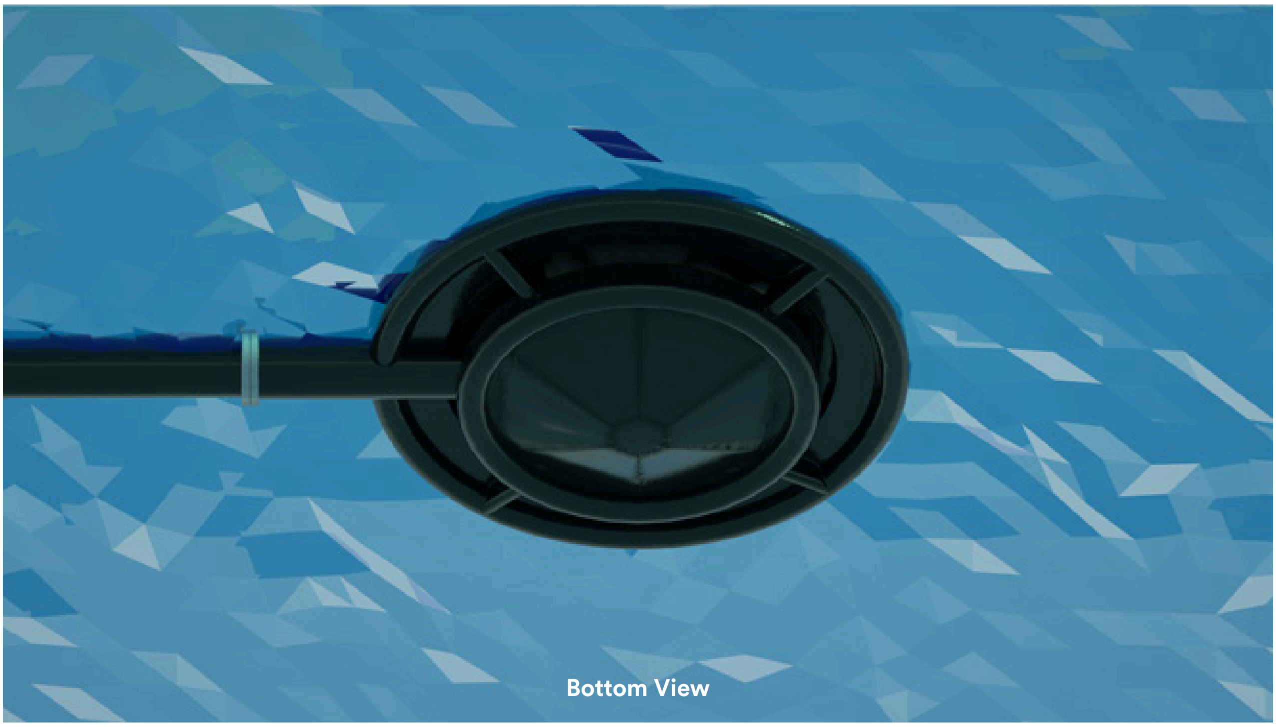
SPACE-AGE SHAPE – LEGENDARY PERFORMANCE

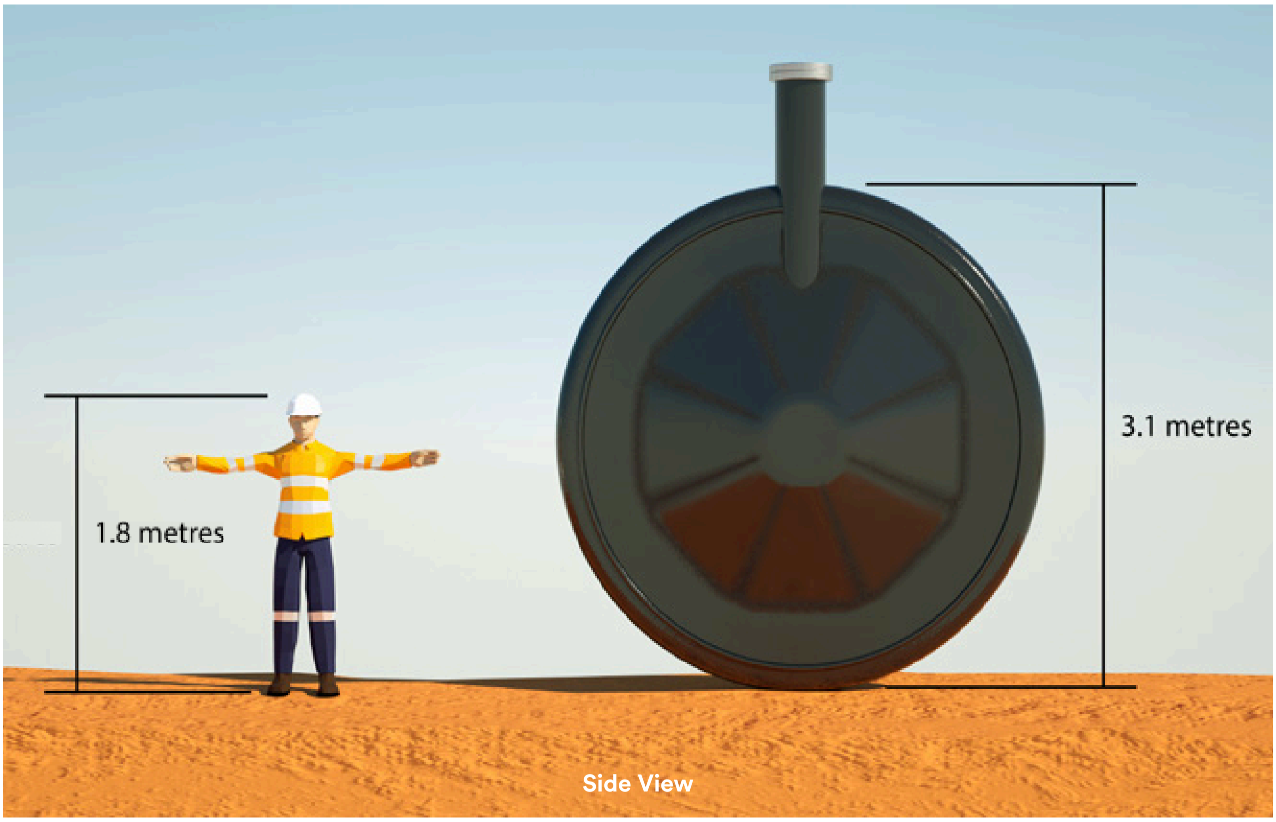
The Turret creates a barrier against vortexing and is a water intake solution for dewatering, tailings dams, decant ponds and raw water dams. It is an all-round solution for shallow water bodies:

- Mobile decanting: The Turret is a replacement for water intakes on existing decant barges and siphon decant systems. Because it takes water from just under the surface, the Turret permits much shallower decant ponds without the need to change the existing decant structure.
- Fixed decanting: When combined with a skid-mounted pump, the Turret is also a land-based, movable, lower capex and safer alternative to fixed decant systems such as cascade decant towers, seepage decant towers and embankment drains.
- Shallower tailings dams: The Turret can draw water from a depth of only 400mm. This means tailings dams can be much shallower and have much less surface area. This can result in lower evaporation losses, increased drying of tailings, improved tailings densities and reduced frequency of embankment raises.

