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MHA GEOTECHNICAL

Feasibility Study
Kundip Tailings Storage Facility

Ravensthorpe Gold Project

ACH Minerals Pty Ltd

October 2018
Rev 4

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Report

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Synopsis:	This document details the findings of a feasibility study pertaining to the proposed Tailings Storage Facility at ACH Minerals' Ravensthorpe Gold Operation.

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Table of Contents

Figures

Tables

Appendices

1. TSF Proposal Summary

MHA Geotechnical (MHA) has prepared this feasibility study (FS) level design of the Kundip Mine Site tailings storage facility (TSF) at ACH's Ravensthorpe Gold Project (RGP) to support the overall project Feasibility Study into the technical and commercial viability of RGP. This report has been prepared following the format recommended in the Government of Western Australia Department of Mines and Petroleum's (DMP) *Guide to the Preparation of a Design Report for Tailings Storage Facilities*. This report is not intended to serve as the detailed design report for submission to the Department on Mines, Industry Regulation and Safety (DMIRS); further design development is required to advance from a FS level design to detailed design with issued for construction (IFC) documentation.

The Kundip Mine Site is situated approximately 20 km by road south-east of the town of Ravensthorpe and can be accessed from the Hopetoun-Ravensthorpe Road. The RGP site layout, Kundip mining tenement and location of the TSF relative the main project features is shown on Figure 1.

The Project schedule envisages total tailings production of 3.1 Mt. At an assumed average dry density of 1.5 t/m^3 for the stored tailings, the required tailings storage capacity is 2.0 Mm^3.

The proposed RGP TSF is a side-hill paddock-style facility. An engineered embankment will provide containment on three sides (east, south and west) whilst the natural topography will provide containment on the north side. The proposed TSF configuration is shown in plan and profile on Figure 6 and Figure 7. In accordance with the DMP Code of Practice (CoP) (DMP, 2013), the RGP TSF attracts a Medium hazard rating.

Construction of the RGP TSF will be undertaken in accordance with IFC drawings and earthworks specification. Furthermore, construction and operation will be in general accordance with the design intent of the final detailed design report.

Tailings are to be deposited from the main embankment in a sub-areal manner in thin lifts and beaching towards the northwest corner of the facility to form a decant pond away from the main embankment. The configuration and location of the decant pond provides capacity for the 1:100 annual exceedance probability (AEP) 72-hour storm event and DMP required freeboard.

It is envisaged that a detailed closure plan will be developed at a later stage in conjunction with an RGP site wide closure plan. The proposed RGP TSF has been developed with closure in mind, taking into consideration the DMP's principal closure objectives for rehabilitated mines and the Environmental Protection Authority's (EPA) objective for Rehabilitation and Decommissioning to ensure that premises are decommissioned and rehabilitated in an ecologically sustainable manner.

2. TSF Design Considerations

2.1 Introduction

MHA Geotechnical Pty Ltd (MHA) have been engaged by ACH Minerals Pty Ltd (ACH) to provide engineering design services as part of a Feasibility Study (FS) level design of the Kundip Mine Site Kundip tailings storage facility (TSF) at ACH's Ravensthorpe Gold Project (RGP).

The overarching objective of this study is to develop a TSF concept through to a FS level. The output of this work will be incorporated into the overall project Feasibility Study (undertaken by others) into the technical and commercial viability of the RGP.

2.2 Background

RGP is larger in scale than the previously approved Phillips River Project, which was to be developed at the same site. Mining of gold and copper bearing ore will be focused on a combination of open-pits and underground mining at Kundip. Processing and tailings storage will also be contained within the Kundip mining leases.

The Kundip Mine Site is situated approximately 20 km by road south-east of the town of Ravensthorpe and can be accessed from the Hopetoun-Ravensthorpe Road. Regional features include the Bandalup Corridor, the buffer zone for the Fitzgerald Biosphere Reserve and areas of uncleared vacant Crown Land as well as private property that supports agricultural land uses.

2.3 Standards, Guidelines and Regulations

The FS level design of the RGP TSF shall follow the recommendations of the following;

- Government of Western Australia Department of Mines and Petroleum (DMP): Guide to Departmental requirements for the management and closure of tailings storage facilities (TSFs), 2015a
- Government of Western Australia Department of Mines and Petroleum (DMP) Code of Practice (CoP): Tailings Storage Facilities in Western Australia, 2013
- Australian National Committee on Large Dams (ANCOLD): Guidelines on Tailings Dams Planning, Design, Construction, Operation and Closure, 2012.

2.4 Storage Capacity

The RGP ore processing route is gravity/flotation/CIL; design slurry density of CIL (Carbon-in-leach) is 50 % solids w/w (no tailings thickener). After the final adsorption tank the slurry will pass through a detox tank before being pumped to the TSF at 50 % solids w/w.

Base case tailings production of 3.1 Mt has been adopted for this study. At an assumed average dry density of 1.5 t/m^3 for the stored tailings, the required tailings storage capacity is 2.0 Mm^3. Annual production rates may vary from 0.3 Mtpa to 0.4 Mtpa however the required tailings storage capacity will remain constant.

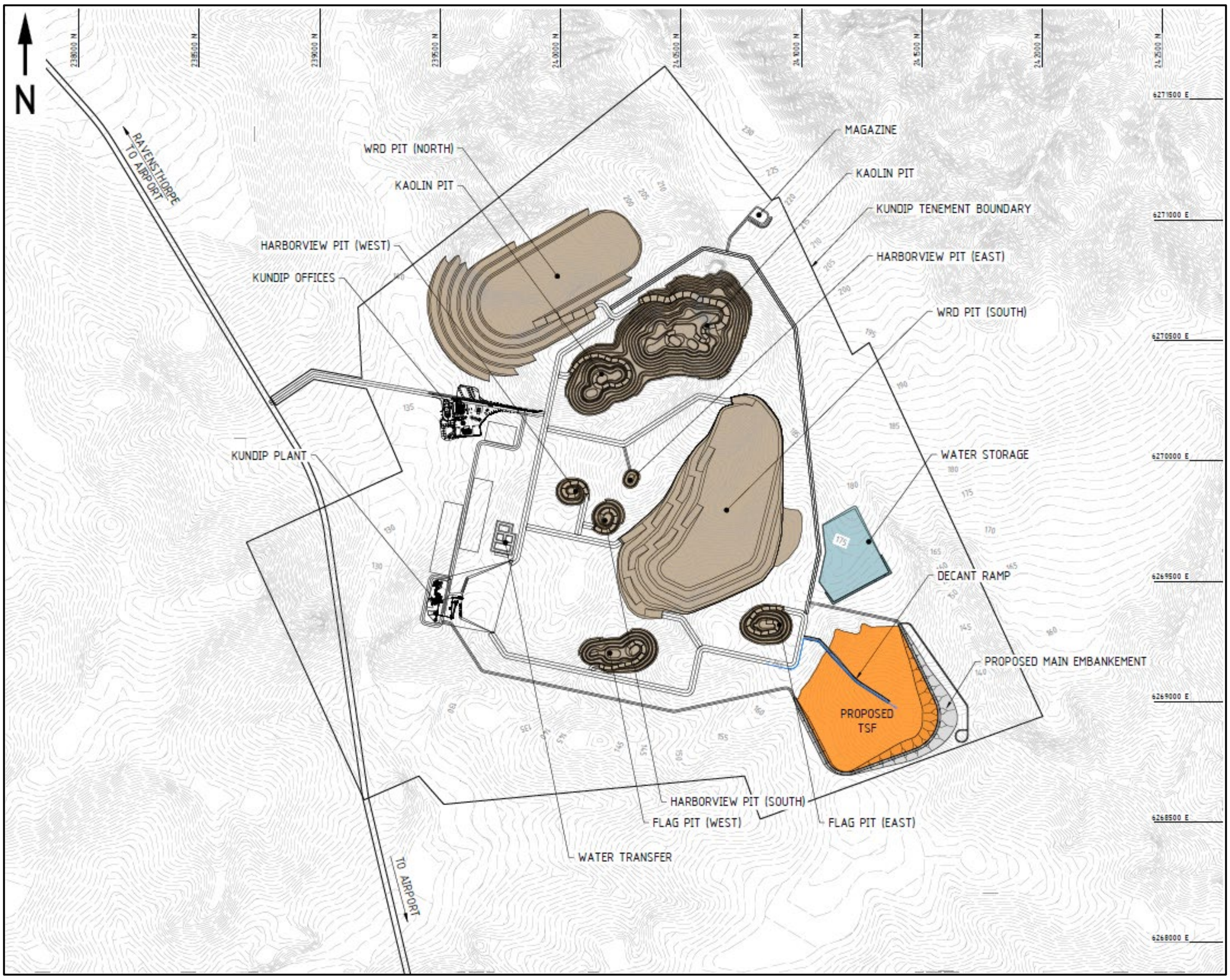
Should additional storage capacity be required, either from increased production or extending the life of the project, the TSF could be raised. However, design of a future raise is outside the scope of this document.

2.5 Tenure and Site Conditions

2.5.1 Location

The Kundip Mine Site is situated approximately 20 km by road south-east of the town of Ravensthorpe and can be accessed from the Hopetoun-Ravensthorpe Road. The RGP site layout, Kundip mining tenements and location of the TSF relative the main project features is shown on Figure 1.

Figure 1: RGP Site layout



2.5.2 Climate

Data from the Bureau of Meteorology (BOM) weather station nearest to the TSF will be used to evaluate the climate of the project area (BoM, 2017). Presented in Table 1 are the long-term temperature and rainfall data (1901-2017) for Ravensthorpe (BoM Site 010633).

Mean monthly rainfall at Ravensthorpe ranges from 24.2 mm in December to 47.3 mm in July, with a mean annual rainfall of 429.5 mm.

Mean daily evaporation at Munglinup Melaleuca (BoM site 012281), approximately 60km from the Kundip mine site, ranges from 2.5 mm in July to 8.3 mm in January; mean annual daily evaporation of 5.0 mm (1,825 mm annual).

Table 1: Long-term Rainfall & Temperature and Evaporation Data - Ravensthorpe 010633.

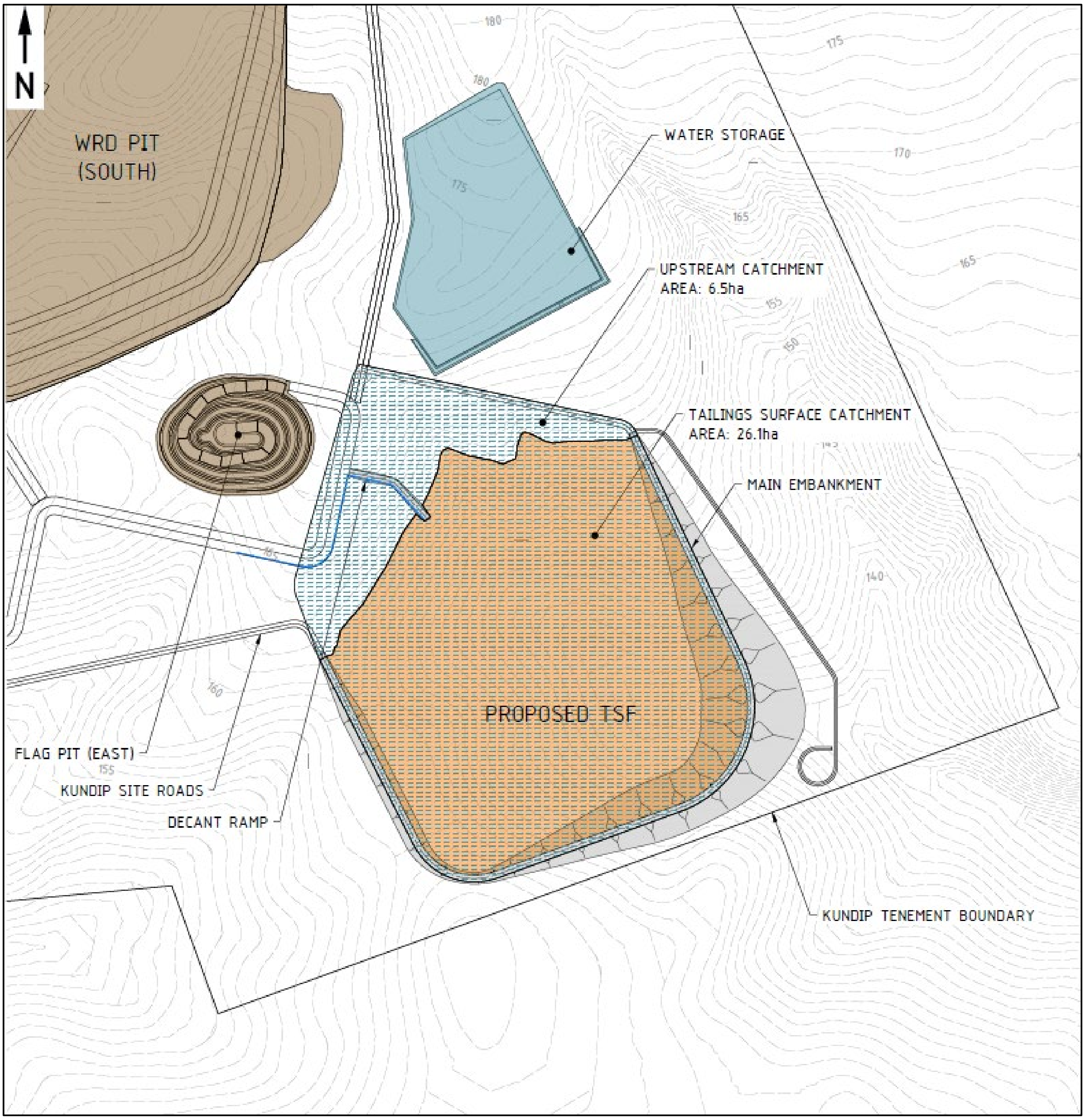
Parameter	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sept	Oct	Nov	Dec	Annual
Mean Max Temp (ºC)*	29.0	28.3	26.6	23.7	20.0	17.3	16.3	17.3	19.5	22.5	25.1	27.2	22.7
Mean Minimum Temp (ºC)*	14.1	14.6	13.6	11.8	9.6	7.9	6.7	6.7	7.4	9.1	11.1	12.8	10.4
Mean Rainfall (mm)**	24.9	26.5	32.8	32.8	44.1	43.6	47.3	45.1	42.3	38.0	30.6	24.2	429.5 (total)
Highest Rainfall (mm)**	223.2	249.2	163.0	144.7	127.0	117.9	129.6	136.6	144.8	121.4	189.4	140.1	734.5 (total)
Mean Daily Evaporation (mm)***	8.3	7.7	6.3	4.7	3.2	2.6	2.5	2.9	3.7	4.8	6.0	7.7	1825 (total)
* 1962-2017; **1901-2017; ***1975-2001 Munglinup Melaleuca (BoM site 012281)													

2.5.3 Hydrology

2.5.3.1 Catchment

The proposed location of the RGP TSF has been chosen to limit the upstream catchment which would report to the TSF. The final TSF disturbance footprint is approximately 29.3 ha (main embankment and maximum tailings extent at year 10). The total tailings surface catchment is 26.1 ha with an upstream catchment of 6.5 ha for a total catchment of 32.6 ha as shown on Figure 2.

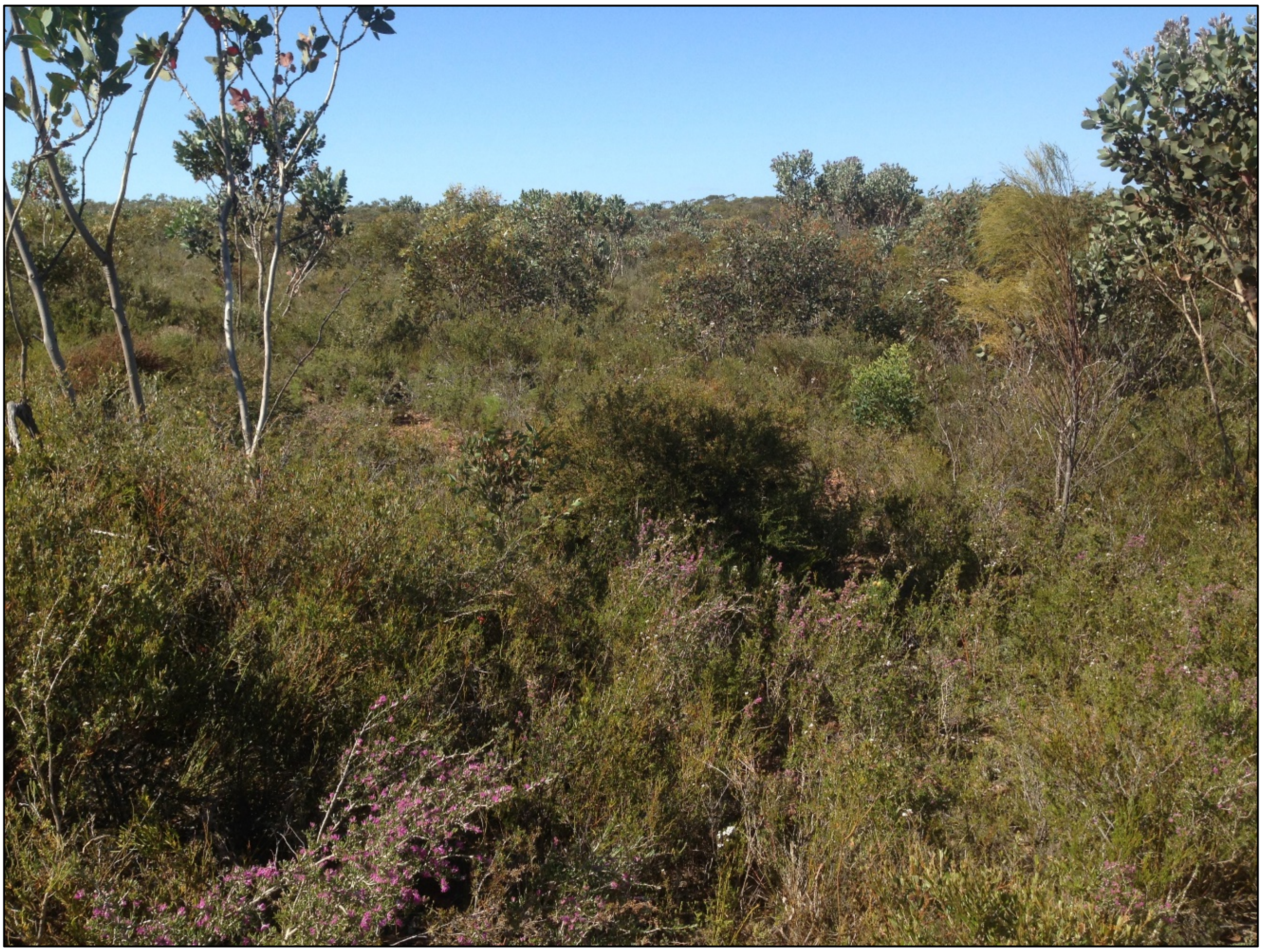
Figure 2: TSF Catchment. (TSF catchment and upstream catchment areas)



2.5.3.2 Runoff

The catchment upstream of the proposed TSF location is densely vegetated as shown on Figure 3.