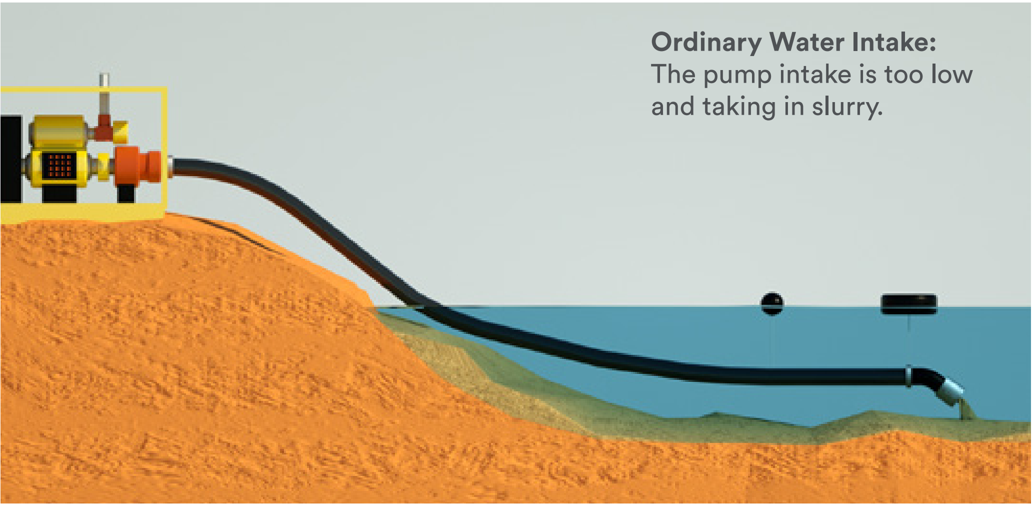
TOUGH, SAFE AND SIMPLE

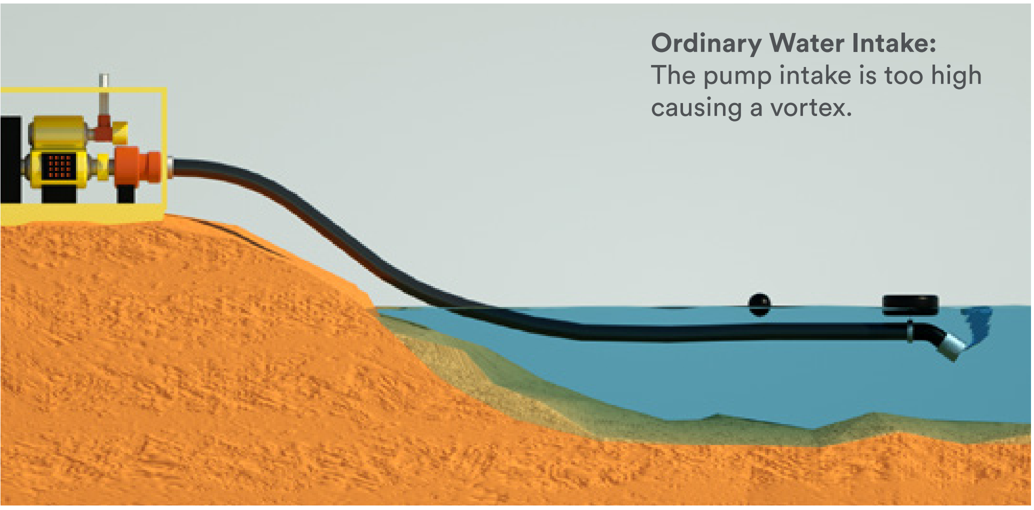
- The Turret is built tough. It is constructed of 10mm thick polyethylene and has been tested in the unforgiving environment of Australia's gold mines. Because of its futuristic design, only the Turret:
- Draws water down to 400mm.
- Creates a barrier against vortexing.
- Can be easily relocated with minimal equipment and human intervention.
- Is scalable – so you can add as many Turrets as you need to the one pump, depending on pump capacity and suction size.

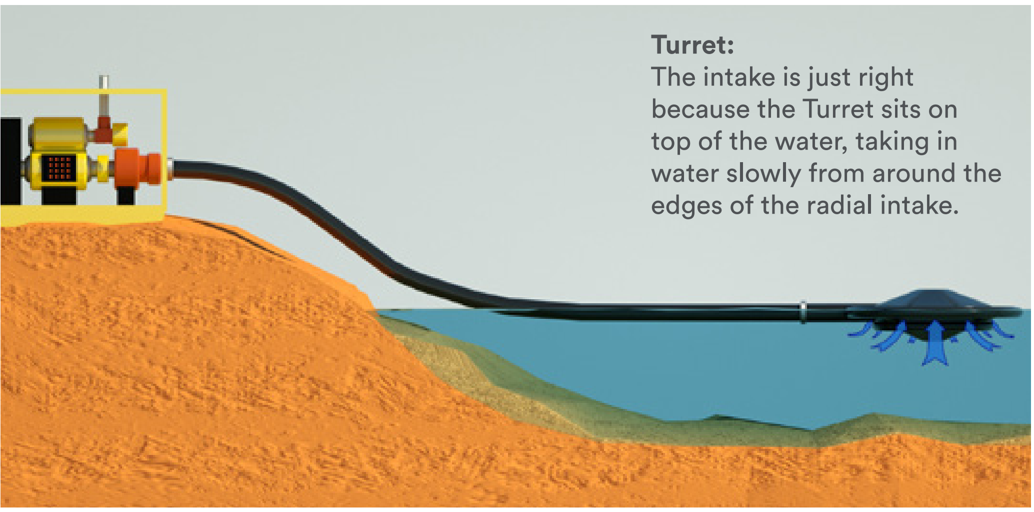

Bottom View


1.8 metres
3.1 metres
Side View

HOW DOES THE TURRET WORK?







THE GOLDEN RULE

The golden rule is that the pump suction must be:

- Sufficiently above the bottom of a dam to avoid slurry, sand or mud being drawn into the pump, and
- Sufficiently submerged to avoid the formation of vortexes on the water surface.

The damage that slurry, silt and debris can do to a pump is well known. The damage a vortex can do is more insidious. A vortex causes air to enter the pump through the suction line, which leads to reduced efficiency and may lead to pump damage. A vortex can also cause cavitation damage to pumps.

THE TURRET ADVANTAGE

The Turret floats on the surface, away from slurry, sand or mud.

The slow speed of water intake around the wide radial intake means slurry, sand and mud are not stirred up. Taking water from around the surface prevents vortexing, which damages pumps and creates downtime.

RUGGEDNESS WITH PLENTY OF PERFORMANCE

Specification	3m Turret
Diameter	3110mm (122 inches)
Height	839mm (33 inches)
Weight	350kg (771 pounds)
Minimum operational depth	400mm (15.7 inches)
Maximum flow	1,000m³ per hr (588 cubic feet per minute)
Minimum flow	10m³ per hr (5.88 cubic feet per minute)
Intake pipe diameter	315mm (12.6 inches) OD or 355mm (14.2 inches) OD
Material	10mm (0.39 inches) high density polyethylene
Colour	Black
Construction method	Polywelded

CRITICAL SUBMERGENCE

There is a relationship between the depth of the submerged suction intake and the intake velocity of the water. The “critical submergence” is the minimum submergence needed to prevent the formation of vortexes.

YOU CAN CALCULATE THE CRITICAL SUBMERGENCE OF YOUR INTAKE WITH FORMULAS...

CRITICAL SUBMERGENCE CALCULATION FORMULAS

Author / Origin	Date	Formula
Hydraulic Institute	1998	$D \cdot (1 + 2.3 \cdot Fr)$
Pumping System Manual		$(V^2/2g) + 0.5$
Prosser	1977	1.5* D
Paterson & Noble	1982	$D \cdot (1 + 2.5 \cdot Fr)$
Hecker	1987	$D \cdot (1 + 2.3 \cdot Fr)$
Knauss	1987	$D \cdot (0.5 + 2 \cdot Fr)$
Flyght	2002	$1.7 \cdot Fr$
Werth & Frizzell	2009	$D \cdot (2.1 + 1.33 \cdot Fr^{0.67})$

...OR YOU CAN USE THE TURRET

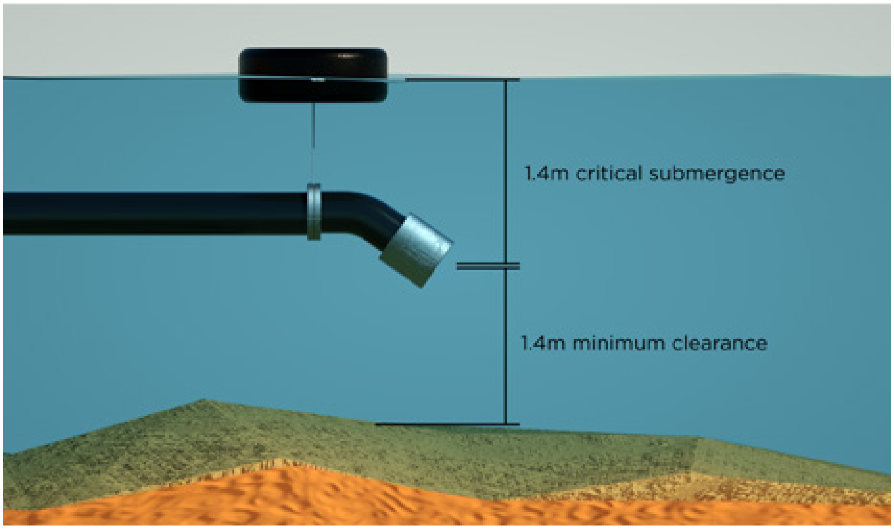
The Turret floats on the surface and automatically keeps the correct critical submergence in water as shallow as 400mm.

THE PRINCIPAL OF FALSIFICATION

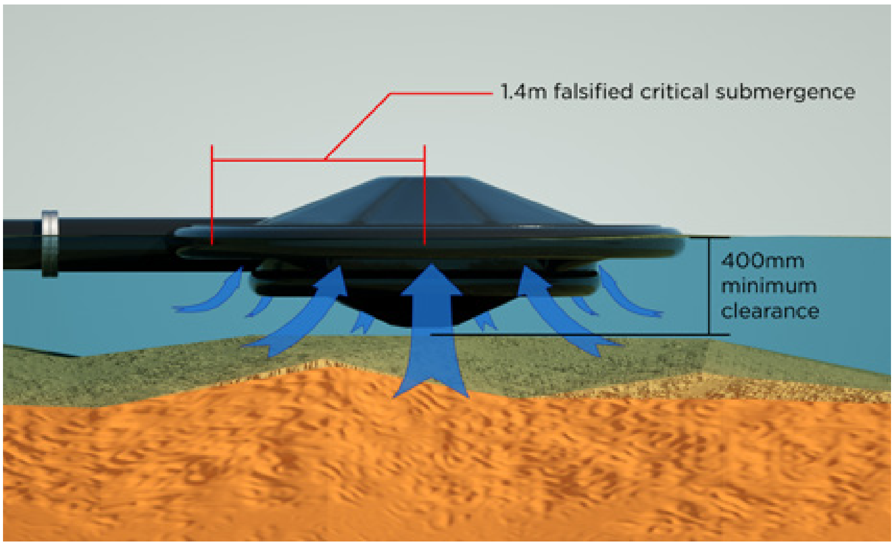
The Turret works on the Principle of Falsification. This principle calculates the critical submergence – and turns it sideways by using a vacuum. Instead of requiring critical submergence *from above* the suction intake, the Turret draws water *from the side* of the intake. The radius of the Turret is equal to the height of the critical submergence.

For example:

- If a pump drawing 1,000m^3/hr of water through a 315mm diameter intake had a critical submergence of 1.4m, this means that the suction intake must be 1.4m below the surface.
- By using the Principle of Falsification, however, the Turret draws the water from 1.4m to the side of the water intake.
- The Turret has falsified the critical submergence by using a vacuum inside the Turret to take water *from the side* of the suction intake, not above it.
- In this example, the pump could draw water from a depth of as shallow as 400mm instead of 1.4m needed for ordinary intakes.