Figure 3: Proposed TSF location - upstream catchment.



An appropriate rational method runoff coefficient for heavily vegetated areas with loamy soil such as those observed for the TSF catchment would be in the range of C = 0.05 to 0.25. Data gathered from the Australian Rainfall & Runoff Data Hub (accessed 6 November 2017) for the proposed TSF location indicate storm initial losses and continuing losses at 28.0 mm and 1.5 mm/hr respectively; roughly equating to a rational method runoff coefficient of C = 0.21 for a 1:100-yr 72-hr event. A conservative runoff coefficient of C = 1.0 and C = 0.25 for the tailings surface (18.9 ha) and upstream catchment (18.6 ha) respectively has been adopted.

A runoff coefficient of C = 1.0 for the upstream catchment was checked for sensitivity in the TSF storm water storage (freeboard) calculation in Section 3.4.4.

2.5.3.3 Design Storm Events

Design rainfall depths (mm) for the project site obtained from the BoM 2016 Rainfall IFD (Intensity Frequency Duration) Data System are shown on Table 2. The design storm storage requirement under DMP (2015a) and ANCOLD (2012) guidelines is for a 1:100 year 72-hour duration rainfall event (highlighted) in Table 2. DMP and ANCOLD design storm storage requirements are discussed further in Section 3.2.3 & 3.3.6 respectively.

Table 2: Rare design rainfall depth (mm) – (BoM 2016 Rainfall IFD data system)

	Annual Exceedance Probability (1 in x)				
Duration	**1 in 100**	**1 in 200**	**1 in 500**	**1 in 1000**	**1 in 2000**
24-hour	136	159	193	222	255
48-hour	162	189	229	263	301
72-hour	172	200	243	279	319
96-hour	177	206	249	286	327
120-hour	178	209	252	290	331
144-hour	179	211	254	292	334
168-hour	179	212	256	293	335

2.5.4 Geology

2.5.4.1 Regional Geology

There are three regional geological units in the area:

- Yilgarn Craton (Archaean) to the north comprising granitoid, granitic gneiss and migmatitic rocks with some greenstone rafts, overlain to the south by;
- Mount Barren Group (Proterozoic) comprising metasedimentary rocks of shale, arenite, dolostone and intruded gabbro-diorite sills; and
- The southeast portion of the region is occupied by Munglinup Gneiss (Proterozoic), which forms part of the Biranup Complex.

The northeast trending Jerdacuttup Fault separates the Munglinup Gneiss from both the Mount Barren Group and the Archaean granite-greenstone terrane. Tertiary sediments of the Plantagenet Group in turn unconformably overlie all Precambrian tectonic units.

2.5.4.2 Local Geology

The Kundip mining area lies in a region of steeply-dipping mafic to intermediate volcanic rocks of Archaean age (Annabelle Volcanics) (Witt, 1997). The volcanic rocks have been intruded to the west by granitic rocks, also of Archaean age. The upper reaches of the Steere River follow the contact between the granitic and the volcanic rocks.

Immediately south of the Kundip mining area, the Archaean rocks are overlain by the Proterozoic Mount Barren Group, including sediments of the Kundip Quartzite and the Kybulup Schist. The quartzite dips at about 15 degrees to the south-south-west.

2.5.5 Sub-surface Conditions and Foundations

A geotechnical site investigation was carried out by MHA November 17th to 23rd 2017 (Appendix A). The purpose of the geotechnical investigation was to:

- Develop ground profiles for the TSF location,
- Determine the geotechnical properties for foundation and borrow materials,
- Provide comment on the suitability of the site for the proposed development.

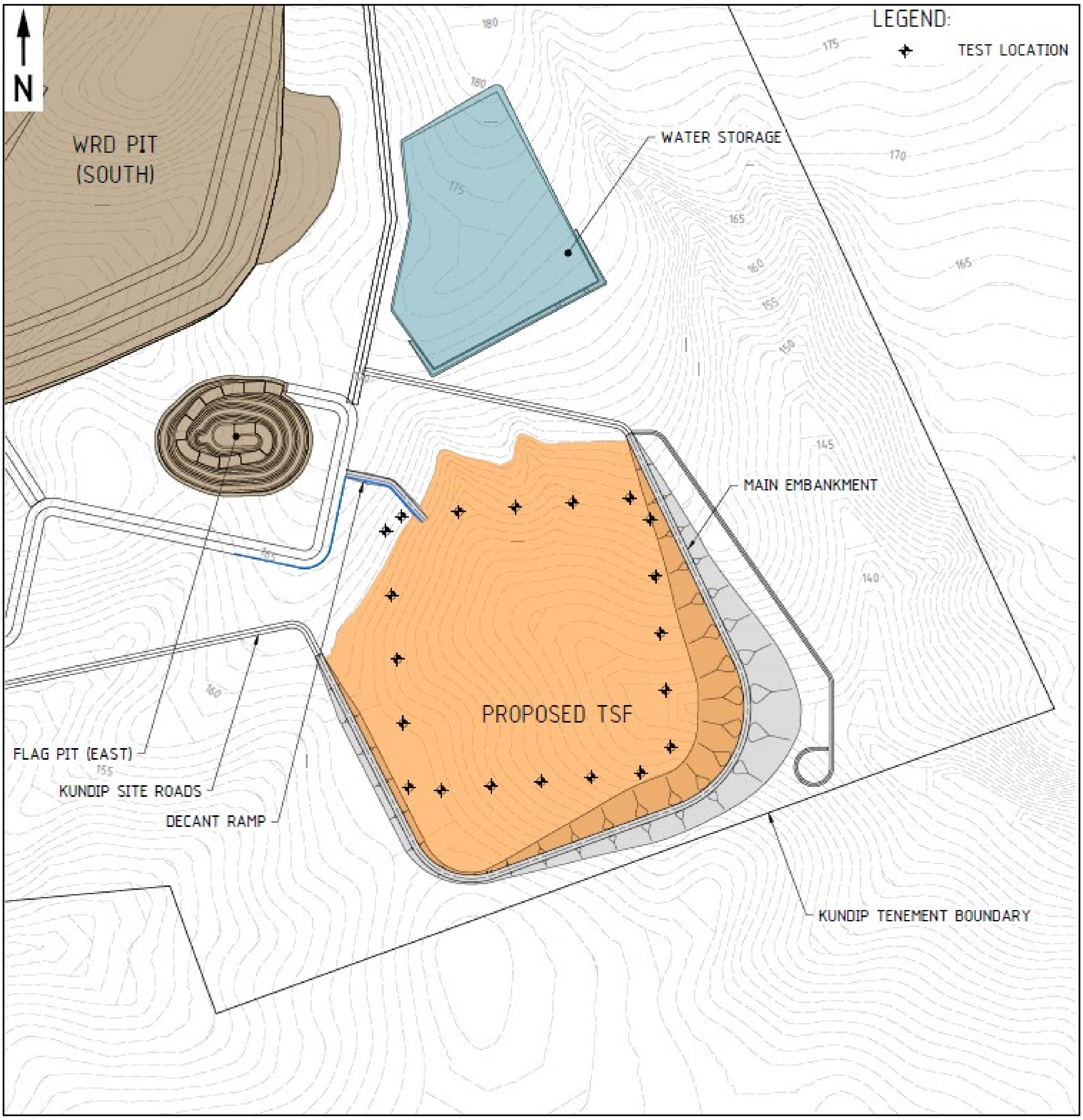
The typical regolith profile at the TSF site comprises a surficial cover of an unconsolidated sandy silt TOPSOIL underlain by sandy gravelly SILT, underlain by SILTSTONE (see Appendix B – Geotechnical Field Investigation Test Pit Logs and Photographs).

The material encountered can be broadly summarised as:

- 0 m – 0.2 m: SILT; sandy, gravelly TOPSOIL with roots and organic matter;
- 0.2 m – 0.6 m: SILT; red brown, sandy with gravel (transitional zone);
- 0.6 m – 1.0 m: SILTSTONE; red brown, conglomeritic;
- 1.0 m – 3.0 m: SILTSTONE, white sandy/gravelly (considered competent bedrock).

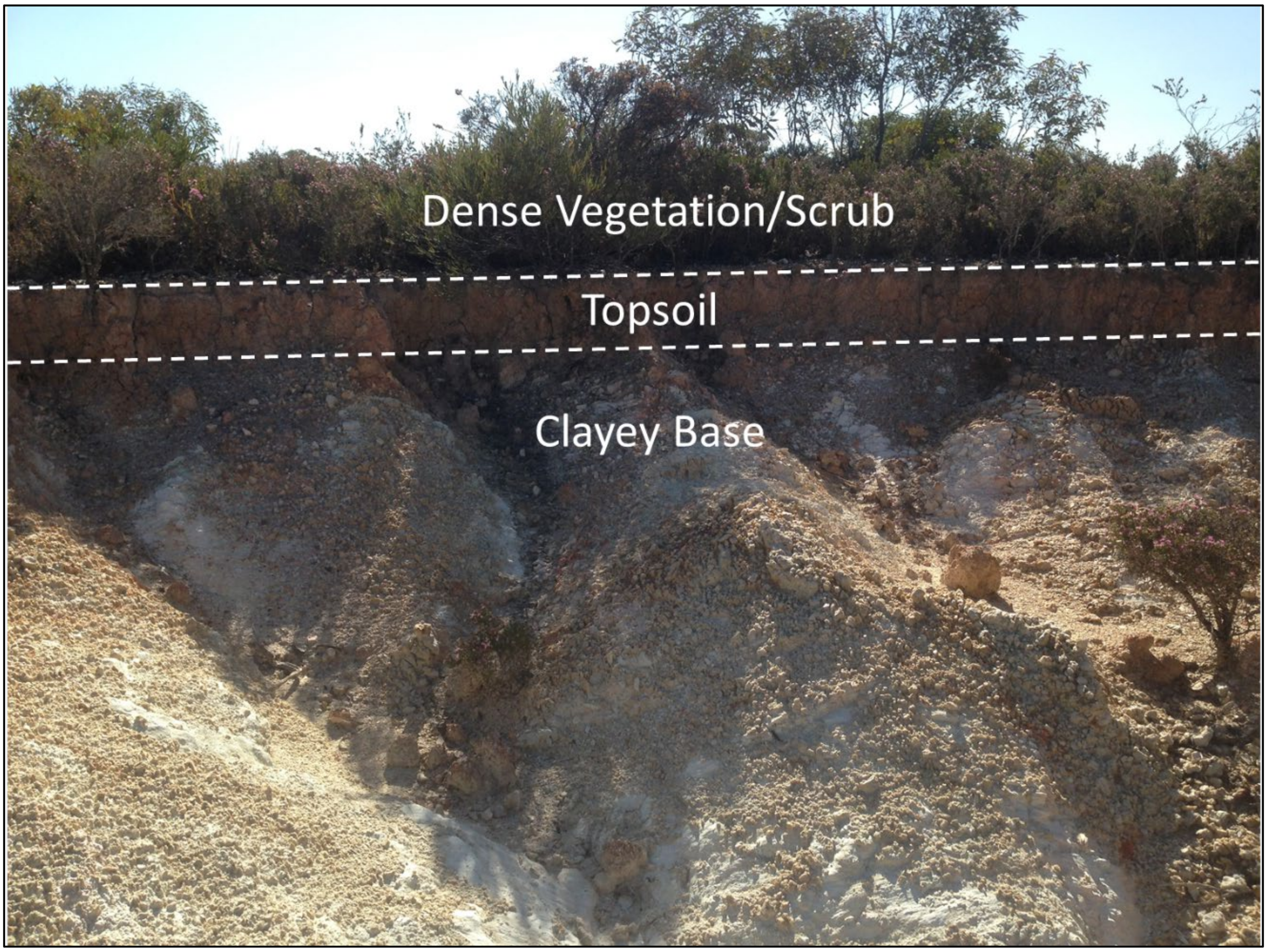
Geotechnical test locations relative to the proposed TSF configuration are shown on Figure 4.

Figure 4: Geotechnical Investigation Setout



Additionally, during a site visit undertaken on 19 October 2017, a portion of the TSF basin was accessible from an existing access track. In an area disturbed by previous prospecting activities a glimpse into the shallow subsurface conditions was gained by viewing the disturbed areas within the TSF basin. The exposed profile captured in Figure 5 shows dense vegetation/scrub underlain by 300 to 500 mm of topsoil with a clayey base below.

Figure 5: TSF basin - shallow subsurface conditions



The geotechnical investigation undertaken by MHA further confirmed the general shallow subsurface stratigraphy. The foundation directly beneath the proposed TSF main embankment was not accessible during the site visit. At this stage it is assumed that the subsurface conditions beneath the main embankment are similar to the test locations immediately upstream of the embankment. Assumed geotechnical parameters for the embankment foundation are presented in Section 3.4.1.3.

2.5.6 Seismic Risk

The seismic hazard risk assessment contained in GA (2012) is used to quantify the seismic setting for the site. This is a relatively recent and detailed assessment and provides peak ground accelerations (PGAs) for earthquakes of return period 500 years and greater (c.f. the project Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) return periods of 50 years and 100 years respectively). As such its use is conservative but it directly relates to PGAs of interest to the design of earth structures as opposed to use of AS1170.4 Structural design actions – Earthquake actions in Australia that is strictly only applicable to steel, concrete and timber structures.

The PGA is estimated to be 0.06g for the project. Mining induced ground motion, such as blast induced shaking, is expected to result in relatively minor PGA and for very short durations (cycles). A blast risk assessment will be covered as part of the detailed design process if required.

2.5.7 Current and After Closure Land Use

Post mining land use options and closure objectives have been broadly identified at the project planning phase and will be further defined during the stakeholder consultation process. The identified post mining land use aim is to return the land to the pre-mining land use of native vegetation at Kundip.

2.6 Retaining Structure Properties

The geotechnical investigation undertaken by MHA (Appendix A) included collection of samples from stockpiles located on the Kundip site as well as samples taken from the TSF basin. The results of the laboratory test work and the geotechnical properties of retaining structure are presented in Appendix E. The geotechnical properties of the proposed embankment construction material sources are presented in Section 3.4.1.2.

2.7 Tailings Properties

2.7.1 General

At the time of writing this report, representative tailings samples were not available for laboratory test work. Assumed engineering design parameters are based on our experience with similar tailings projects. This is considered acceptable for FS level design, particularly so because the proposed design does not rely on the geotechnical properties of the tailings for stability or containment, as would be the case with an upstream raised embankment configuration.

2.7.2 Tailings Design Parameters – Civil Infrastructure and Planning

The RGP TSF embankment will provide tailings storage capacity for the currently projected life of asset tailings production, as set out in Section 2.4. The embankment does not rely on the strength of the tailings for stability and no future raises are currently planned. Tailings samples for geotechnical test work were not available at the time of preparing this report. Assumed parameters for FS level design of the TSF are shown on Table 3.

Table 3: Assumed tailings design parameters

Parameter	Value
In situ dry density	1.5 t/m^3
Shear Strength for slope stability assessment	zero
Hydraulic permeability	1x10^{-3} to 1x10^{-7}
Slurry density	50 % (w/w)

2.7.3 Tailings Design Parameters – Mechanical Infrastructure

Tailings samples for rheological test work were not available at the time of preparing this report. Furthermore, design of mechanical infrastructure is not within the scope of this study.

2.7.4 Geochemical Characterisation of Tailings

A preliminary review of the Phillips River Project: Geochemical Characterisation of Tailings-Slurry Samples (Trilogy Deposit) - Implications for Process-Tailings Management (GCA, 2011) indicates the “Cu/Au-Tailings” are potentially acid forming (PAF). It is understood that additional geochemical test work and characterisation will be undertaken prior to or as part of the detailed design process.

3. TSF Design

3.1 Introduction

The proposed RGP TSF is a side-hill paddock-style facility. An engineered embankment will provide containment on three sides (east, south and west) whilst the natural topography will provide containment to the north. The proposed TSF configuration is shown in plan and profile on

Figure 4 and Figure 5. FS level design drawings are included in Appendix F.