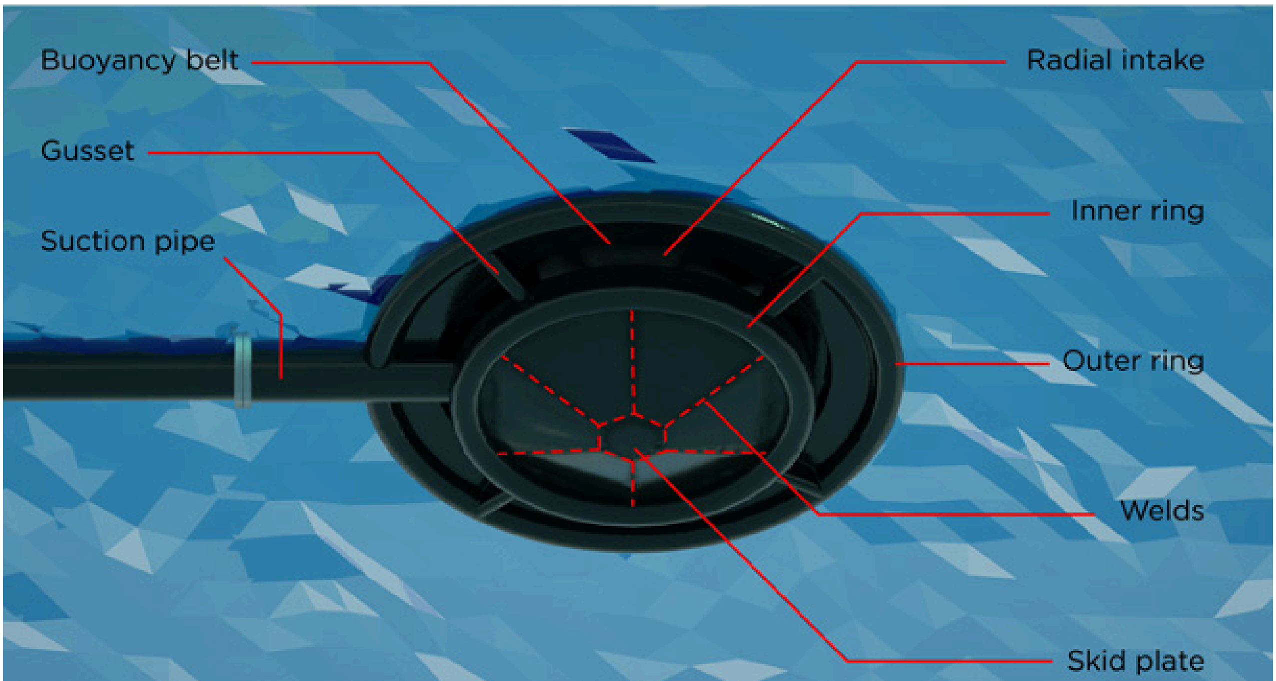


Pump intake in water with example critical submergence of 1.4m

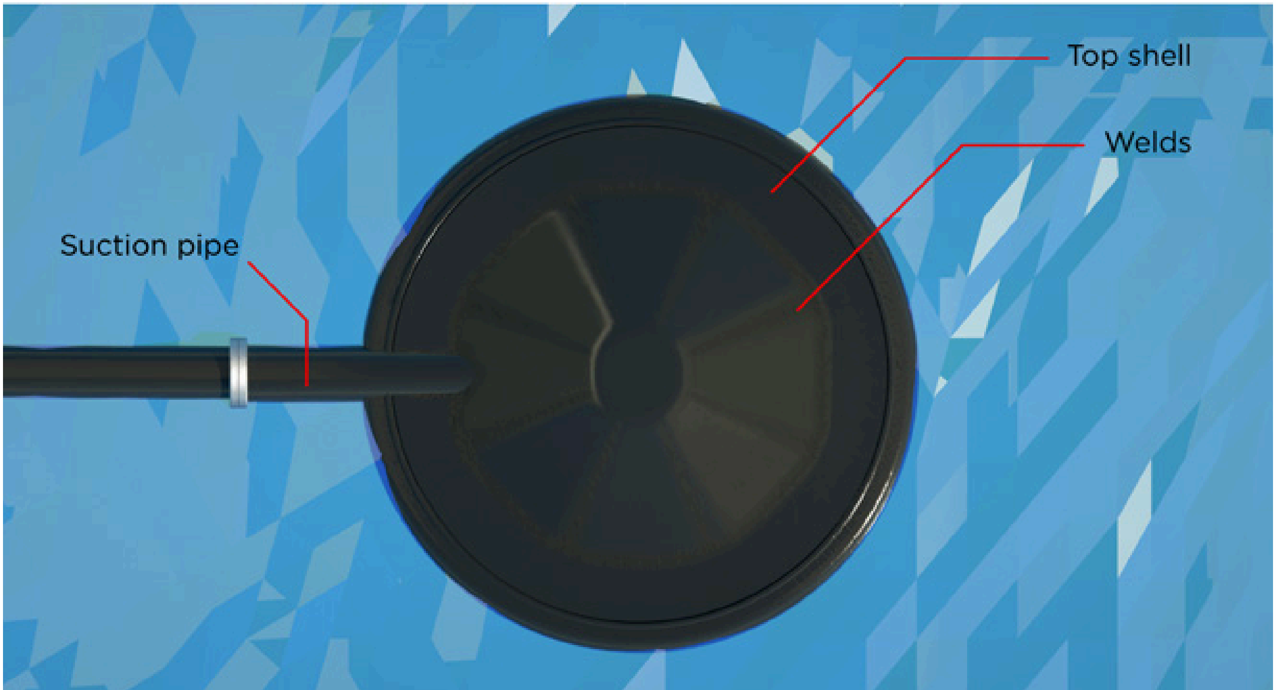


Turret floating on top of water, operating in only 400mm of water.

TURRET DESIGN FEATURES



Bottom View



Top View

SMART THINKING ALL THE WAY

The Turret gives priority to increased safety, reduced costs and reliability. The design was field tested at Newmont Boddington Gold Mine for more than two years. The Turret gives you a long-life, safe and reliable water intake for your mining pumps with a barrier against vortexing.

Outer Ring

The tough 160mm PN16 polyethylene Outer Ring provides the seal around the water so that air can't get into the Turret. This Outer Ring is filled with air and sealed to provide buoyancy and flotation for the Turret.

Radial Water Intake

The wide diameter of the Radial Water Intake means that even though your pump might be drawing 1,000 cubic metres per hour, the velocity of the water at the outside of the Turret is much, much lower than the velocity at the main intake of the suction line. The benefit of this is that even at depths of only 400mm, a 1,000 cubic metre an hour pump can operate without the danger of vortexing or getting air into your pump.

Buoyancy Belt

Between the Outer Ring and the Inner Ring is the Buoyancy Belt filled with extruded polyurethane foam. The advantage of this is that the Turret will sit level in water at the right depth.

Inner Ring

The 110mm PN16 Inner Ring is also filled with air and sealed. The Inner Ring is poly welded to the Top and Bottom Shells. It has eight polyethylene gussets welded at equal distances around it and onto the Outer Ring. This provides strength and torsional stiffness.

Top and Bottom Shells

The Top Shell and Bottom Shell are made of high density 10mm polyethylene. Each Shell has eight equidistant poly welds that provide ribs of strength through the Shell. Poly welding of the Top Shell to the Bottom Shell and the Inner Ring minimises the risk of failure inherent in mechanical fixings such as nuts and bolts. The round shape of the Bottom Shell makes the Turret easy to slide in or out of the water in muddy or rocky conditions. The round shape also helps the Turret slide easily over a polyethylene tailings dam liner without ripping the liner.

Skid Plate

An additional 10mm polyethylene skid plate is poly welded to the Bottom Shell. This means that dragging the Turret around a mine site simply ablates the Skid Plate and this minimises the risk of damage to the Bottom Shell itself.

Suction Pipe

There is a choice between 355mm or 315mm diameter polyethylene PNIO Suction Pipe. Both are designed to maximise the flow up to 1,000 cubic metres an hour. The 355mm Suction Pipe is used to minimise line loss when the Turret is further out into the water body, whilst the 315mm Suction Pipe is used closer to the shore. The Suction Pipe is poly welded between the Top Shell and the Bottom Shell for structural integrity.

Stub End

Both the 355mm and 315mm polyethylene Stub Ends are butt welded to the Suction Pipe. Special care is taken to grind out the butt weld line on the inside of the Intake Pipe so that there is low-to-zero turbulence as water passes over the weld line.

Backing Ring

A high quality 355mm or 315mm Table E Backing Ring is fitted up against Stub End so that the Turret can be connected to a suction line by bolting up to mining hose or poly pipe.

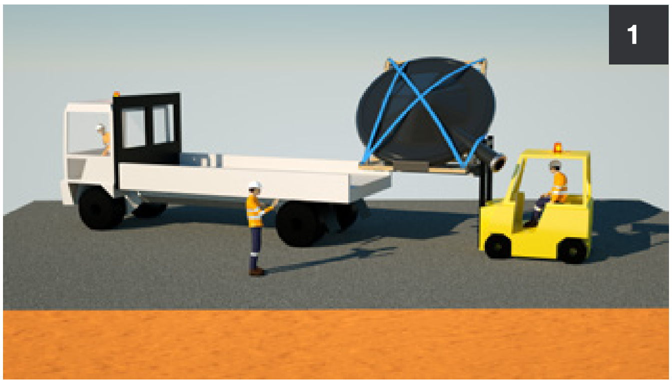
Optional Screen

The Turrent has an optional Filter Screen. The width of the Outer Ring means the water approaching the Radial Water Intake is so slow that foreign objects often just drift past – and those that don't are more often than not caught in the Filter Screen before they have a chance to enter the Turret and the pumping system.

Optional Floating Coupling

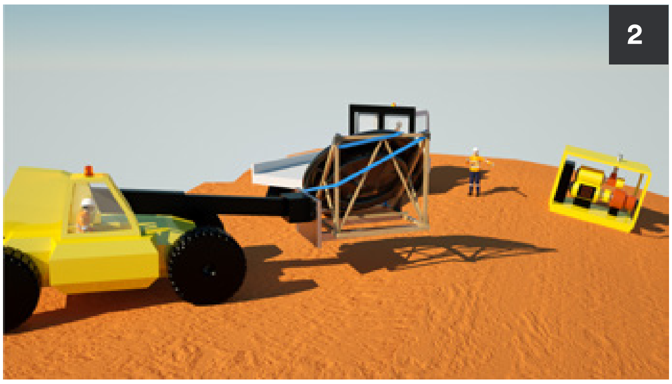
The optional snap-on Floating Coupling from RBH makes connecting the Turret to the pump Suction Line a breeze – taking only minutes to snap together.

HOW TO USE THE TURRET

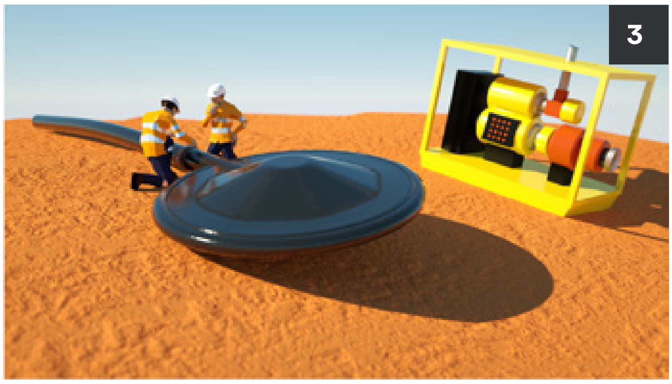


1. Take the Turret to site on a light work truck – the biggest Turret is only 350kg.

a) The Turret comes with a carry frame so that it can be strapped into a standard 2.4m wide truck tray at 45-degree angle so it does not protrude over the sides of the truck.



2. Remove the Turret from the vehicle in its frame with a telehandler or forklift.



3. Unbox the Turret, lay it level on the ground and connect it to the suction line.

a) If using a flood prime system, fit a non-return valve between the suction line and the Turret.

b) If using a vacuum system, a non-return valve is not required.