Figure 6: TSF General arrangement (plan)

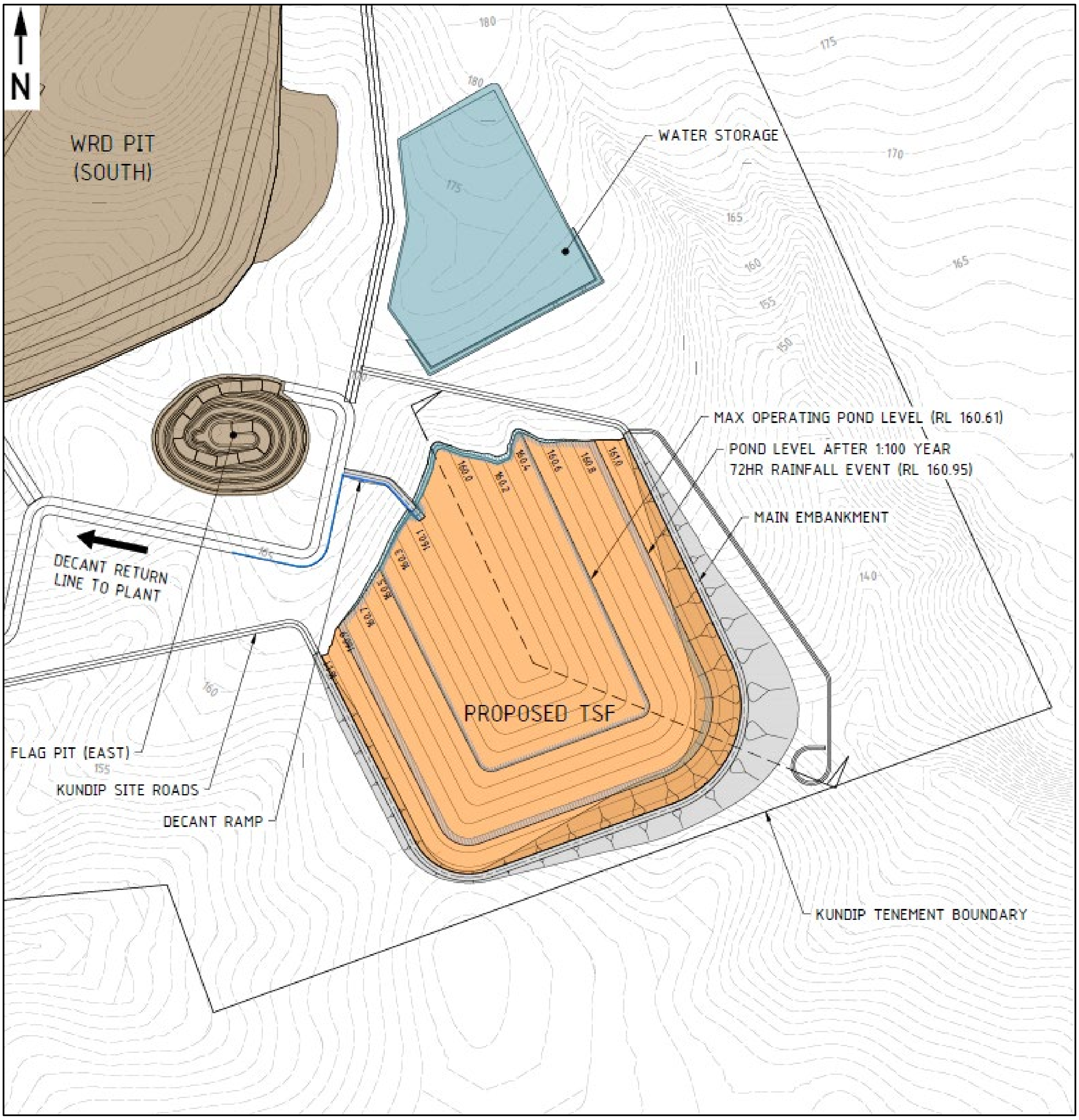
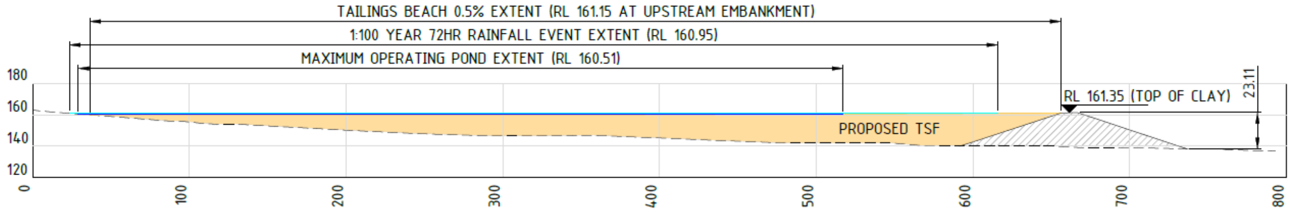


Figure 7: TSF General arrangement (profile)



3.2 DMIRS Classification

3.2.1 Hazard Rating

In accordance with the DMP CoP (DMP, 2013), the RGP TSF attracts a Medium hazard rating as demonstrated in Table 4.

Table 4: Hazard rating system applicable to TSFs in Western Australia

Type of impact or damage	Hazard rating	Extent or severity of impact or damage
		Embankment or Structural Failure Controlled or uncontrolled release of tailings/water, or seepage
Loss of human life or personal injury	Low	For the proposed location of the TSF the potential population at risk (ANCOLD terminology) is <1.
Adverse human health due to direct physical impact or contamination of the environment	Low	For the proposed location of the TSF there is no potential for human exposure due to direct physical impact. Potential human exposure due to contamination of the environment is low, but the possibility is acknowledged.
Loss of assets due to direct physical impact or contamination of the environment	Low	Livestock will not be present locally, hence there is no potential for loss of livestock from failure. The impact to stock water supply downstream is acknowledged but considered to be minimal; nearest farm is approximately 9km south.
	Low	There are no infrastructure or other mining, public or pastoral assets immediately downstream of the TSF.
	Medium	Loss of TSF storage capacity is possible and repair is practicable.
Damage to items of environmental, heritage or historical value due to direct physical impact or contamination of the environment	Medium	*The Kundip leases are surrounded by an area of the Ravensthorpe Range recommended by the EPA Red Book (Recommendation 3.8) to become a nature reserve (Proposed Nature Reserve 56).* *Kundip lies within the eastern sector of the Fitzgerald Biosphere Reserve, in the zone of cooperation. The Biosphere Reserve is a part-tenured management concept recognised by UNESCO as well as State and Commonwealth governments. The concept includes a core area (the Fitzgerald River National Park) a buffer zone (Crown land and some unvested reserves) and a zone of cooperation (private freehold farmland including 557,000ha cleared and 160,000ha uncleared). Mining, subject to sound environmental management practices, is one of many human impacts considered to be acceptable in the zone of cooperation. Kundip is outside of all defined zones.* (Tectonic, 2011). The Kundip Mine Site is in close proximity to areas of significant environmental value (nature reserve). Temporary damage to the natural environment is possible.
	Medium	Temporary adverse effects on flora and fauna are possible
	Low	Limited or no potential for damage of items of heritage or historical value

3.2.2 TSF Category

In accordance with the DMP Code of Practice (DMP, 2013), the RGP TSF would be classified as a **Category 1** facility as the TSF attracts a hazard rating of **Medium** and the embankment will be greater than 15 m in height.

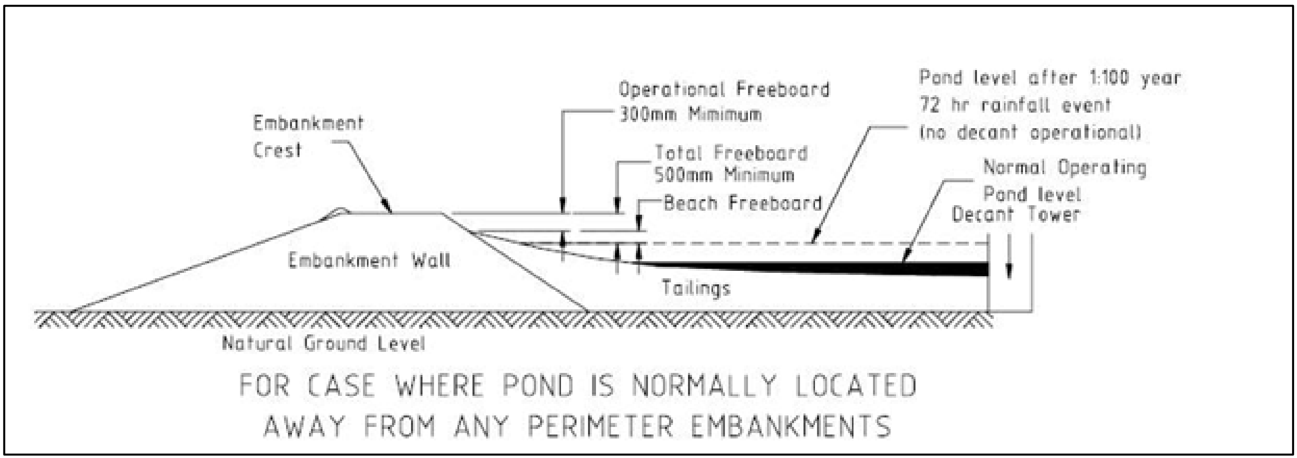
3.2.3 DMP Recommended Freeboard (DMP, 2015a)

Total Freeboard is defined as the vertical height between the lowest point on the crest of the perimeter embankment of the TSF and the normal operating pond level plus an allowance for an inflow corresponding to the 1:100 year 72-hour duration rainfall event falling in the catchment of the pond, assuming that no uncontrolled discharge takes place for the duration of the rainfall event (Total Freeboard also corresponds to the sum of the “Operational Freeboard” and the “Beach Freeboard” as shown on Figure 8).

Operational Freeboard is defined as the vertical height between the lowest elevation of the perimeter embankment and the tailings beach immediately inside the embankment. The operational freeboard varies over the course of a deposition cycle as the storage is raised and fills with tailings. The operational freeboard becomes critically important at the end of a deposition cycle, particularly to minimise the potential for back flow and overtopping as a result of mounding of tailings at discharge points.

Beach Freeboard is defined as the vertical height between the normal operating pond level plus an allowance for an inflow corresponding to the 1:100 year 72-hour duration rainfall event falling in the catchment of the pond, assuming that no uncontrolled discharge takes place for the duration of the rainfall event, and the point on the beach where the wall freeboard is measured. The Beach Freeboard can vary significantly during the life of the storage and depends upon beach length, slurry/tailings characteristics, deposition methodology etc. Beach Freeboard is not applicable where the pond is normally located against a perimeter embankment.

Figure 8: Freeboard definition (DMP, 2015a)



3.3 ANCOLD Consequence Category

3.3.1 General

There are two Consequence Categories that need to be assessed as part of Tailings Dam design. These are the Dam Failure Consequence Category and the Environmental Spill Consequence Category. These are used to determine various design and operational requirements including design of spillways and for flood storage requirements.

3.3.2 Dam Failure Severity Level

In accordance with ANCOLD (2012) Guidelines there are seven (7) damage type categories (infrastructure, business importance, public health, social dislocation, impact area, impact duration and impact on natural environment) that need to be assessed in order to determine the severity level/impact (Minor, Medium, Major and Catastrophic) of a potential facility failure or spill.

The severity levels of impacts associated with failure of the RGP TSF embankment are:

- Infrastructure – Minor: less than $10M production losses and repair costs;
- Business importance – Medium: significant impacts to operations, including reduced or suspended operations whilst repairs are made;
- Public health – Minor: no person's health is affected (see Table 4);
- Social dislocation – Minor: no persons impacted;
- Impact area – Medium: potential impact area greater than 1 km^2 but less than 5km^2;
- Impact duration – Minor: less than 1 year;
- Impact on natural environment – Medium: (see Table 4).

3.3.3 Dam Failure Population at Risk

The population at risk (PAR) is defined as all people who would be directly exposed to floodwaters assuming they took no action to evacuate. No homes, businesses, recreational areas, offices, workshops or laydowns are located downstream of the embankment, and operational personnel would not be present in low lying areas downstream of the embankment. Based on this, the PAR for the TSF is considered to be 0 (ANCOLD PAR category of <1).

3.3.4 Dam failure Consequence Category

Based on a dam failure severity level of '**Medium**' and a PAR <1, the ANCOLD guidelines recommend adoption of a '**Low**' Dam Failure Consequence Category rating for purpose of design.

3.3.5 Environmental Spill Consequence Category

The Environmental Spill Consequence Category is assessed by considering the effect of spilling dam water to the downstream environment (typically through the dam spillway during a flood event). The aerial extent of the spill impact will be significantly smaller than the area which would be affected in the event of dam failure.

The effect of spilling dam water to the environment is primarily driven by the geochemistry of the tailings solids and supernatant; see Section 2.7.4. Water spilled from the dam under extreme weather events, will be significantly diluted, and further diluted again given the downstream environment of the dam is also likely to be flooded.

Therefore, the severity of impact on the natural environmental from environmental spills through a TSF spillway would be 'Minor'.

The PAR assigned to a dam spill is <1.

The combined Dam Spill Consequence Category is assessed as 'Very Low' at this stage of the design.

3.3.6 ANCOLD Design Criteria

ANCOLD recommended design criteria for a 'Low' consequence category facility have been adopted, including;

Minimum freeboard comprising:

- 1:100 annual exceedance probability (AEP), 72-hr flood;
- Contingency freeboard – nil;

- Additional freeboard – nil.

Earthquake loadings:

- Operating Basis Earthquake (OBE) – 1:50 AEP; and
- Maximum Design Earthquake (MDE) – 1:100 AEP.

3.4 Modelling and Design Studies

3.4.1 Stability Assessment

3.4.1.1 Embankment Compaction

The maximum height of the tailings dam embankment is 21.8 m and would be classified as a large height dam embankment. On this basis the compaction criterion for embankment materials is based on the Modified Compaction test method, with a minimum required dry density of 95 % MMDD (Maximum Modified Dry Density).

3.4.1.2 Embankment Material

Based on the results of the laboratory test work, the embankment material is likely to comprise low plasticity clay with silt, sand and gravel, and is likely to encompass the following material types in Table 5 under the Unified Soil Classification System (USCS). Table 5 presents expected values for maximum unit wet density, effective stress cohesion and friction angle for these materials after Hunt (1986). Design density and strength values adopted for the embankment material are also presented.

Table 5: Embankment Material Geotechnical Parameters

USCS	Description	Maximum Wet Density (σ)	Saturated Effective Cohesion (c')	Effective Stress Friction Angle (ϕ')
		kN/m^3	kPa	degrees
SM-SC	Sand-silt clay mix with slightly plastic fines	19.9 – 22.7	14	33
SC	Clayey sand, poorly graded sand-clay mix	19.6 – 21.8	11	31
ML	Inorganic silts and clayey silts	18.5 – 21.2	9	32
ML-CL	Mixture of inorganic silt and clay	19.2 – 21.1	22	32
CL	Inorganic clays of low to medium plasticity	18.5 – 21.1	13	28
DESIGN	Embankment Material	21.0	10	30

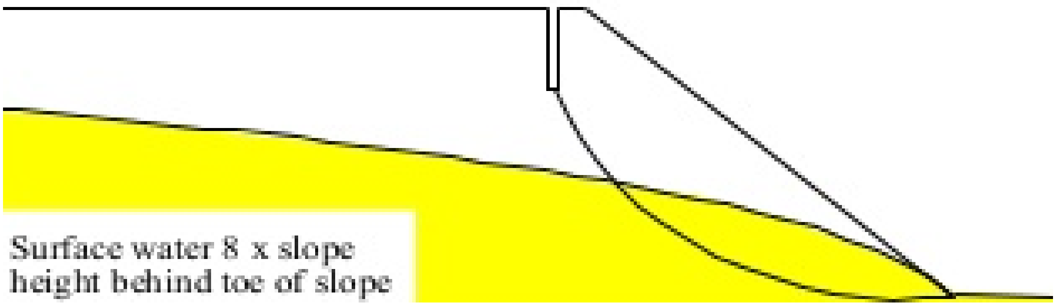
3.4.1.3 Foundation Material

The foundation material is likely to comprise pallid clayey soil with an expected minimum undrained shear strength of 100 kPa. This affords a suitable founding material for the proposed 21.8 m high embankment (applied bearing pressure of about 400 kPa, maximum, and 250 kPa, average).

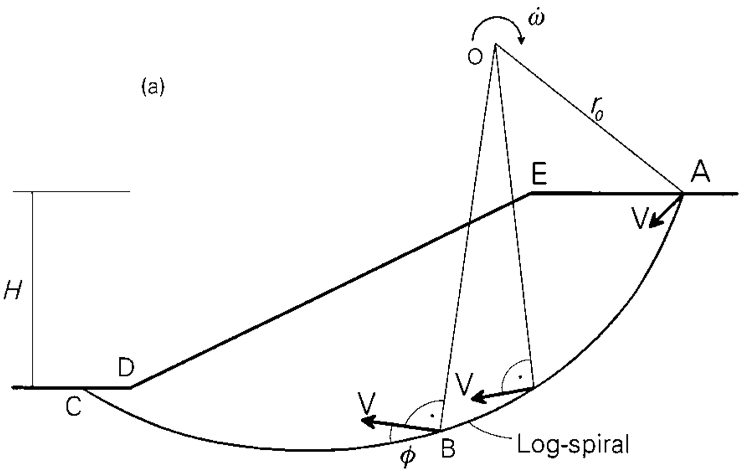
3.4.1.4 Slope Stability Assessment Methodology

Slope stability assessment was undertaken assuming a uniform slope of 1V:3H upstream and downstream batters. The target static stability factor of safety (FoS) is 1.50, and the maximum allowable degree of saturation in the slope to achieve this was assessed.

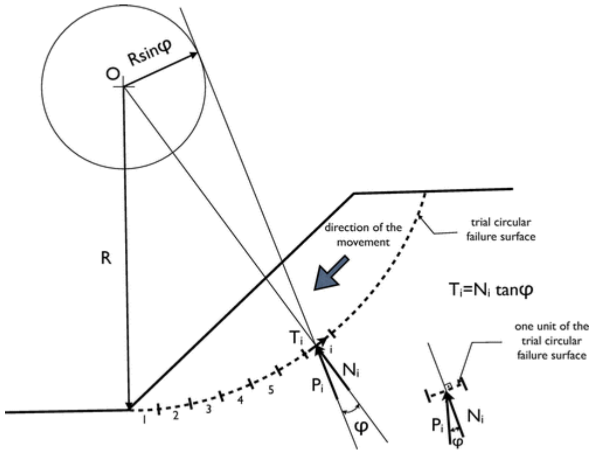
The following analysis techniques were used:



- Hoek & Bray (1981) – chart solution for circular failure slip with upstream tension crack



- Michalowski (2002) – chart solution for log-spiral failure slip



- Cousins (1978)– chart solution for circular failure slip, presented in Hunt (1986)

The embankment material is unlikely to be susceptible to seismic liquefaction, given its high fines content and well-compacted state. Seismic stability was assessed by considering

- What percent reduction in soil strength was required in order to achieve a post seismic FoS of unity;
- What coefficients of horizontal (kh) and vertical (kv=+/-0.5kh) acceleration were required to achieve a FoS of unity.

3.4.1.5 Results

3.4.1.5.1 Static Stability

Results of static stability analyses are presented in Table 7 for target factor of safety (FoS) value of 1.50. These results indicate adequate stability even for the case of a part-saturated embankment.

Table 6: Static Stability Results

Analysis Method	Static FoS	Embankment Percentage Saturation
Hoek & Bray (1981)	1.50	50 %
Michalowski (2002)	1.50	90 %
Cousins (1978)	1.50	60 %

3.4.1.5.2 Seismic Stability

Results of seismic stability analyses are presented in Table 8 for a target factor of safety (FoS) value of unity.

The strength reduction results point to a robust embankment even if marked strength reduction occurs post seismic shaking.

Simplistic pseudo-static assessment using kh and kv indicate adequate seismic stability. The peak ground acceleration (PGA) for the site is <0.06g for the Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) events. The kh and kv for FoS of unity are 0.10g and +/- 0.055g respectively.

Table 8: Seismic Stability Results

Assessment Method	Seismic FoS	Result
Strength Reduction	1.0	30 % reduction in c' and ϕ'
	1.0	100 % reduction in c' No reduction in ϕ'
Lateral Acceleration	1.0	k_h = 0.10g k_v = +/- 0.5k_h

3.4.2 Erosion Control

The proposed TSF embankment configuration incorporates 1V:3H upstream and downstream batters to help manage batter erosion. The shallow downstream batter will serve as both an operational and final closure slope, envisaged to be vegetated shortly after construction in order to mitigate batter erosion. A shallow upstream batter has been adopted for the upstream batter due to the length of time the batter will be exposed prior to being covered with tailings.

The embankment will be constructed of non-dispersive material and includes a protective wood chip/mulch sheeting layer for further protection of the batter from erosion.

3.4.3 Seepage

3.4.3.1 General

Design measures and operational controls aimed at minimising seepage include;

- Design measures
    - Small TSF catchment;
    - Location of the decant pond;
    - Low rate of rise;
    - Low permeability floor.

- Operational controls
  - Sub-areal deposition to promote air-drying (evaporation) whilst continually depositing in thin lifts to minimise dust generation;
  - Maintaining a small decant pond away from the embankment against natural topography;
  - Monitoring of pore pressure development within and downstream of the main embankment;
  - Monitoring of groundwater levels and groundwater quality downstream of the main embankment.

3.4.3.2 Design Measures

Small TSF Catchment

The location of the proposed TSF has been optimised to provide the required storage capacity whilst minimising the catchment runoff that reports to the facility i.e. seepage is minimised by minimising TSF inflows, see Section 2.5.3.1.

Location of the Decant Pond

The TSF is designed such that tailings will be discharged from the embankment and beaching towards the natural topography. This will facilitate the decant pond being located substantially away from the embankment, reducing the potential for phreatic conditions (pore pressures) from developing beneath and with the main embankment. Decant pond development and location are described in Section 3.6.2.

Low Rate of Rise

The TSF will benefit from a low rate of rise (RoR) of <2 m/yr (year 3 to year 10) which will allow for deposition of tailings in thin lifts. Sub-areal deposition in thin lifts will promote consolidation through air-drying resulting in a reduced permeability of the deposited tailings and thus reduced seepage potential (compared to other deposition strategies such as sub-aqueous deposition or deposition in thick lifts i.e. high RoR). The RoR is shown graphically on Figure 12.

Low Permeability Floor

The in-situ TSF floor material is assumed to be of low permeability based on preliminary field observations during the site visit and geotechnical investigation undertaken by MHA, see Appendix C and D. Further test work will be undertaken in the main embankment footprint to confirm this assumption is valid throughout the TSF. In the event that areas of the TSF floor are found to be more permeable than expected (>1x10-9), clay borrow material sourced from the Kundip mine site may be used to construct a compacted clay liner.

3.4.3.3 Operational Controls

Sub-areal Deposition

As discussed above, sub-areal deposition in thin lifts will serve to increase evaporative losses (reducing water available for seepage) and decrease permeability of the deposited tailings.

Decant Pond Management

Maintaining a small decant pond away from the embankment will reduce (if not eliminate) the potential for embankment seepage. Furthermore, a small decant pond both in depth and areal extent against natural topography will minimise hydraulic head driving seepage.

Pore Pressure Monitoring

Pore pressure development within and downstream of the main embankment will be monitored via vibrating wire piezometers (VWPs) as shown in Section 4.5.

Groundwater Monitoring

A groundwater monitoring bore and VWP will be installed downstream of the main embankment to monitor groundwater levels and groundwater quality (against background groundwater quality) downstream of the main embankment as shown in Section 4.5.

3.4.3.4 Seepage Quality

Seepage quality and background groundwater quality in the area of the proposed TSF location has yet to be quantified. At this stage in the design development process the primary seepage management strategy is to limit the amount of seepage.

3.4.4 Surface Water Flow and Storage

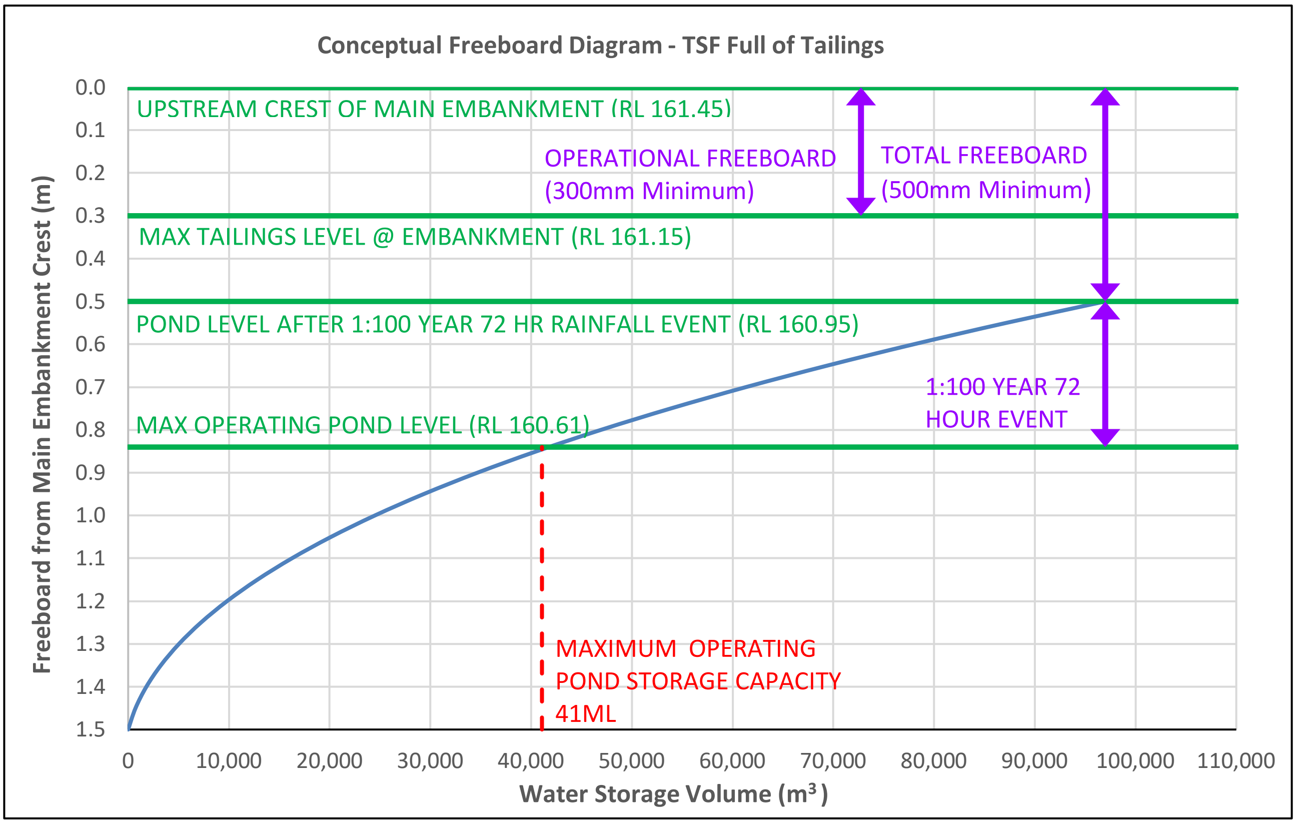
Assessment of freeboard has been conducted taking into consideration the *ANCOLD Guidelines on Tailings Dams – Planning, Design, Construction, Operation and Closure (ANCOLD, 2012)* and the Code of Practice (CoP): Tailings Storage Facilities in Western Australia (DMP, 2013). The TSF catchment is shown on Figure 2.

The freeboard was assessed as shown on Figure 9 (top down approach); the figure shows that based on a maximum operating pond level of RL 160.61 m, there is sufficient freeboard to contain a 1:100 AEP 72-hour storm event whilst maintaining 500 mm total freeboard. A very conservative runoff coefficient (C=1.0) was adopted for the entire catchment to demonstrate the robustness of the TSF storm water storage capacity.

The storm water storage capacity is dependent upon the actual beach slope achieved during operation. The volume estimate presented in Figure 9 is based on a 0.5 % beach slope.

It should be noted that the maximum operating pond level (RL 160.61 m) could be a combination of small storm events prior to a 1:100 AEP 72-hour storm event; i.e. the maximum operating pond level at a dam full (tailings) scenario should not be viewed as a maximum operating level under normal circumstances. The freeboard assessment should be revisited prior to reaching dam full of tailings to assess if the above assumptions around beach slope are correct.

Figure 9: TSF Freeboard Assessment



3.5 Design and Construction Details

3.5.1 General

Construction of the RGP TSF will be undertaken in accordance with issued for construction (IFC) drawings and earthworks specification. Furthermore, construction and operation will be in general accordance with the design intent of the final detailed design report.

This report and the drawings included in Appendix F present a FS level design of the RGP TSF which may serve as the basis for subsequent development of a detailed design report and IFC drawings.

3.5.2 Bill of Quantities

A preliminary earthwork bill of quantities (BOQ) is provided in Table 9. A more detailed BOQ will be developed during detailed design based on issued for construction (IFC) drawings

Table 9: Preliminary bill of quantities

Item #	Item description	Quantity	Unit
1	Clear and grub TSF footprint (may be done in stages over the 10-year operational life of the facility to limit the cleared surface area to minimise dust generation and erosion). Trees cleared as part of this item to be chipped/mulched and stockpiled for later use as batter protection (ACH's dieback management plan will be implemented to ensure that any dieback affected vegetation is not utilised as woodchip cover for the embankment batters).	295,000	m^2
2	Prepare main embankment footprint - Immediately prior to construction, trimming of all loose material, ripping to a depth of 200 mm, moisture conditioned and compact as per the Earthworks Specification.	77,500	m^2
3	Place clay main embankment - Win, load, haul from within 2km of embankment and place, condition onsite and compact as per the Earthworks Specification and design profile. The construction is to allow for compaction out to the design batters and include removal of excess material to a location directed by ACH.	527,500	m^3 (CCM)
4	Install crest roads geofabric - Install geofabric for crest edge detail including supply of steel pins to secure fabric from wind uplift.	1300	m
5	Crest road - Win, load, haul, place and compact crest road gravel 200 mm thick on the embankment; includes windrow construction and supply of gravel from onsite stockpiles.	13,000	m^2
6	Guide posts - Prepare location by survey, supply and install Main Roads standard wooden guide posts with delineator (50m intervals on straights and 10m on curves <200m radius).	130	#
7	Woodchip batters - Win, load, haul and place wood chip 100 mm thick on the downstream batter slope.	34,000	m^2
8	Supply and install vibrating wire piezometers – Direct push installation with cone penetration test (CPT) rig. Includes supply and installation of cabling, data logger, and lightning protection box.	4	#
9	Install downstream monitoring bore – Depth and specifications to be determined during detailed design.	1	#
10	Prepare decant access ramp – Cut and fill as shown on the drawings.	3,000	m^3 (CCM)
11	Decant ramp road - Win, load, haul, place and compact gravel 200 mm thick on the decant access ramp; includes windrow construction and supply of gravel from onsite stockpiles.	4,550	m^2

3.6 Tailings Discharge and Water Management

3.6.1 Tailings Deposition

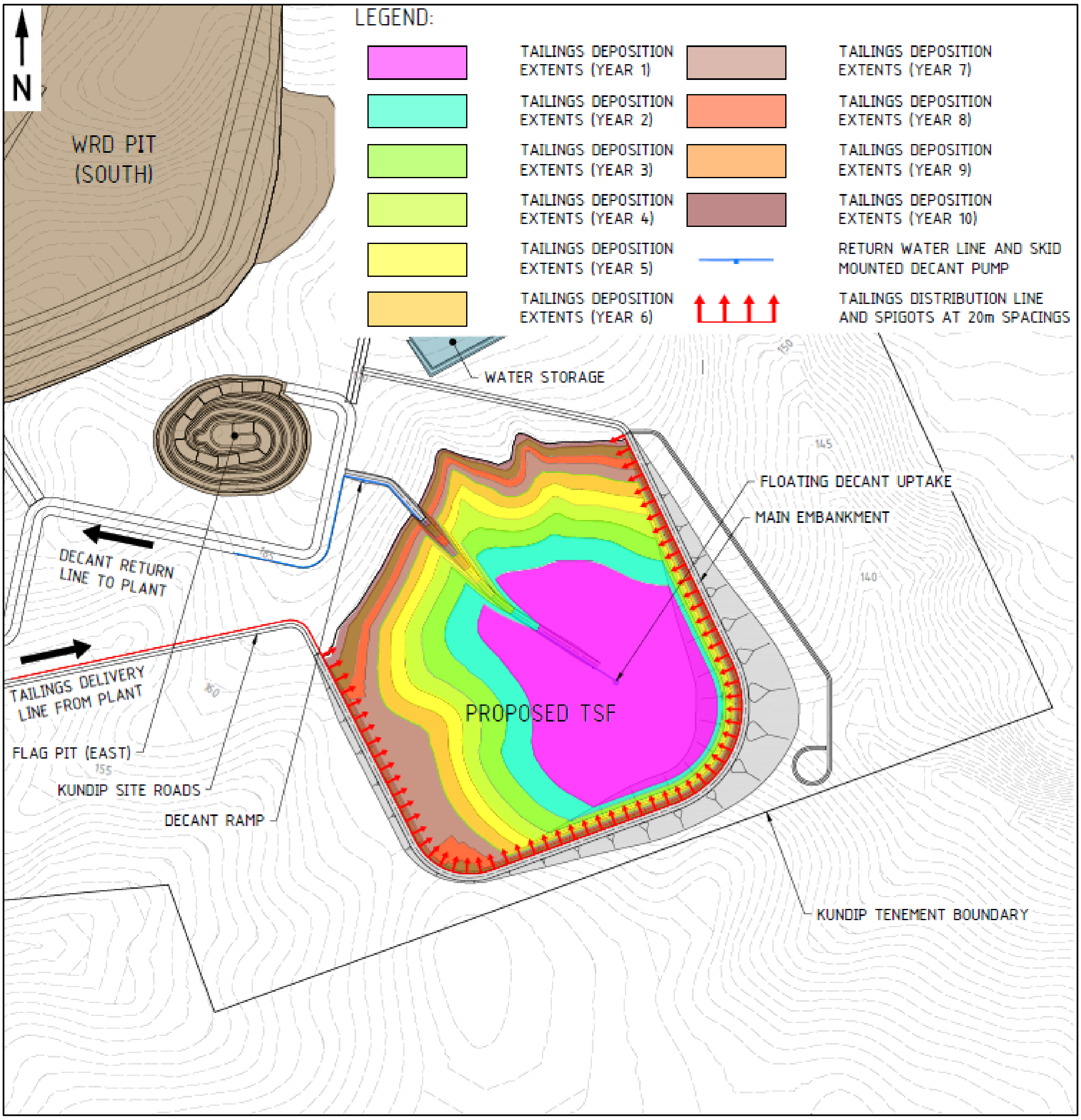
Tailings are expected to be delivered from the Kundip Plant at a production rate of 300,000 tonnes of solids per annum (tpa) for 10 years (base case production scenario). At times throughout the mine plan, the rate of deposition may increase as softer weathered ores are processed. The solids content (% solids) is expected to be approximately 50 %.