4. Fix floats to your suction pipe if necessary.

a) Heavy mining hose or heavy wall polyethylene (PN11 or above) needs a half float or a full float to keep the suction pipe level with the top of the water.

b) Lighter polyethylene (PN10 and below) usually doesn't need a float.

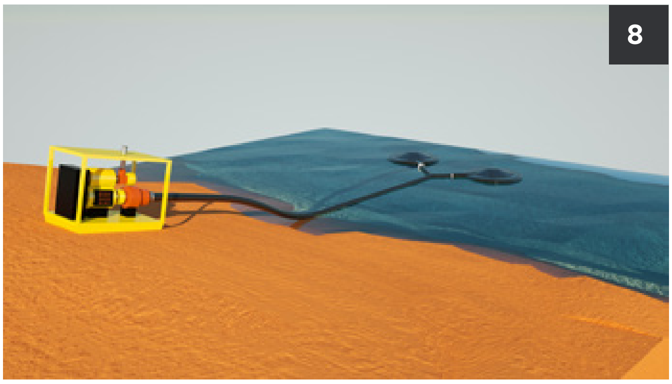
Combining patented smart design, tough polyethylene construction and zero moving parts, the Turret provides a safe and cost-effective water intake solution for the demands of tailings dams, decant ponds and raw water dams. The Turret is easily moveable and can operate in as little as 400mm of water. All this means you get uninterrupted production, lower costs and higher safety.

5. Slide the Turret and the suction pipe into the water keeping them both level.

6. Connect the suction pipe to the pump. Wait five minutes for the bottom of the Turret to fill with water and allow the Outer Ring of the Turret to touch the water level.

7. Prime the pump.

a) Once primed, there will be slight air turbulence coming through pump for two to three minutes which is the last of the air in the Turret. Then water will flow normally.



8. To increase water intake more than 1,000m^3 per hour, put a Y piece or Christmas Tree manifold in the suction line connect more Turrets. This is dependent on pump capacity and suction size.

HOW THE TURRET DRAMATICALLY IMPROVES TAILINGS DAM EFFICIENCIES

In this article by Dr Ross De Kretser, formerly Rio Tinto Technology and Innovation's Principal Advisor for Water, Waste and Tailings, you can read how the Turret can:

- Significantly reduce water loss through evaporation.
- Dramatically:
  - increase the drying of tailings,
  - improve tailings densities, and
  - reduce the frequency of embankment raises.
- Reduce the risk of seepage-related tailings embankment stability risks.

POTENTIAL OPERATING BENEFITS OF THE TURRET IN A TAILINGS STORAGE FACILITY

By Dr Ross De Kretser, B. Eng (Chem); Ph.D.

Former Principal Advisor, Water, Waste and Tailings with Rio Tinto Technology and Innovation Pty Ltd.

Operating a tailings storage facility (TSF) with as small a decant pond as is practically possible can deliver the following benefits:

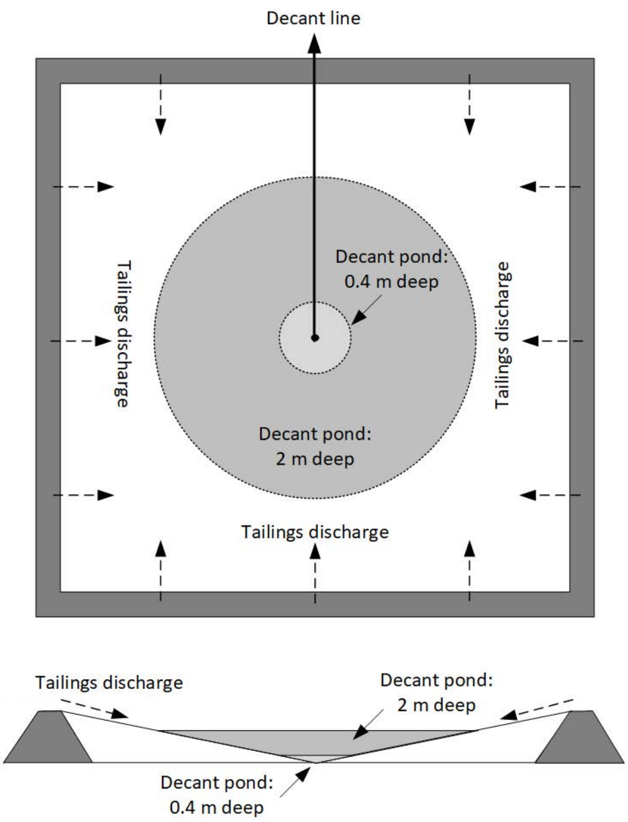
- **Water efficiency:** Evaporative losses from the decant pond are directly proportional to its area. A reduction in decant pond size will directly reduce water losses from it, positively impacting site fresh water demand.
- **Tailings management costs:** A smaller decant pond exposes a greater proportion of the tailings to evaporative drying and gravity drainage effects. This promotes improved tailings densities, efficient storage utilisation and a reduced frequency of embankment raises.
- **Tailings management risk:** A greater buffer can be maintained between the decant pond and embankments, reducing seepage-related embankment stability risks. Alternatively, design constraints (e.g. TSF size, embankment construction method) relating to seepage risk can be positively impacted, with associated cost reduction implications.

Through its ability to draw water from a much shallower depth than ordinary decant water intakes, the Turret can allow operation of a TSF with dramatically reduced decant pond sizes resulting in the potential to realise all of the above operational benefits. Importantly, these benefits do not necessarily require the complete reconfiguration of a decant system design (e.g. replacement of a floating pontoon with a shore-based system), rather only the replacement of a traditional pump intake with a Turret to allow shallow decant pond operation, as is illustrated in the following conceptual scenario.

1 Calculations and numbers quoted are conceptual and indicative. They are based on simplistic calculations that, for instance, do not account for all aspects of the water balance around a TSF. Actual benefits will be highly site-specific, and other factors (e.g. suspended fine solids) may also impact the practical minimum operating depth of the TSF.

COMPARISON OF A STANDARD 2M DEEP WATER INTAKE COMPARED TO A TURRET'S 400MM DEEP RADIAL WATER INTAKE

The benefits of the Turret can be demonstrated for a conceptual square, 500m x 500m TSF operating with perimeter discharge designed to maintain a central decant pond (see illustration).



Schematic of conceptual perimeter discharge TSF operating with conventional 2m deep and Turret, 0.4m deep decant ponds (not to scale).

The conceptual pond will therefore be circular and decant removal is from a central, deepest point. Tailings beach angles vary, but a beach angle of 1.5% is considered reasonably common, and used in this example (i.e. for every 100m of tailings beach horizontally from the discharge point, the beach elevation decreases by 1.5m).

Using this information, the required radius and resultant area of decant pond and exposed tailings beach area were determined for two scenarios:

- a standard decant intake requiring a minimum 2m pond depth, and
- a Turret requiring only 0.4m minimum pond depth.

The comparison between the two scenarios is:

1. Standard intake, 2m pond depth:
   a. Required minimum pond radius around the decant intake: 133m
   b. Resultant decant pond surface area: ~70,500m^2
   c. Exposed beach area: ~179,500m^2 or 72% of total TSF area.
2. Turret intake, 0.4m pond depth:
   a. Required minimum pond radius around the decant station intake: 27m
   b. Decant pond surface area: ~2,300m^2
   c. Exposed beach area: ~247,700m^2 or 99% of total TSF area.

For this conceptual scenario, the use of Turret allows the potential for[1]:

1. 96% reduction in the decant pond area, which also reduces evaporative losses from the pond itself by 96%.
2. At an example daily mean evaporation rate of 10mm per day, this equates to a reduction in evaporative losses of 247 megalitres per year from the decant pond.
3. 38% increase in exposed tailings beach area, enhancing tailings drying with potential improvement in deposited density resulting in less frequent embankment raises and lower associated costs.

Whilst indicative in nature, and specific to the square TSF configuration depicted, the conceptual scenario presented illustrates the potential magnitude of the benefit delivered by the Turret to an operation from environmental, reputational, risk management and economic perspectives. Similar benefits would also be evident in deployment of the Turret into other TSF configurations.