A tailings delivery pipeline will be routed to the crest of the embankment and connect to a single ring main with 62 discharge spigots positioned approximately 20 m apart, as shown on Figure 10.

The Kundip TSF has been designed to provide 10 years of tailings storage capacity. The proposed TSF configuration and tailings deposition methodology results in a tailings surface (beach) area of approximately 7.6 ha after 1 year of tailings deposition.

However, the incremental tailings surface (beach) area for each subsequent year is relatively small as shown on Figure 10. Initially 23 of the 62 spigots should be installed on the embankment with 4 additional spigots installed in each subsequent year up to year 6. At this point the remaining spigots should be installed.

Figure 10: Tailings surface (beach) development



Development (filling) of the TSF is shown graphically on Figure 11 and Figure 12 in terms of storage volume, tailings surface area and time rate of rise.

Figure 11: Tailings storage capacity curve

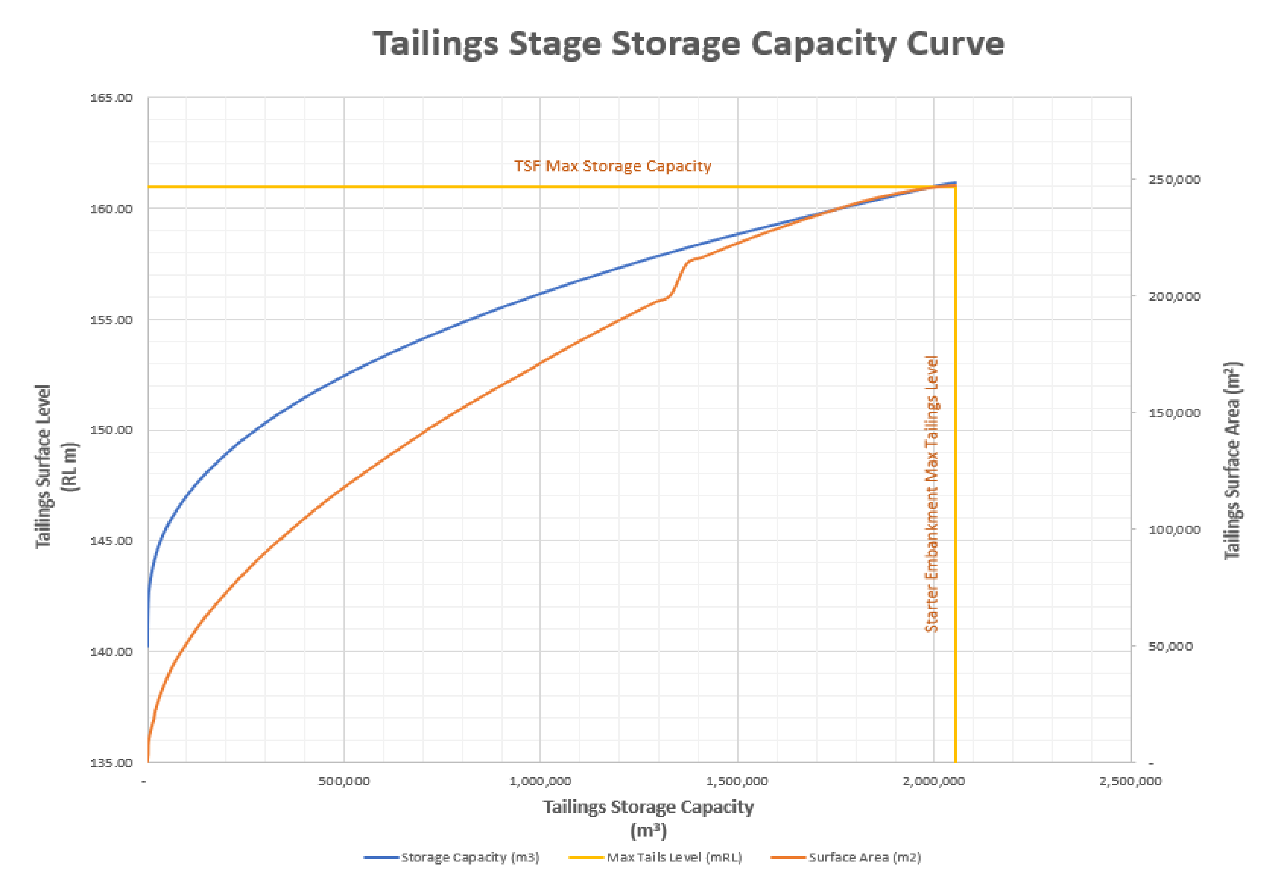
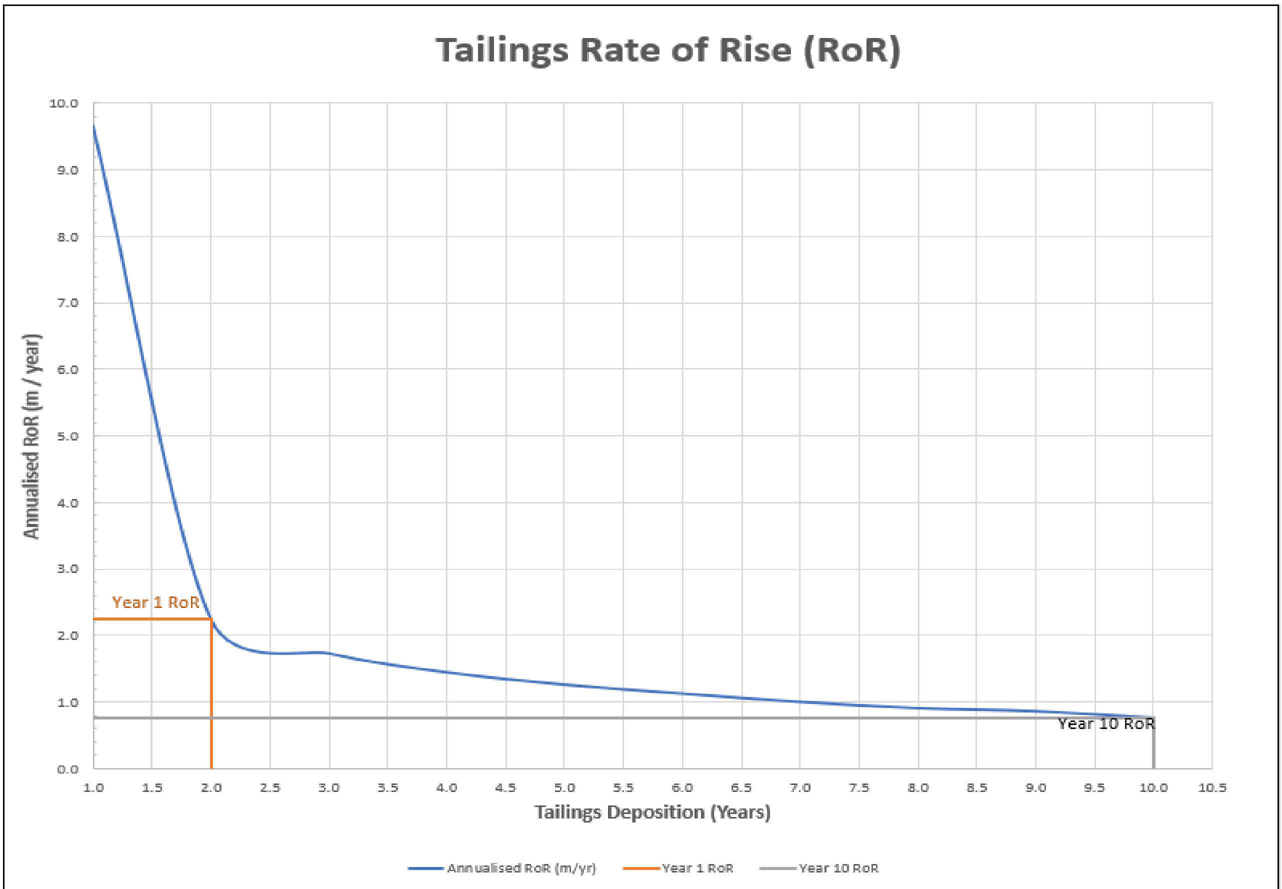


Figure 12: Tailings Rate of Rise



3.6.2 Decant Pond Management

The RGP TSF has been configured to manage the decant pond away from the embankment. Tailings discharged from the embankment will beach towards the natural surface where the decant pond will form in the north-west corner of the facility.

A ramp will be constructed from the north-west corner towards the middle of the TSF basin. A skid-mounted pump will be located on the ramp with a floating uptake located in the pond.

Figure 13: Decant configuration (initial pump and floating uptake location – Year 1)

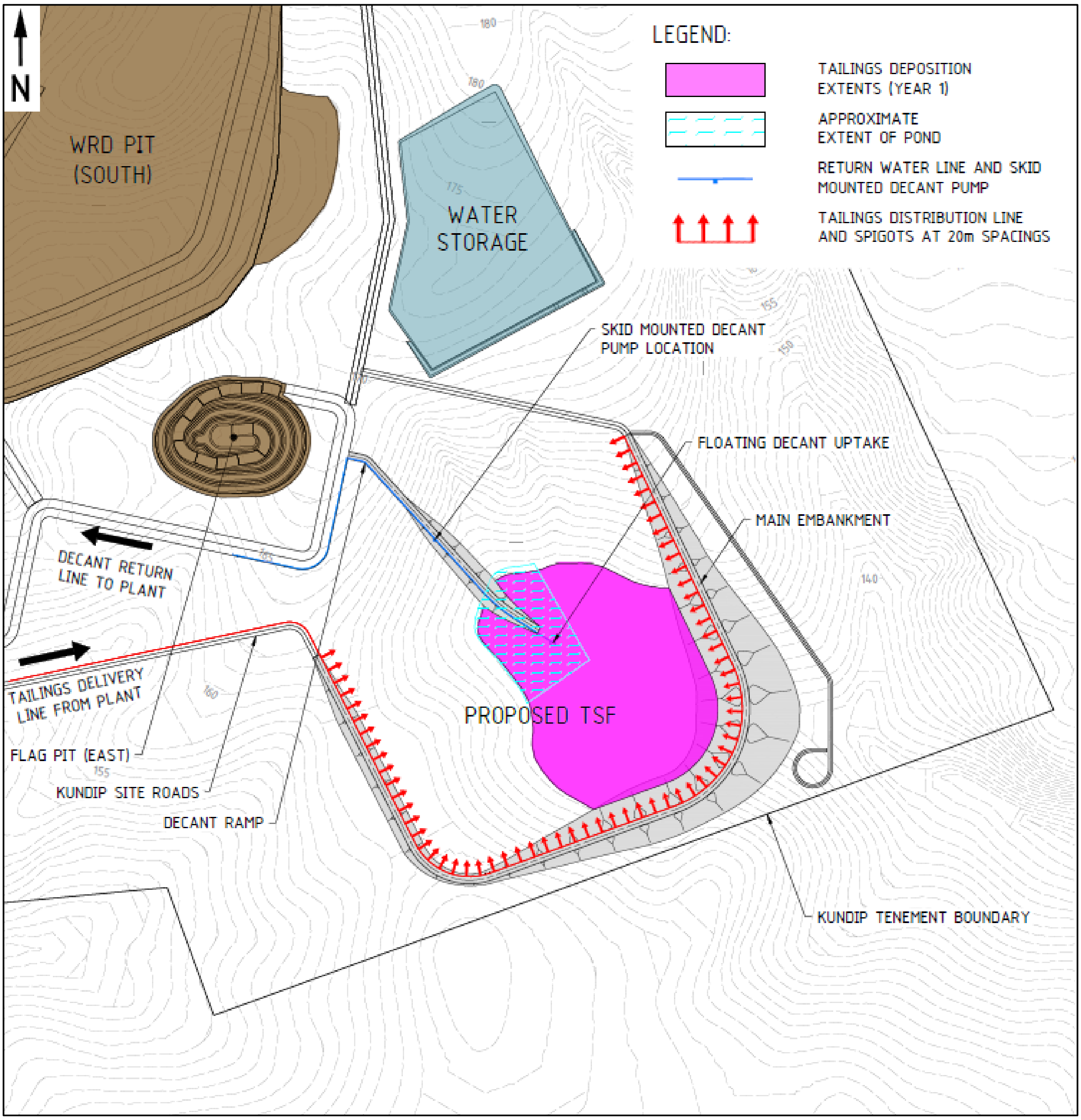
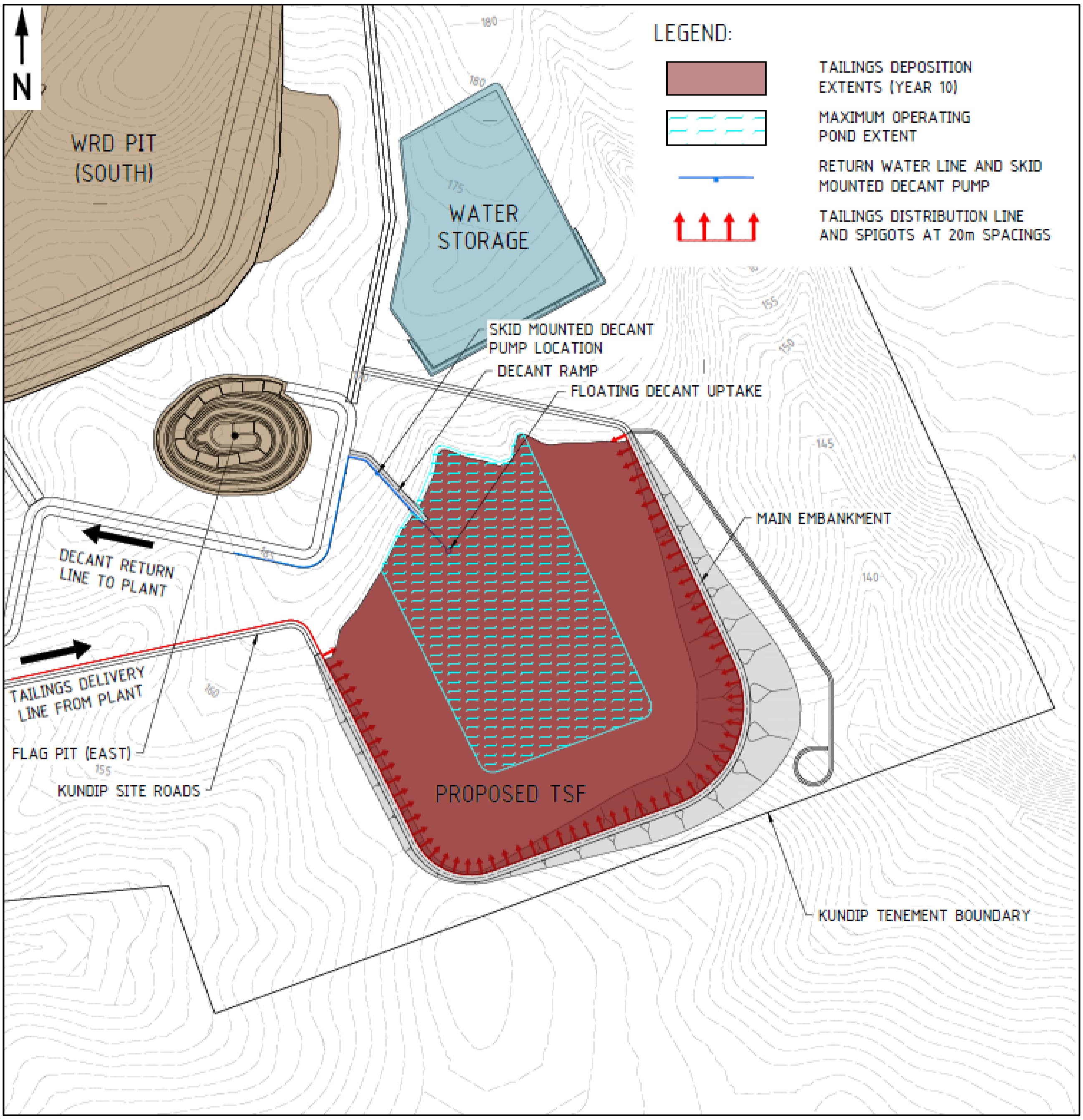


Figure 14: Progressive relocation of decant pump (pump and floating uptake location – Year 10)



As the tailings (beach) surface area continues to expand, the skid mounted pump will be relocated up the ramp to ensure that the pump does not become submerged. Pump specification and sizing are expected to be undertaken during detailed design.

3.7 Covers and Liners

The Kundip TSF design does not call for a liner. However, the design has taken into consideration the low permeability of the existing subsurface material to assist in managing seepage from the TSF basin.

The proposed closure concept outlined in Section 5.0 includes the provision of a vegetation soil cover. Specification of the cover is envisaged to be undertaken during final closure planning and design.

3.8 Quality Assurance

An Earthworks Specification will be developed as part of detailed design development. The specification will include a construction quality assurance (CQA) plan and requirements for on-site third-party quality assurance (QA) monitoring. A construction completion report will be prepared by a Competent Person (typically the design engineer) following substantial completion of TSF construction; in line with the requirements of the CoP: Tailings Storage Facilities in Western Australia (DMP, 2013) and Guide to Departmental requirements for the management and closure of tailings storage facilities (TSFs) (DMP, 2015a).

3.9 Spillways

The CoP: Tailings Storage Facilities in Western Australia (DMP, 2013) states that in Western Australia, the use of spillways is not encouraged. As such, no spillway has been allowed for as part of the design.

4. Operational Requirements

4.1 General

An operating manual will be developed as part of detailed design in accordance with the DMPs Guide to Departmental requirements for the managements and closure of tailings storage facilities (TSFs) and Code of Practice (CoP): Tailings Storage Facilities in Western Australia.

4.2 Management of tailings deposition and water

Tailings are to be deposited from the main embankment in a sub-areal manner in thin lifts and beaching towards the northwest corner of the facility to form a decant pond away from the main embankment. The size of the normal operating pond should be as small as practical to minimise seepage potential whilst providing sufficient depth for operation of the decant pump. The maximum normal operating pond level for a dam full (tailings) scenario which still provides capacity for the 1:100 AEP 72-hour storm event and DMP required freeboard is RL 160.51 m. The maximum normal operating pond level represents the storage of 31 ML, highlighting the robustness of the proposed TSF design arrangement to prevent overtopping. However, it is not the intent of the TSF design that such a large amount of water is stored within the facility.

4.3 Seepage management

Seepage management is achieved by the presence of a low permeability floor, sub-areal deposition in thin lifts to promote air-drying (evaporation), and maintaining a small decant pond away from the main embankment as described in Section 4.2.

4.4 Erosion control

Erosion mitigation features are described in Section 3.4.2. The main embankment batter, upstream and downstream should be inspected on a regular basis and following heavy rainfall events for signs of excessive erosion and repairs made accordingly. Sub areal tailings deposition on thins lifts across the entire tailings beach will ensure the tailings surface is kept sufficiently moist to prevent excessive wind erosion and dusting of the tailings surface.

4.5 Embankment instrumentation

Monitoring instrumentation will be installed in the TSF embankment as shown in plan and section on Figure 15 and Figure 16, including;

- Vibrating wire piezometers (VWP) – to monitor the development of pore pressures (phreatic surface) within the embankment and embankment foundation for assessment of embankment stability (in line with Section 3.4.1),
- Monitoring bores w/VWP – to monitor groundwater levels immediately downstream of the facility for comparison with pore pressures (phreatic surface) measured within the embankment VWP's.

Figure 15: Embankment instrumentation (Plan)

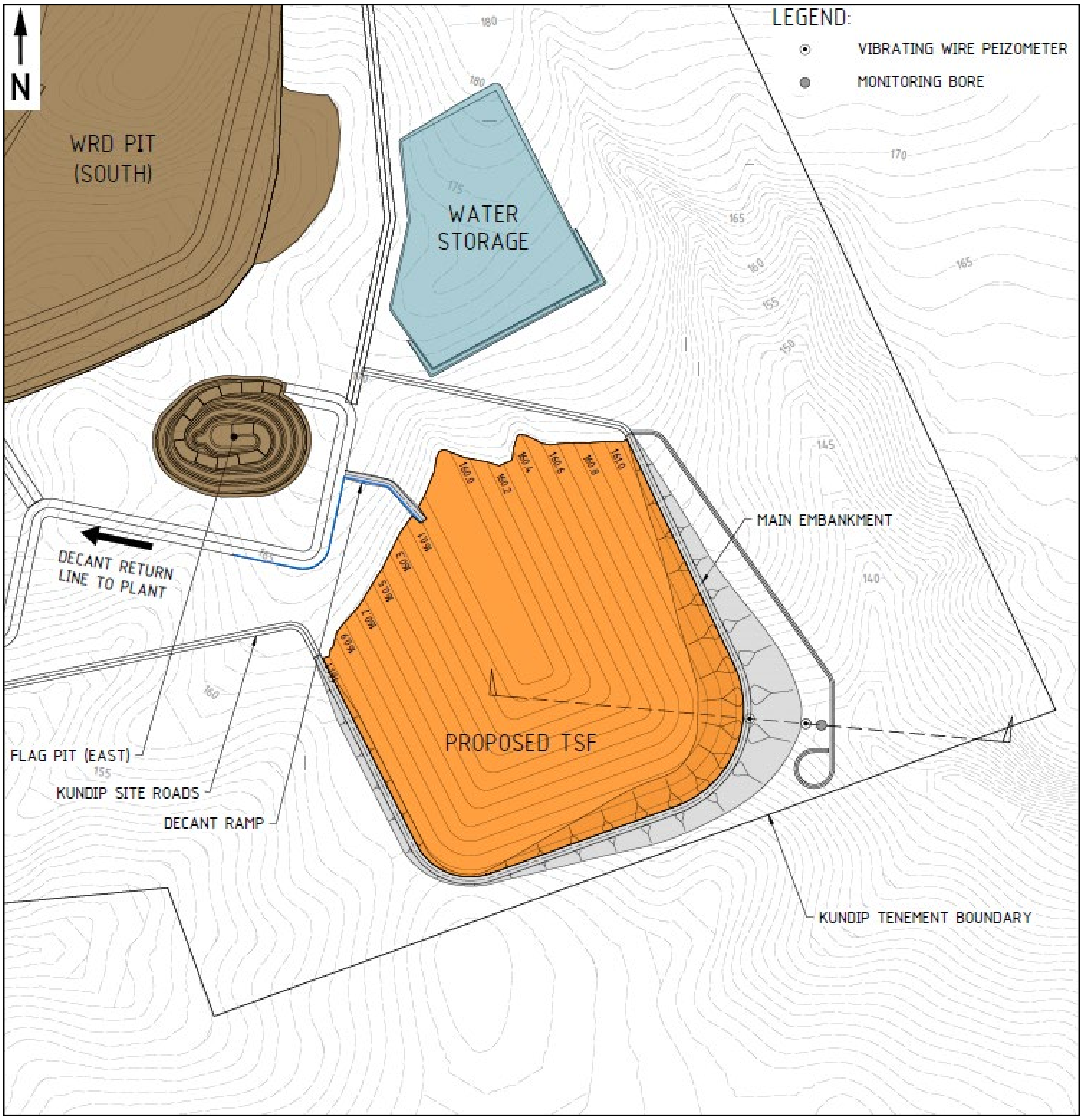
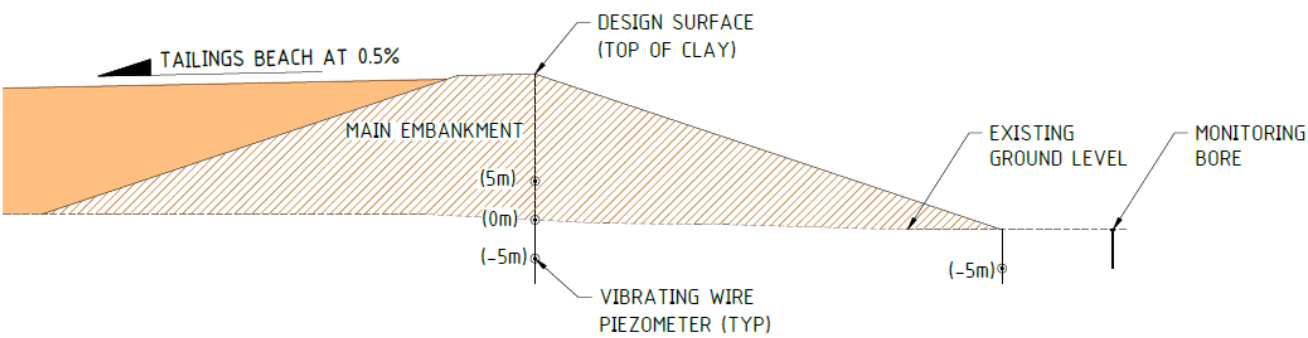


Figure 16: Embankment instrumentation



5. Closure considerations

5.1 General

A RGP Closure Plan has not yet been developed. It is envisaged that a detailed closure plan will be developed at a later stage in conjunction with an RGP site wide closure plan. The proposed RGP TSF has been developed with closure in mind, taking into consideration;

- The DMP's principal closure objectives for rehabilitated mines - Guidelines for Preparing Mine Closure Plans (DMP, 2015b);
  - (physically) safe to humans and animals,
  - (geo-technically) stable,
  - (geo-chemically) non-polluting/non-contaminating, and
  - capable of sustaining an agreed post-mining land use.
- The Environmental Protection Authority's (EPA) objective for Rehabilitation and Decommissioning to ensure that premises are decommissioned and rehabilitated in an ecologically sustainable manner.

The proposed closure concept for the RGP TSF is shown on Figure 17.

Figure 17: RGP TSF closure concept

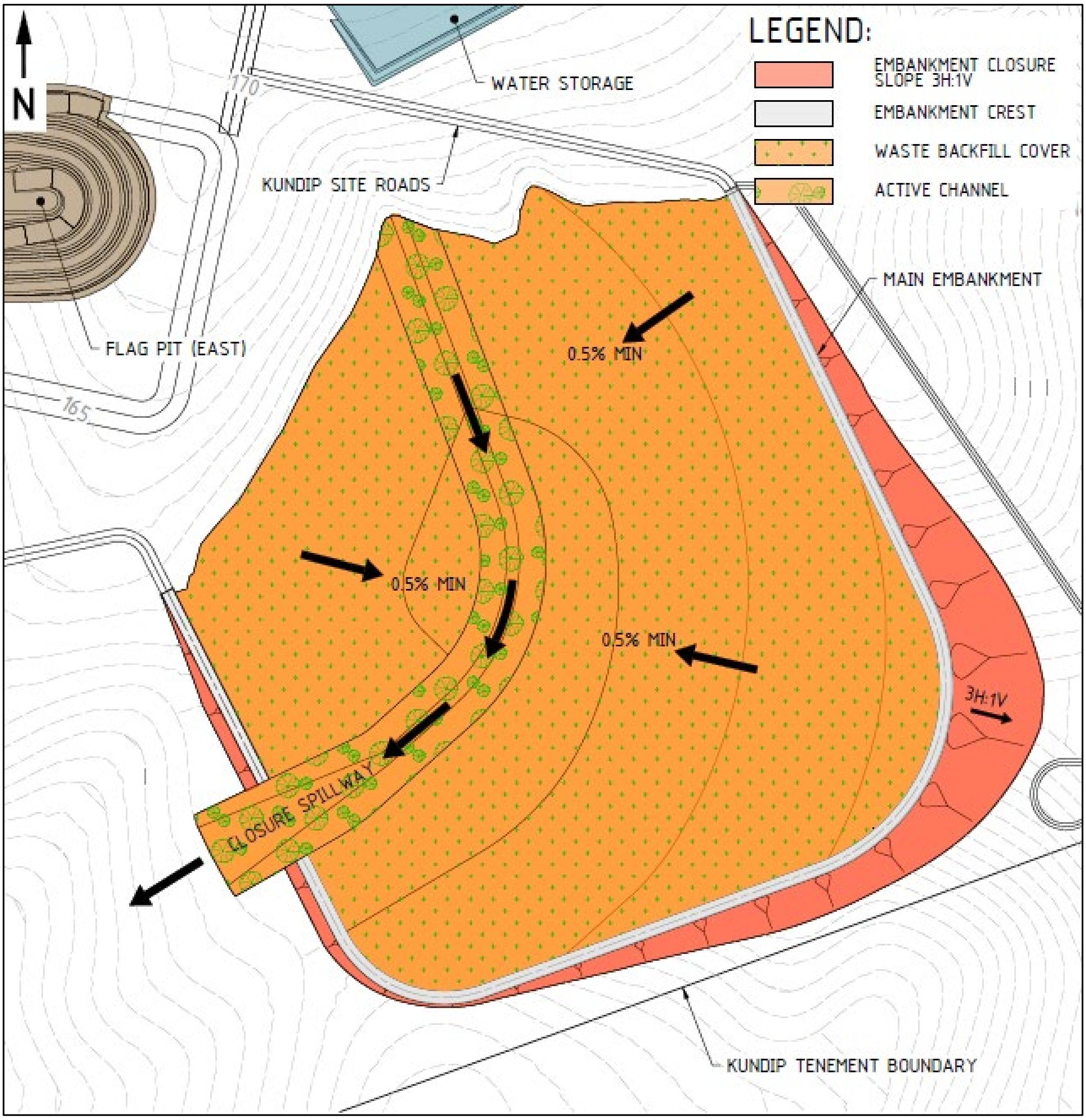
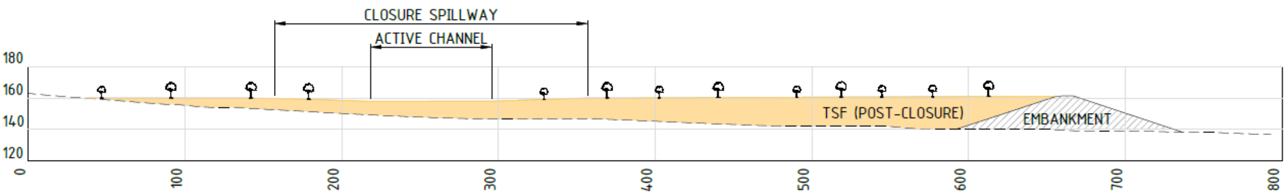


Figure 18: RGP TSF closure concept profile



The closure concept utilises the tailings beach formed during deposition and the natural topography to divert surface water away from the highest part of the embankment. Surface water from the upstream catchment and the TSF surface will drain off the TSF surface via a spillway. The downstream embankment batter at 1V:3H will serve as a final closure surface.

5.2 Decommissioning

Once the TSF has reached capacity and no further deposition is to occur, the tailings delivery line and distribution system will be removed from the main embankment. The decant system may remain in place or on standby until the tailings surface cover and closure spillway have been installed; to provide an interim means of surface water removal. Further detail around decommissioning of the TSF should be coordinated with the project-wide decommissioning and closure plan.

5.3 Tailings surface cover

The tailings surface will be covered with waste and topsoil to provide long term containment and erosion protection of the tailings, as well as providing a suitable medium for re-establishment and sustenance of vegetation. The cover will vary from 0.5 m to 2.0 m in thickness (generally 1.0 m thick) depending on the location of the tailings surface and estimated surface water flow velocities. The tailings surface cover will make use of the tailings beach slope and grade away from the embankment towards the spillway.

5.4 Spillway

A closure spillway will be constructed in the general area shown on Figure 17. The spillway will allow for controlled discharge of surface water collecting within the TSF catchment. The spillway will discharge away from the embankment, providing protection from erosion.

5.5 Rehabilitation

A rehabilitation plan will be developed at a later stage in conjunction with an RGP site wide closure plan.

5.6 Performance monitoring against closure criteria

Closure criteria and a post closure monitoring plan will be developed at a later stage in conjunction with an RGP site wide closure plan.

6. References

1. ANCOLD 2012, Australian National Committee on Large Dams: Guidelines on Planning, Design, Construction, Operation and Closure of Tailings Dams
2. BoM 2016/17, Bureau of Meteorology Website
3. DMP 2013, Code of Practice (CoP): Tailings Storage Facilities in Western Australia
4. DMP 2015a, Guide to Departmental requirements for the management and closure of tailings storage facilities (TSFs)
5. DMP 2015b, Guidelines for Preparing Mine Closure Plans
6. DMP 2015c, Guide to the preparation of a design report for tailings storage facilities (TSFs)
7. Geoscience Australia 2012, The 2012 Australian Earthquake Hazard Map
8. Golder 2016, Concept Design for Waste Landform and Tailings Storage Facility Kundip Gold Project
9. Golder 2017, Updated Concept Design for Tailings Storage Facility at ACH Global Ravensthorpe Gold/Copper Project
10. GCA 2011, Phillips River Project: Geochemical Characterisation of Tailings-Slurry Samples (Trilogy Deposit) - Implications for Process-Tailings Management (DRAFT)
11. Tectonic 2011, Phillips River Project Definitive Feasibility Study

7. Limitations

MHA Geotechnical (MHA) has prepared this feasibility study (FS) level design of the Kundip Mine Site tailings storage facility (TSF) at ACH's Ravensthorpe Gold Project (RGP) to support the overall project Feasibility Study into the technical and commercial viability of RGP in accordance with MHA's proposal dated 1 June 2018. This report is provided for the exclusive use of ACH Minerals Pty Ltd and their consultants for this project only and for the purposes as described in the report. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of MHA, does so entirely at its own risk and without recourse to MHA for any loss or damage. In preparing this report MHA has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after MHA's field testing has been completed.

MHA's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by MHA in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. MHA cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by MHA. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of MHA.

MHA may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to MHA.

Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Should you have any questions regarding the content of this report, please do not hesitate to contact us directly.

Sincerely,

For and on behalf of MHA Geotechnical,

Mitch Hanger

Director – MHA Geotechnical

Geotechnical Engineer

MHAGEOTECHNICAL

Appendix A

Geotechnical Site Investigation Report

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MHA GEOTECHNICAL

Geotechnical Site Investigation Tailings Storage Facility

Ravensthorpe Gold Project

ACH Minerals Pty Ltd

January 2018
Rev 1

MHA Consulting Group Pty Ltd

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Report

Title:	Geotechnical Site Investigation
File:	P02-17-RF
Author(s):	Mitch Hanger
Client:	ACH Minerals Pty Ltd
Contact:	Paul Bennett
Synopsis:	This document details the findings of a geotechnical site investigation pertaining to the proposed Tailings Storage Facility at ACH Minerals' Ravensthorpe Gold Operation.