Ross de Kretser, B. Eng (Chem), Ph.D., director, Acclarium Tailings and Solid Liquid Separation Consulting, formerly Principal Advisor, Water, Waste and Tailings Rio Tinto Technology and Innovation Pty Limited

TOUGH. RELIABLE. SMART. SAFE. SCALABLE.

THE BENEFITS OF THE TURRET

	Decant Barge, Decant Tower etc.	Turret
Minimum pond radius	133m	27m
Decant pond surface area	~70,500m^2	~2,300m^2
Reduction in pond surface area	0%	96%
Exposed beach area	~179,500m^2	~247,700m^2
Exposed beach percentage	72% of total TSF area	99% of total TSF area
Increase in exposed beach area	0%	38%
Yearly evaporation saving	0 megalitres pa	247 megalitres pa

FEATURES, ADVANTAGES AND BENEFITS OF THE TURRET

Tailings Dams, Decant Ponds and Raw Water Dams

RECLAIMING WATER FROM SHALLOW DEPTHS

- Can draw water down lower than other conventional systems, thus returning more of your reclaimed water to processing.
- Can keep production going when other systems can't operate at low levels.
- Can operate down to 400mm.

SAFETY ISSUES

- The Turret can deal with large increments in tide and water level before it has to be moved, cutting down human involvement in operations.
- Moving the Turret in and out is easy because it slides across the ground and floats on the surface of the water.
- There is minimal interaction of humans with heavy equipment to install or move Turret.

SIMPLICITY OF TASK

- The work can be carried out on the roadway, tailings beach or side of the dam and doesn't have to be done close to the water.
- Installation is easy: take the Turret off the work vehicle with a telehandler or forklift lay it level on the ground and connect it to the suction line - minimal need for heavy equipment.
- Low-to-zero risk of tearing poly-lined tailings dams because of the dish shape design.

REPAIRS TO TURRET

- Low-to-zero repairs because there are no moving parts.
- The Turret is long-lasting because it is made from polyethylene and is UV resistant.
- Resists corrosion in tailings dams because it is made of polyethylene.

PRODUCTION

- Can extract greater volume of water because it draws water down to a lower depth.
- Draws water from just beneath the surface which has the least number of suspended particles, meaning fewer solids go into returned water.
- Less reliance on raw water because more reclaimed water from tailings dams is available.

COST

- Low-to-zero maintenance to Turret because there are no moving parts.
- Less damage to pumps because Turret draws from just beneath the surface where there are fewer contaminants and solids, thus fewer solids and corrosive chemicals in the pumps.
- Design creates a barrier to vortexing and therefore cavitation damage from vortexing when used as directed.

THE TURRET ADVANTAGE

The Turret floats on the surface, away from slurry, sand or mud.

The slow speed of water intake around the wide radial intake means slurry, sand and mud are not stirred up. Taking water from around the surface prevents vortexes forming, damaging pumps and creating downtime.

SHALLOW WATER? NO PROBLEM.

Are you ready to take charge? Then order your Turrets today.

ENQUIRIES AND ORDERS:

RBH Engineering, 39 Farmers Avenue,
Boddington, Western Australia 6390
T: +61 8 9883 8206
E: robh@rbhmechanical.com.au

RBH ENGINEERING

APPENDIX F

Seepage Assessment

APPENDIX F – SEEPAGE ANALYSIS

F.1. GENERAL

The seepage analysis program SEEP/W (Ref. F.1) was used to assess the phreatic surfaces in Tailings Storage Facility (TSF) embankments and the seepage loss from the TSF at Stage 1 and Stage Final.

F.2. SEEPAGE MODELS

The location of analysed section is shown in Figure F.1. The analysed section cut through the embankment with the maximum height and the pond, which represents the most critical conditions for this proposed facility.

The maximum pond elevation based on average weather condition was assumed to be RL 63.9 m for Stage 1 and RL 82.2 m for Stage Final. The maximum elevation of the storm pond was assumed to rise to RL 66.8 m for Stage 1 and RL 82.7m for Stage Final for a storm lasting for maximum 72 hours. Steady-state modelling was carried out to assess the cases with average pond, and transient modelling with storm pond.

F.3. MODEL ASSUMPTIONS

The cross-sections analysed for the TSF embankment at Stage 1 and Stage Final elevations were based on the following parameters:

Stage 1:

- Crest elevation: RL 70.0 m
- Upstream slope: 1V:2.5H
- Downstream slope: 1V:3H

Stage Final:

- Crest elevation: RL 87.4 m
- Upstream slope: 1V:2.5H
- Downstream slope: 1V:3H with 5m bench every 10m in height

All models have basin drains at 100 m spacing and designed basin lining (300 mm thick compacted clay layer within the facility).

The subsurface conditions were based on the site investigation. The model comprises the following:

- Tailings.
- 300 mm compacted soil liner layer – Zone A.

- HDPE Liner at TSF basin and upstream embankment face.
- Zone B material – heap leach material.
- Zone C material – mine waste.
- Basin drains at 100 m spacing.
- Toe drains at upstream embankment
- Transported and residual soil.
- Extremely weathered material.
- Highly weathered material
- A beach slope of 150H: 1V
- Tailings freeboard of 2.5m for Stage 1 and 0.5m for Stage Final

Groundwater was measured during the site investigation in July 2021, in the middle of the dry season (Ref. F.2). The boreholes and test pits investigations indicate the groundwater ranged from 0.4m to 12.0m in depth. Based on the above investigation observations, the seepage modelling adopted regional groundwater level as 0.5m below the ground.

F.4. MATERIAL PARAMETERS

The cross-sections of Stage 1 and Stage Final models are shown in figures F.2 and F.3. The material types and the saturated hydraulic conductivity parameters used in the models are based on the findings from geotechnical investigation (Ref. F2) and Tailings physical testing (Ref. F3) and are summarised in Table F.1.

Table F.1: Summary of Material Permeabilities

Material Type	Material Layer Thickness	Saturated Permeability (m/s)
Tailings	Varying	1×10^{-7}
Zone A	6 m	1×10^{-8}
Zone B	6 m	5×10^{-7}
Zone C	Varying	1×10^{-5}
Soil Liner	300 mm	1×10^{-8}
HDPE Liner	300 mm	1×10^{-9}
Transported and residual soil	4.5 m	1×10^{-7}
Extremely weathered material	0.5 m	5×10^{-7}
Highly weathered material	15 m	5×10^{-8}
Bedrock	N/A	5×10^{-8}

F.5. BOUNDARY CONDITIONS

The following boundary conditions were adopted in the analyses:

- A constant head boundary was assumed based on average climatic conditions.
- A constant head boundary has been assumed at the downstream of the TSF embankment.
- The downstream embankment face was set as a possible seepage face.

F.6. SCENARIO MODELLED

The following scenarios were modelled:

- Steady-state seepage model for Stage 1 configuration with average pond.
- Transient seepage model for Stage 1 configuration with storm pond.
- Steady-state seepage model for Stage Final configuration with average pond.
- Transient seepage model for Stage Final configuration with storm pond.