Document Control

Revision No	Date	Author(s)	Reviewer(s)
A	November 2017	MH; HM	MH
0	January 2018	MH	
1	January 2018	MH	

Distribution

Revision No	Date	Approved	Recipient(s)	No of Copies
A	December 2017	MH	JW	1
0	January 2018	MH	PB	1
1	January 2018	MH	PB	1

Revision

Revision No	Date	Description	Approved
A	December 2017	Draft	MH
0	January 2018	Final Report	MH
1	January 2018	Final Report	MH
Recipients are responsible for eliminating all superseded documents in their possession			

Table of Contents

Tables

Appendices

Abbreviations

AHD	Australian Height Datum
CBR	California Bearing Ratio
DCP	Dynamic Cone Penetrometer
HDPE	High Density Polyethylene
MDD	Maximum Dry Density
OMC	Optimum Moisture Content
QA/QC	Quality Assurance/Quality Control
PI	Plasticity Index
PSD	Particle Size Distribution
SP	Poorly graded sand (USCS)
UU	Undrained Unconsolidated
USCS	The Unified Soil Classification System

d	day
ha	hectare
hr	hour
km	kilometre
m	metre
mm	millimetre
min	minute
yr	year
s	second
t	ton

Executive Summary

Background

MHA Geotechnical (MHA) has prepared this Geotechnical Report as part of a Feasibility Study (FS) level design of the Kundip Mine Site tailings storage facility (TSF) at ACH's Ravensthorpe Gold Project (RGP), to support the overall project Feasibility Study into the technical and commercial viability of RGP. A geotechnical investigation of the proposed site was carried out by MHA Geotechnical (MHA) between the 17th and 23rd of November 2017.

The work has been undertaken at the request of Paul Bennett (Managing Director – ACH Minerals).

The purpose of the geotechnical investigation was to:

- Develop ground profiles for the TSF location,
- Determine the geotechnical properties for foundation and borrow materials,
- Provide comment on the suitability of the site for the proposed development.

The general sub-surface profile is consistent across the site, except for variation in the thickness of alluvial surficial cover and depth to competent bedrock. The sub-surface profile is summarised below:

- The regolith is comprised of a sandy to silty alluvial detrital sediment, with a consistent weathered profile. The horizon was dominantly alluvial and comprised an unconsolidated grey to brown, silty SAND with gravel to sandy SILT with gravel, with roots and organic matter;
- Inconsistently, across the TSF site, immediately underlying the top soil horizon, a transition material comprising a sandy to silty GRAVEL, more consolidated pale brown to white, gravelly horizon. This horizon was locally cemented and diagenetically altered;
- A red brown soil horizon underlies the transition material and where this transition phase was not present, the red brown silty clay layer was present immediately below the top soil. This layer was consistently observed to be indurated and gravelly, with local instances showing a lateritic and conglomeritic texture. This unit was excavated as rock and had been diagenetically altered;
- The material found at the base of all the test pit locations was described as a white, sometimes grey to mottled red, sandy SILTSTONE.

TSF Assessment

Detailed geotechnical analysis of the TSF footprint and materials identified for construction have been performed using data obtained from the site investigation and succeeding lab tests. The foundation and embankment conditions and soil types have been determined from the test pit logs with additional data derived from laboratory testing and previous investigations.

Borrow Material Assessment

An assessment of the borrow material has been undertaken to assess the suitability of both the in-situ and proposed borrow material for construction of the TSF. This includes an assessment of the subsurface conditions and the suitability of blended in-situ and borrow material for embankment construction.

1. Introduction

This report presents the findings of a geotechnical site investigation of the proposed Tailings Storage Facility (TSF) and assessment of proposed construction borrow material. The investigation forms part of a Feasibility Study (FS) level design of the Kundip Mine Site tailings storage facility (TSF) at ACH's Ravensthorpe Gold Project (RGP), to support the overall project Feasibility Study into the technical and commercial viability of RGP, located approximately 25 km by road south-east of the town of Ravensthorpe, Western Australia.

The investigation was carried out by MHA Geotechnical (MHA) between the 17th and 23rd of November 2017. The work has been undertaken at the request of Mr. Paul Bennet (Managing Director – ACH Minerals).

The purpose of the geotechnical investigation was to:

- Develop ground profiles for the TSF location,
- Determine the geotechnical properties for foundation and borrow materials,
- Provide comment on the suitability of the site for the proposed development.

The proposed TSF footprint and test locations are shown on **Figure 1**.

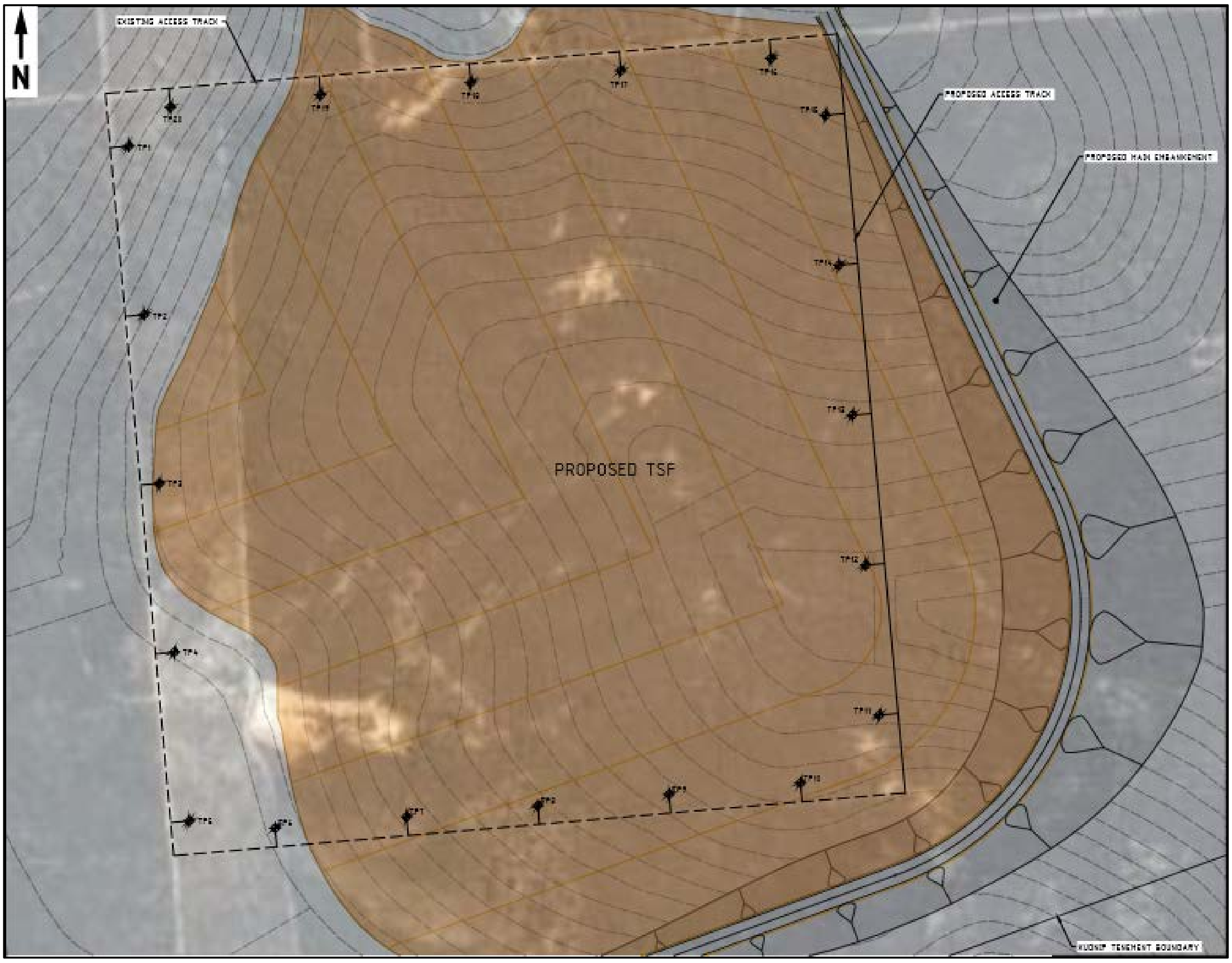

Figure 1 Proposed TSF footprint with associated Test Pit locations.

This report details the results of the geotechnical investigation (test pits, in situ testing, and laboratory test results) that have been carried out at the proposed TSF location. Descriptions of in situ ground conditions are presented, together with interpretation of founding conditions for the TSF and associated structures.

Interpretations, site conditions and design parameters in this report are based on in-situ testing, test pit excavations and laboratory test results from recovered samples, in addition to information gathered as part of previous site investigations.

2. Site Characteristics

2.1 Site location

The Ravensthorpe Gold Project (RGP) is hosted within the north-west trending Archaean Ravensthorpe Greenstone Belt. Nelson (1995) and Savage et al. (1995) have constrained the age of the greenstone belt in this area to 2950-3000 Ma.

2.2 Regional Geology

There are three regional geological units in the area:

- Yilgarn Craton (Archaean) to the north comprising granitoid, granitic gneiss and migmatitic rocks with some greenstone rafts, overlain to the south by;
- Mount Barren Group (Proterozoic) comprising metasedimentary rocks of shale, arenite, dolostone and intruded gabbro-diorite sills; and
- The southeast portion of the region is occupied by Munglinup Gneiss (Proterozoic), which forms part of the Biranup Complex.

The northeast trending Jerdacuttup Fault separates the Munglinup Gneiss from both the Mount Barren Group and the Archaean granite-greenstone terrane. Tertiary sediments of the Plantagenet Group in turn unconformably overlie all Precambrian tectonic units.

2.3 Local Geology

The Kundip mining area lies in a region of steeply-dipping mafic to intermediate volcanic rocks of Archaean age (Annabelle Volcanics) (Witt, 1997). The volcanic rocks have been intruded to the west by granitic rocks, also of Archaean age. The upper reaches of the Steere River follow the contact between the granitic and the volcanic rocks.

Immediately south of the Kundip mining area, the Archaean rocks are overlain by the Proterozoic Mount Barren Group, including sediments of the Kundip Quartzite and the Kybulup Schist. The quartzite dips at about 15 degrees to the south-south-west.

2.4 Typical TSF Regolith Profile

A geotechnical site investigation was carried out by MHA between the 17th and 23rd of November 2017. The purpose of the geotechnical investigation was to:

- Develop ground profiles for the TSF location,
- Determine the geotechnical properties for foundation and borrow materials,
- Provide comment on the suitability of the site for the proposed development.

The typical regolith profile at the TSF site comprises a surficial cover of an unconsolidated sandy silt TOPSOIL underlain by sandy gravelly SILT, underlain by SILTSTONE.

The material encountered can be broadly summarised as:

- 0 m – 0.2 m: SILT; sandy, gravelly TOPSOIL with roots and organic matter;
- 0.2 m – 0.6 m: SILT; red brown, sandy with gravel (transitional zone);
- 0.6 m – 1.0 m: SILTSTONE; red brown, conglomeritic;
- 1.0 m – 3.0 m: SILTSTONE, white sandy/gravelly (considered competent bedrock)

2.5 Seismic Assessment

A seismic hazard assessment of the project site was carried out by MHA to determine design ground acceleration for the RGO area.

The seismic hazard risk assessment contained in GA (2012) is used to quantify the seismic setting for the site. This is a relatively recent and detailed assessment and provides peak ground accelerations (PGAs) for earthquakes of return period 500 years and greater (c.f. the project Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) return periods of 50 years and 100 years respectively). As such its use is conservative but it directly relates to PGAs of interest to the design of earth structures as opposed to use of AS1170.4 Structural design actions – Earthquake actions in Australia that is strictly only applicable to steel, concrete and timber structures.

The PGA is estimated to be 0.06g for the project.

Mining induced ground motion, such as blast induced shaking, is expected to result in relatively minor PGA and for very short durations (cycles). A blast risk assessment will be covered as part of the detailed design process if required.

3. Geotechnical Investigation

3.1 Introduction

A site investigation was undertaken between 17th and 23rd of November 2017. The investigations aimed to assess the ground conditions and evaluate the suitability of the in-situ and borrow material for construction of the proposed TSF.

3.2 Scope of Work

The field work for the investigation comprised:

- Excavation of 20 test pits broadly tracing the internal embankment of the proposed TSF design footprint (arranged in a square configuration with 5 test pits on each side);
- Dynamic Cone Penetrometer testing alongside each test pit location to a maximum depth of 3.0 m.
- Cone Penetrometer Testing (CPT) at 7 locations;
- Dissipation Testing at 4 specified locations;
- Falling head permeability tests at 6 locations;
- Recovery of disturbed soil samples for laboratory testing.

Fieldwork was carried out by two experienced senior geotechnical engineers. The test pit locations were set out using a hand-held Global Positioning (GPS) instrument and were based on the proposed layout of the TSF. No additional tracks cleared throughout the investigation site due to clearing and access restrictions.

The geotechnical fieldwork was undertaken in accordance with the guidelines presented in AS1726-1993, Geotechnical Site Investigations (Ref. 3) and samples were collected for laboratory testing.

3.3 Test Pitting

A total of twenty (20) test pits were excavated across the proposed TSF investigation area using a backhoe excavator to depths of up to 3.1 m. All test pits were logged and photographed by the investigating engineers and samples were collected from selected horizons in each pit for laboratory testing. All the test pits were backfilled with excavated soil on completion of sampling.

A summary of the test pitting is presented in Table 1.

Table 1: Summary of Test Pits

Site	Test Pits	No. of Pits	Max Depth (m)
TSF North track	16, 17, 18, 19 & 20	5	2.8
TSF East track	11, 12, 13, 14 & 15	5	2.8
TSF South track	6, 7, 8, 9 & 10	5	2.9
TSF West track	1, 2, 3, 4 & 5	5	3.1

Logs, photographs and the locations of the test pits are presented in Appendix B. The proposed test pit locations are shown in **Figure 1**.

3.4 Field Permeability Testing

Field permeability tests were conducted in 6 auger boreholes adjacent to selected Test Pits in order to estimate the permeability of the surficial ground profile across the project site. A summary of these tests and their locations are presented below in Table 2.

Table 2: Summary of Field Permeability Tests

Test No.	Test Depth (m)	Target Material	Reason for Termination
TP 3	0.6	SILTSTONE	Target Depth
TP 7	0.4	SILTSTONE	Target Depth
TP 12	0.6	SILTSTONE	Target Depth
TP 13	0.7	SILTSTONE	Target Depth
TP 16/17	0.8	SILTSTONE	Target Depth
TP 19	0.6	SILTSTONE	Target Depth

The detailed calculations for the permeability tests are presented in Appendix C

3.5 Cone Penetrometer Testing

Both Cone Penetration Tests (CPT) and Piezocone Tests (CPTU) were conducted across the site to characterise the sub-surface ground profile and estimate in-situ permeability of target materials. A summary of these tests and their locations are presented below.

3.5.1 Electric Friction Cone Penetrometer Testing

A total of seven (7) Electric Friction Cone Penetrometer Tests (CPTs) were conducted at selected Test Pit locations in order to characterise the sub-surface profile across the site. A summary of the tests conducted is presented in Table 3.

Table 3: Summary of CPTs

Site	EFCPT No.	Maximum Termination Depth (m)	Reason for Termination
TSF South track	CPT6	1.24	Refusal
TSF South track	CPT7	2.37	Refusal
TSF East track	CPT12	2.28	Refusal
TSF East track	CPT12 B	1.68	Refusal
TSF East track	CPT13	2.82	Refusal
TSF East track	CPT14	2.24	Refusal
TSF North track	CPT17	3.04	Refusal

Logs for the CPTs are presented in Appendix D. Test numbers correlate to Test Pit numbers.

3.5.2 CPTU – Dissipation Testing

Based on the results of the EFCPT, a total of 4 Piezocone (CPTU) tests were conducted to target specific material types identified. Stop pause dissipation testing was conducted within each target material to assess the in-situ permeability of the target materials. The piezocone tests conducted are summarised in Table 4.

Table 4: Summary of Piezocone Tests

CPTU No.	Target Depth (m)	Target Material	Test Duration (hrs)
CPTU 12	2.3	SILTSTONE	20.6
CPTU 14	4.3	SILTSTONE	14
CPTU 7/8A	2.4	SILTSTONE	0.1
CPTU 7/8B	2.4	SILTSTONE	14.8

Logs for the CPTU and the dissipation test results are presented in Appendix D. Test numbers correlate to Test Pit numbers.

3.6 Laboratory Testing

Laboratory testing was carried out on borrow material and selected disturbed samples recovered from test pits, in order to characterise the in-situ and borrow materials for design and construction purposes. The testing was carried out by a NATA accredited laboratory in accordance with Australian Standards and comprised the following:

- Particle Size Distribution;
- Specific Gravity;
- Atterberg Limits;
- Compaction Testing (Standard and Modified Compactive Effort);
- Multi Stage Unconsolidated Undrained Triaxial Testing;
- Single Stage Consolidated UndrainedTriaxial Testing
- Crumb Test; and
- Pinhole Dispersion test (95% and 98% MMDD).

A summary of the average laboratory test results for each material type is presented below in Table 5. Details of the samples selected for testing, the laboratory test schedule and results are presented in Appendix E.

Table 5: Summary of Laboratory Test Data for Materials Encountered

Material Location	Typical Depth to Base	PSD			Physical Parameters			
		Fines (<75 µm)	Sand (>75 µm)	Gravel (>2mm)	Maximum Dry Density	Optimum Moisture Content	Emmerson Class	Plasticity Index
	m	%	%	%	%	%	No.	%
Alluvial Cover – Sandy SILT	0.6	49.0	32.0	19.0	1.9	13.0	2.3	16.5
Bedrock - SILTSTONE	>3.0	43.0	37.0	20.0	1.8	15.0	4.0	9.0
Borrow Material - STOCKPILED	NA	-	-	-	1.7	14	6	NP

*NP denotes Not Plastic.