F.7. MODELLING RESULTS

The results of the seepage analysis are summarised in Table F.2, and shown in figures F.4 – F.7. Assuming the underdrainage does function, the estimated seepage loss is listed in Table F.3

Table F.2: Estimated Indicative Seepage Loss with functional underdrainage

Stage	Basin Drains Operation	Estimated Seepage Loss to Ground			Figure
		2D model ($m^3/s/m$)	Whole facility (L/day)*	Whole facility (kL/ha/day)*	
Stage 1	Average pond	6.06×10^{-8}	4.50	0.03	F.4
	Storm pond	8.32×10^{-8}	6.18	0.04	F.5
Stage Final	Average pond	5.88×10^{-8}	4.37	0.02	F.6
	Storm pond	6.10×10^{-8}	4.53	0.02	F.7

Table F.3: Estimated indicative Seepage Loss with non-functional underdrainage

Stage	Basin Drains Operation	Estimated Seepage Loss to Ground		
		2D model ($m^3/s/m$)	Whole facility (L/day)*	Whole facility (kL/ha/day)*
Stage 1	Average pond	1.97×10^{-7}	14.64	0.10
	Storm pond	2.49×10^{-7}	18.50	0.12
Stage Final	Average pond	4.80×10^{-7}	35.67	0.15
	Storm pond	4.86×10^{-7}	36.11	0.15

F.8. CONCLUSIONS

Based on the seepage assessment of Stage 1 and Stage Final of the proposed TSF configuration, the following conclusions are made:

- Phreatic surface in the embankment would be expected to be relatively low.
- The seepage loss is considered small for both Stage 1 and Stage Final with both average and storm ponds.

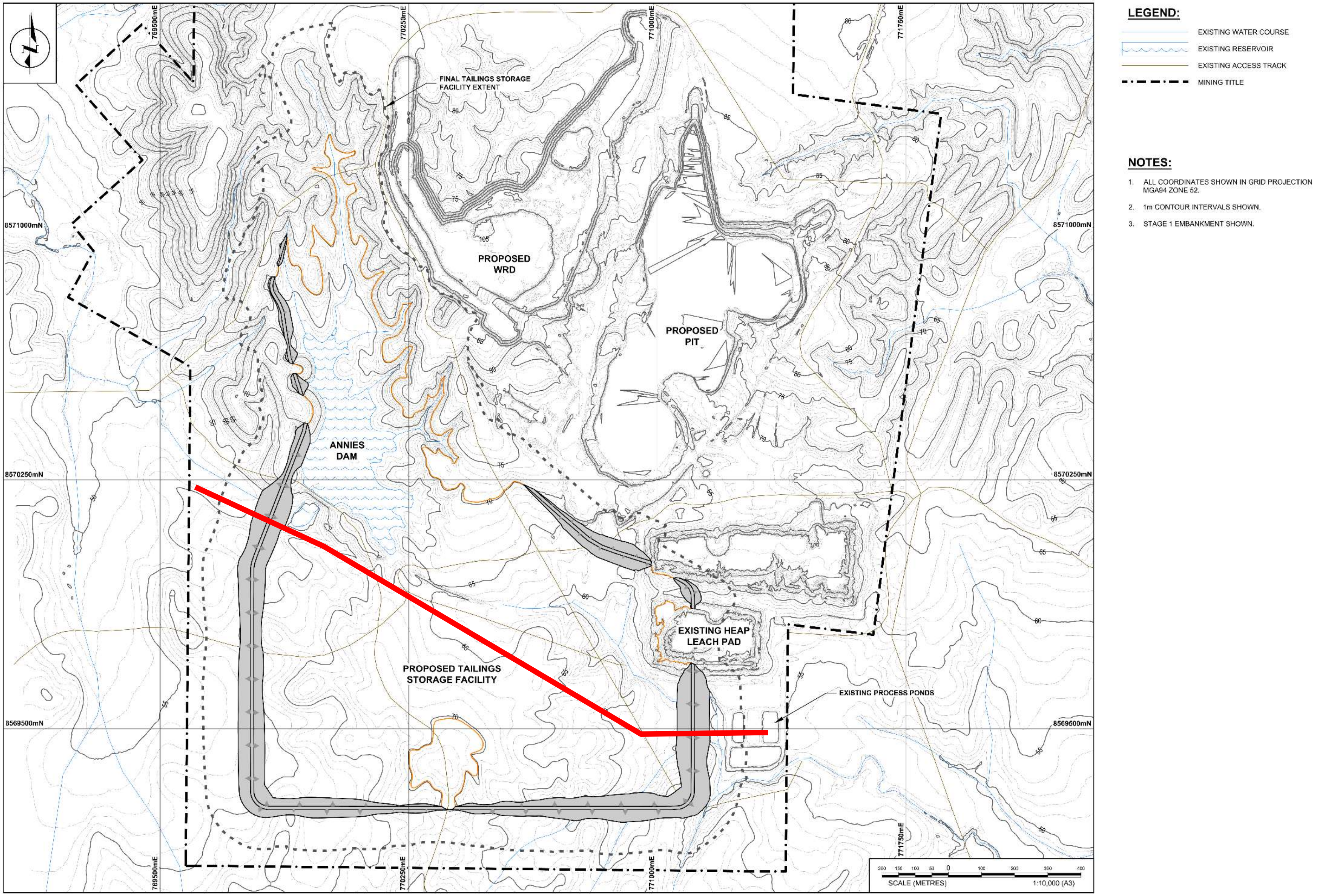
F.9. REFERENCES

F.1. GEO-SLOPE International Ltd.,“SEEP/W”, 2018.

F.2. Knight Piésold Australia, Report Ref. PE801-00102/03, “Definitive Feasibility Study Geotechnical Interpretative Report”, December 2021

F.3. Knight Piésold Australia, Memorandum Ref. PE801-00102/04, “Rustlers Roost Gold Project – Tailings Physical Testing”, December 2021

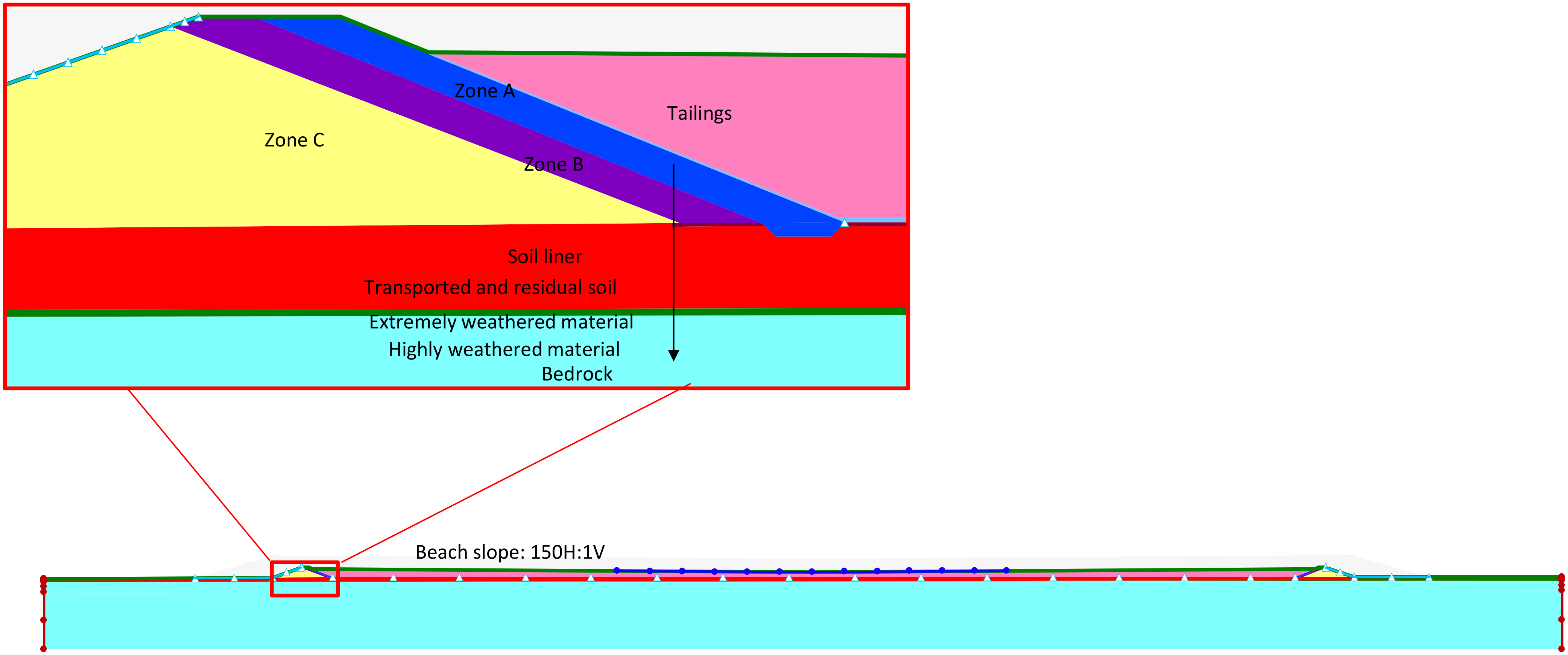
APPENDIX F – FIGURES

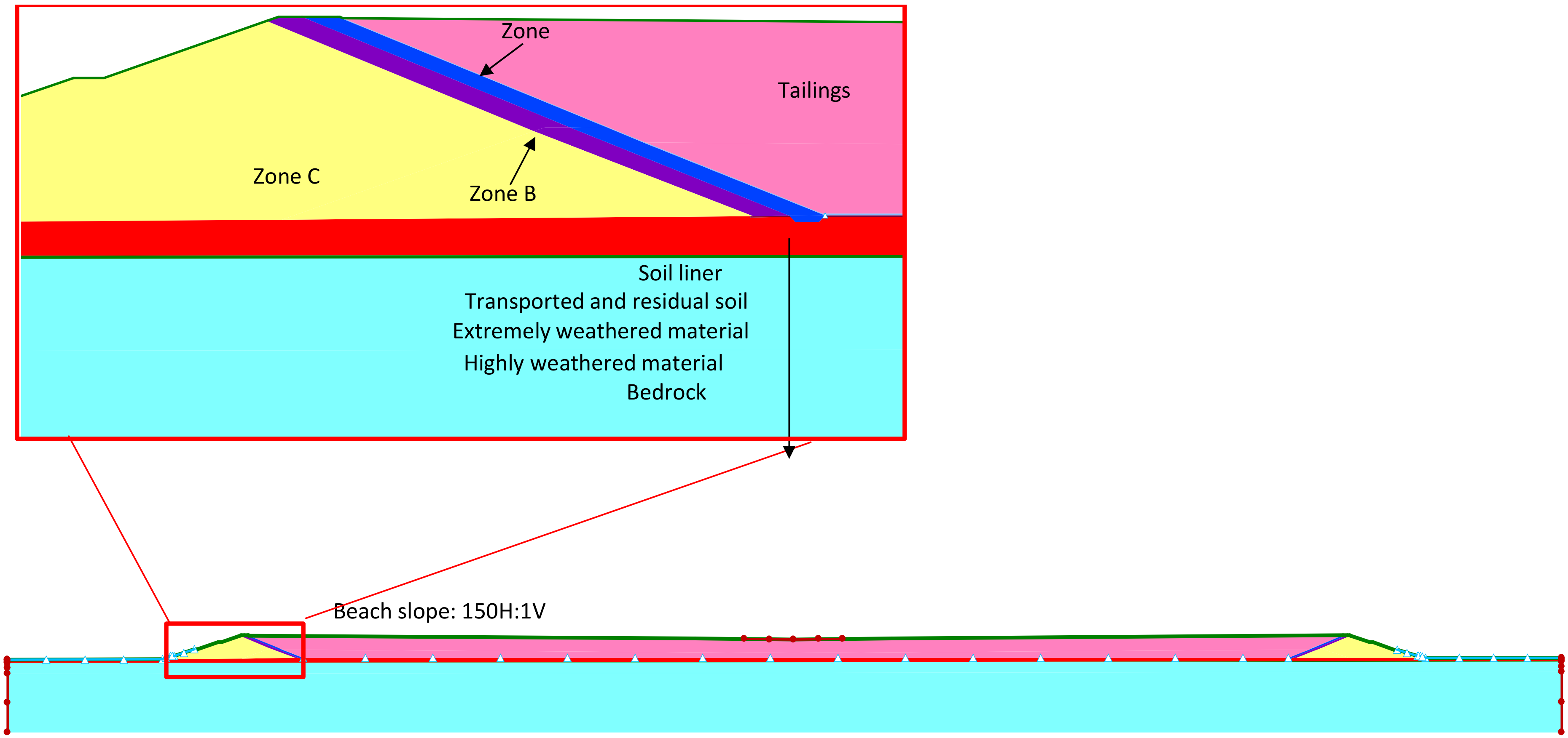


Knight Piésold CONSULTING

RUSTLERS ROOST GOLD MINE
TSF STAGE 1 AND FINAL SEEPAGE ASSESSMENT
LOCATION OF CUT SECTION

PE801-00102/04
Figure F.1

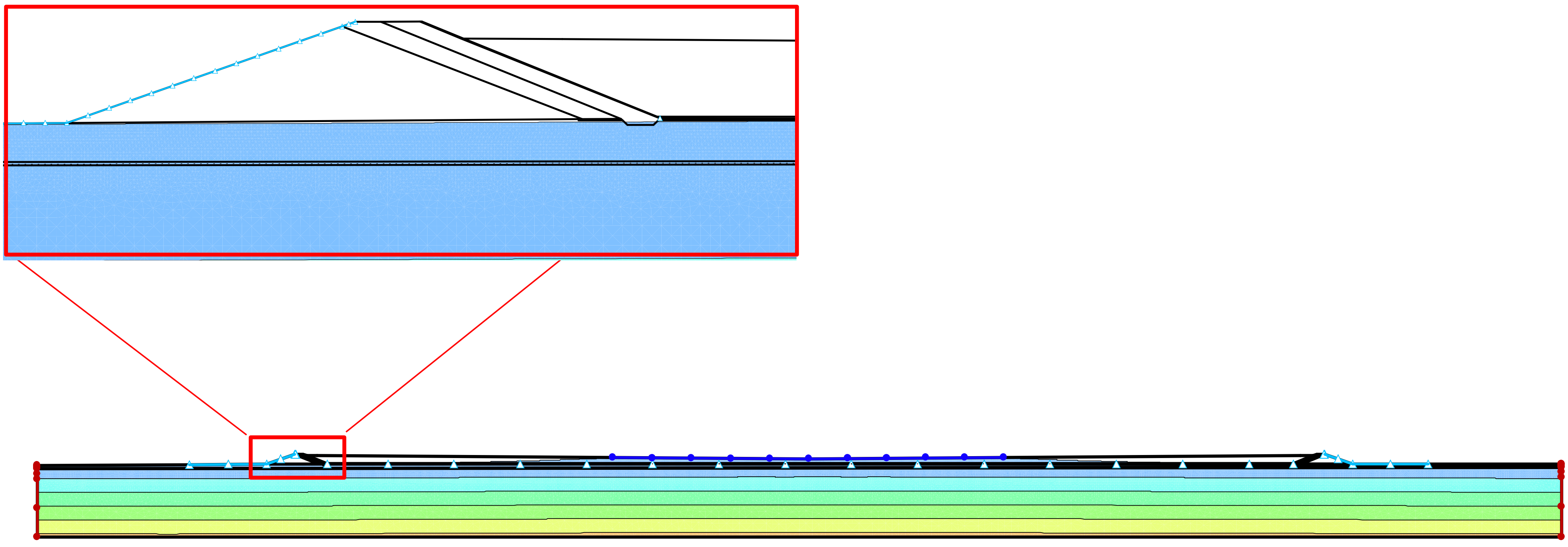

Zone A
Tailings
Zone C
Zone B
Soil liner
Transported and residual soil
Extremely weathered material
Highly weathered material
Bedrock
Beach slope: 150H:1V



Knight Piésold CONSULTING

RUSTLERS ROOST GOLD MINE
TSF STAGE 1 AND FINAL SEEPAGE ASSESSMENT
SEEPAGE MODEL - STAGE FINAL

PE801-00102/04
Figure F.3



RUSTLERS ROOST GOLD MINE
TSF STAGE 1 AND FINAL SEEPAGE ASSESSMENT
SEEPAGE RESULT - STAGE 1 WITH AVERAGE POND

PE801-00102/04
Figure F.4