4. Sub-surface Ground Conditions

4.1 Introduction

The sub-surface ground conditions described below are based on the findings of the geotechnical investigations performed along the 4 access tracks which broadly trace the interior of the proposed TSF footprint.

4.2 TSF Footprint

A total of twenty (20) test pits were excavated within the vicinity of the TSF footprint. These were located along the interior of the proposed TSF embankments.

The general sub-surface profile is consistent across the site, with only local variation in the degree of cementation and percentage of sand and gravel contained within the soil layers. The upper most layer is composed entirely of a sandy, silty, organic soil, which typically comprises the top 0.2m. Beneath the surface topsoil, is a layer of alluvium which is spatially inconsistent nature; this is likely related to influence of sub-surface ground waters and the resultant weathering.

Where this horizon is present, it typically extends from 0.2 m – 0.6 m and gradually transitions into the more diagenetically altered siltstone below. The transitional alluvial cover consists of medium dense to dense sandy SILT with gravel underlain by a sandy SILT with local cementation and gravel.

The typical sub-surface profile beneath the TSF Site is summarised in Table 6.

Table 6: Summary of Typical Sub-Surface Profile at TSF Site

Location	Description
GL – 0.2 m	Sandy SILT [ML], soft, non-plastic, brown-grey with gravel, dry, contains roots and organics.
0.2 m – 0.6 m	Sandy SILT [ML], soft, non-plastic, pale-brown with gravel, dry, transition phase between topsoil and red-brown horizon. Loose and unconsolidated material.
0.6 m – 1.2 m	Sandy SILT [ML], stiff, non-plastic, red-brown with gravel, dry, locally very conglomeritic with occasional lateritic texture.
1.2 m – 3. 0m	Sandy SILT [ML], stiff, non-plastic, white-grey, mottled red, dry, contains quartz cobbles (excavated as rock - SILTSTONE).

5. Borrow Material Assessment

5.1 Introduction

Borrow material for the construction of the embankments has been identified as material from within the footprint of the TSF and stockpiled material from adjacent open pits. For the purpose of the assessment and in order to estimate the shear strength parameters of the borrow material, the following assumptions have been made;

- The same borrow material will be available from these stockpiles, in sufficient volumes for construction of the embankments;
- The material selected for borrow retains similar cohesion, permeability and shrink / swell properties across the pits;
- Prior to being used as a construction material, the material will be worked as needed to conform to specifications for embankment fill.

5.2 Borrow Materials

Based on the results of the laboratory test work, the embankment material is likely to comprise low plasticity clay with silt, sand and gravel, and is likely to encompass the following material types in Table 7 under the Unified Soil Classification System (USCS).

Table 7 presents expected values for maximum unit wet density, effective stress cohesion and friction angle for these materials after Hunt (1986). Design density and strength values adopted for the embankment material are also presented.

Table 7: Embankment Material Geotechnical Parameters

USCS	Description	Maximum Wet Density (σ)	Saturated Effective Cohesion (c')	Effective Stress Friction Angle (ϕ')
		kN/m^3	kPa	degrees
SM-SC	Sand-silt clay mix with slightly plastic fines	19.9 – 22.7	14	33
SC	Clayey sand, poorly graded sand-clay mix	19.6 – 21.8	11	31
ML	Inorganic silts and clayey silts	18.5 – 21.2	9	32
ML-CL	Mixture of inorganic silt and clay	19.2 – 21.1	22	32
CL	Inorganic clays of low to medium plasticity	18.5 – 21.1	13	28
DESIGN	Embankment Material	21.0	10	30

6. TSF Foundation Assessment

6.1 Introduction

An analysis of the TSF location has been performed using data obtained from the site investigation. The foundation conditions and soil types have been determined from the in-situ testing and laboratory testing.

For each soil type, compressibility characteristics were determined according to a combination of the following:

- Soil description;
- Cone Penetrometer Tests;
- Dynamic Cone Penetrometer Tests (DCP's), and;
- Laboratory data.

Stability analyses were carried out, based on the soil compressibility characteristics within the profile. The results of these analyses are presented in the following sections.

6.2 Soil Characterisation

For the purpose of constructing a model considered representative of the project site, the sub-surface profile was characterised in terms of composition by means of physical inspection, laboratory test results and CPT results. Visual inspections combined with the results of specific laboratory tests aided in refining the sub-surface ground profile according to description, however CPT results were used to further characterise the sub-surface profile based on mechanical characteristics by accurately measuring in-situ parameters.

The CPT can provide estimates as to the mechanical characteristics (strength, stiffness, compressibility) of the soil and the soil behaviour type (SBT). CPT data provides a repeatable index of the aggregate behaviour of the in-situ soil in the immediate area of the probe. Hence, a prediction of soil type based on CPT is referred to as Soil Behaviour Type (SBT).

6.2.1 CPT Soil Behaviour Type

The most commonly used CPT Soil Behaviour Type (SBT) chart was suggested by Robertson et al. (1986). This chart uses the basic CPT parameters of cone resistance (qt) and friction ratio (Rf). The chart is global in nature and can provide reasonable predictions of soil behaviour type for CPT soundings to a depth of 20 m without the need for normalising the parameters. Overlap in some zones should be expected and the zones can be modified somewhat based on local experience.

The accuracy of the soil behaviour type characterisation can be further improved when pore pressure measurements are collected, and the data is normalised for the effective overburden stress. In soft soils the penetration pore pressures can be very large, whereas, in stiff heavily over-consolidated CLAY or dense SILT and silty SAND the penetration pore pressures (u2) can be small and sometimes negative relative to the equilibrium pore pressures (u0). The rate of pore pressure dissipation during a pause in penetration can also guide in the characterisation of soil type and is discussed in detail in Section 6.4. To simplify the characterisation, the normalized cone parameters Qt and Fr can be combined into one Soil Behaviour Type index, Ic. The Soil Behaviour Type index can be defined as follows;

$$\mathrm{Ic} = ((3.47 - \log \mathrm{Qt})^2 + (\log \mathrm{Fr} + 1.22)^2)^2$$

where: $Qt = (qt - \sigma_{vo}) / \sigma'_{vo}$ (normalized cone penetration resistance).

$Fr = (fs/(qt - \sigma_{vo})) \times 100$ (normalized friction ratio, in%).

Table 8 below can be used to characterise soil based on the Soil Behaviour Type index, Ic:

Table 8: Soil Behaviour Index Summary

Ic	Soil Behaviour Type
>3.6	Organic CLAY
2.95 – 3.6	Silty CLAY and CLAY
2.6 – 2.95	Clayey SILT to Silty CLAY
2.05 – 2.6	Silty SAND – Sandy SILT
1.31 – 2.05	SAND to Silty SAND
<1.31	Gravelly SAND – Dense SAND

For the purpose of defining the subsurface ground conditions across the project site, test results were characterised using normalised SBT charts and the Soil Behaviour Type Index.

6.3 Material Permeabilities

In order to further refine the sub-surface ground profile, the in-situ permeability of the ground profile were estimated from field tests, laboratory tests and CPT tests and CPTU dissipation tests. The details of each of these is discussed in the following sections.

6.3.1 Field Permeability Testing

Field permeability testing conducted as part of the geotechnical investigative works comprised falling head tests conducted on hand auger boreholes across the project site. A summary of the field permeability testing is presented in Table 9.

Table 9: Summary of Field Permeability Tests

Location	Test Depth (m)	Average Permeability k_h (m/s)
TP 3	0.6	1.0 E^{-7}
TP 7	0.4	3.0 E^{-7}
TP 12	0.6	5.6 E^{-7}
TP 13	0.7	1.5 E^{-6}
TP 16/17	0.8	4.5 E^{-7}
TP 19	0.6	1.2 E^{-5}

The results of the field permeability testing varied slightly and were considered only indicative of the average permeabilities for the increase in depth below ground and not for each material type encountered within the sub-surface ground profile.

6.3.2 Laboratory Permeability Testing

In order to further define the permeability of target materials, laboratory permeability testing was conducted as part of the geotechnical investigative works and comprised falling head tests. These tests were conducted on selected samples identified as representative of the sub-surface profile across the site.

The results of the laboratory permeability test results are summarised in Table 10.

Table 10: Summary of Laboratory Permeability Tests

Sample Location	Sample Depth	Constant Head Permeability (m/s)
TP02	0.5 m – 1.1 m	6.5 E^{-9}
TP03	0.0 m – 0.7 m	1.6 E^{-8}
TP06	0.6 m – 2.7 m	5.2 E^{-9}
TP13	0.7 m – 2.8 m	4.1 E^{-9}
TP16	1.0 m – 2.8 m	6.2 E^{-9}

Note: "-" denotes that a sample was not tested.

It is important to note that the results of the falling head laboratory permeability tests are considered "remoulded" permeability tests as a result of being compacted to 95 % of the maximum dry density of the material at an optimum moisture content of between 21 % and 24 %, as determined by the Maximum Dry Density testing. As a result, the remoulded permeabilities are considered two orders of magnitude lower than the expected in-situ permeabilities, depending on the state of the soil.

The results of the laboratory permeability testing identified that a combination of the sandy SILT, mottled silty CLAY and high plasticity CLAY used as borrow material for the construction of the embankments and cut-off keys would provide a low permeability composite suitable for construction. The results also provided guidance as to the materials to be targeted with the use of CPT and Piezocone testing.

6.3.3 Piezocone Permeability Testing

An approximate estimate of soil hydraulic conductivity or coefficient of permeability, k, can be made from an estimate of the Soil Behaviour Type index. The average relationship between soil permeability (k) and SBTn Ic can be represented by:

$$k = 10^{(0.952 - 3.04\, Ic)} \text{ m/s; for } 1.0 < Ic \leq 3.27$$

$$k = 10^{(-4.52 - 1.37\, Ic)} \text{ m/s; for } 3.27 < Ic < 4.0$$

The above relationships can be used to provide an approximate estimate of soil permeability (k) and to show the likely variation of soil permeability with depth from a CPT sounding. Since the normalized CPT parameters (Qtn and Fr) respond to the mechanical behaviour of the soil and depend on many soil variables, the suggested relationship between k and Ic is approximate and should only be used as a guide.

For improved estimates, pore pressure dissipation tests were performed in soil layers defined by the CPT. These values were interpreted using two separate methods as detailed in Appendix D. The interpretation methods account for the soil shear strength, soil rigidity, and confining stresses likely to influence the soil behaviour and as a result the parameter to be interpreted. Numerical analyses have previously demonstrated that the rate of dissipation increases as the soil rigidity or the soil confining pressure increases, which is a consequence of higher excess pore pressure gradient at higher depths or at larger rigidities.

During a pause in penetration, any excess pore pressure generated around the cone will start to dissipate. The rate of dissipation depends upon the coefficient of consolidation, which in turn, depends on the compressibility and permeability of the soil. The rate of dissipation also depends on the diameter of the probe. A dissipation test is performed at any required depth by stopping the penetration and measuring the decay of pore pressure with time. In order to accurately estimate the in-situ permeabilities of the target materials, the equilibrium pore pressure was required. As such, each dissipation test was continued until no further dissipation was observed. This can occur rapidly in SAND, but may take many hours in plastic clays.

A total of four (4) piezocone dissipation tests were conducted within target materials. The results of the dissipation tests are summarised in Table 11.

Table 11: Summary of Piezocone Test Result

CPTU No.	Target Depth (m)	Target Material	Permeability k_h (m/s)
CPTU 12	2.3	SILTSTONE	5 E^{-9}
CPTU 14	4.3	SILTSTONE	5.2 E^{-9}
CPTU 7/8A	2.4	SILTSTONE	1.7 E^{-6}
CPTU 7/8B	2.4	SILTSTONE	4.89 E^{-8}

It should be noted that given that the SILTSTONE is inherently dry, the development of negative pore pressure necessitated that the piezocone be extracted and re-saturated several times at each location. This may affect the results of the dissipation test. The results of the in-situ permeability testing are considered indicative of the average permeabilities for the increase in depth below ground

6.4 Laboratory Triaxial Test Interpretation

6.4.1 Multistage Unconsolidated Undrained Triaxial Testing

A part of the geotechnical investigation, three (3) samples were collected for Multistage Unconsolidated Undrained triaxial testing. These samples were considered representative of the following material types:

- Sample 1: White-grey Sandy SILT from Test Pit 02 at 0.5 m below ground level;
- Sample 2: White-grey Clayey SILT from Test Pit 13 at 0.7 m below ground level; and
- Sample 3: Stockpiled borrow material B1 – SILT/CLAY.

Critical to the estimation of shear strength parameters used in the analysis of the settlement and stability of the embankments is the interpretation of the triaxial data. The results of the multistage consolidated undrained triaxial tests were reviewed and interpreted in order to estimate the in-situ and composite shear strength and compressibility parameters.

The multistage unconsolidated undrained triaxial tests were conducted at nominal cell pressures of 75, 150, 300 kPa for each specimen, except for Sample 1, where the test at 300 kPa could not be carried out because the sample had reached a strain of 20% before the third stage could begin. Each test specimen was compacted to 95% standard compactive effort at optimum moisture content. A summary of the interpreted test is data for each sample presented below.

Table 12: Sampled 1 UU Triaxial Test Results

Parameter	Result
Initial Moisture Content (%)	9.4
Wet Density (t/m^3)	2.13
Dry Density (t/m^3)	1.94
Estimated Voids Ratio	0.37
Estimated Saturation Ratio	71
Cohesion	120
Internal Angle of Friction	20
Strain at Failure (%)	11.5 (Stg 1); 19.28 (Stg 2); -
Deviator Stress at Failure	444 (Stg 1); 522 (Stg 2); -
Modulus of Elasticity E_s (Mpa)	22.1 (Stg 1); 63.8 (Stg 2); -

Table 13: Sampled 2 UU Triaxial Test Results

Parameter	Result
Initial Moisture Content (%)	16.2
Wet Density (t/m^3)	1.88
Dry Density (t/m^3)	1.62
Estimated Voids Ratio	0.64
Estimated Saturation Ratio	67
Cohesion	24.1
Internal Angle of Friction	16.5
Strain at Failure (%)	5.4 (Stg 1); 9.78 (Stg 2); 15.09 (Stg 3)
Deviator Stress at Failure	120 (Stg 1); 191 (Stg 2); 301 (Stg 3)
Modulus of Elasticity E_s (Mpa)	12.8 (Stg 1); 24.5 (Stg 2); 44.7 (Stg 2)

Table 14: Sampled 3 UU Triaxial Test Results

Parameter	Result
Initial Moisture Content (%)	15.6
Wet Density (t/m^3)	1.86
Dry Density (t/m^3)	1.61
Estimated Voids Ratio	.65
Estimated Saturation Ratio	64
Cohesion	46.1
Internal Angle of Friction	21.0
Strain at Failure (%)	5.84 (Stg 1); 10.17 (Stg 2); 15.22 (Stg 3)
Deviator Stress at Failure	208 (Stg 1); 293 (Stg 2); 446 (Stg 3)
Modulus of Elasticity E_s (Mpa)	17.1 (Stg 1); 41.9 (Stg 2); 46.2 (Stg 2)

All three shear strength envelopes were observed as being close to linear. All the samples were partially saturated, between 63 and 71%, and this will have had the effect of increasing both the angle of friction and the apparent cohesion. If the soil becomes saturated (this is unlikely if it is to be used in fill and the results are to be used for design during the construction period) then the shear strength in this condition is likely to be less than indicated by the shear strength envelopes.

Sample 1 was extremely compact having a low void ratio of 0.37 and a dry density of 1.94 t/m3. As a result, the apparent cohesion was high at 120 kPa. Presumably this is the effect of the gravel component of the soil. Only two all-round stress increments were carried out on this specimen because a strain of 20% had been reached after the second increment. The specimen failed by barrel failure at high strains, indicating that any shear failure of a structure built with this soil will have experienced excessive settlements well before any shear failure. Based on a 1% strain as the failure criterion, the equivalent shear strength parameters would equate to a cohesion of 0 kPa and an internal angle of friction of 35°. The stress strain curves start to become significantly non-linear beyond 1% strain. The failure criterion used was maximum deviator stress, but it appears from the stress strain curves that this might not have been completely attained.

Sample 2 and Sample 3 had relatively high void ratios of 0.64 and 0.65 respectively, and showed correspondingly lower cohesions of 24.1 and 46.1 kPa. Both are low strength materials, presumably owing to their silt and clay content, and the low dry densities of 1.61 and 1.62 t/m^3.

The modulus of elasticity was calculated from the stress strain curves, using the steepest portion of the curves, and are tabulated above. All the test results appear to be internally consistent and reliable although consideration will need to be given to the level of saturation expected during operation of the facility.

6.4.2 Multistage Consolidated Undrained Triaxial Testing

A part of the geotechnical investigation, three (3) samples were collected for Single Stage Consolidated Undrained triaxial testing. These samples were considered representative of the following material types:

- Sample 4: White-grey Sandy SILT from Test Pit 02 at 0.5 m – 1.1 m below ground level;
- Sample 5: White-grey Clayey SILT from Test Pit 13 at 0.5 m – 1.1 m below ground level; and
- Sample 6: Stockpiled borrow material B1 – SILT/CLAY.

Critical to the estimation of shear strength parameters used in the analysis of the settlement and stability of the embankments is the interpretation of the triaxial data. The results of the multistage consolidated undrained triaxial tests were reviewed and interpreted in order to estimate the in-situ and composite shear strength and compressibility parameters.

The multistage unconsolidated undrained triaxial tests were conducted at nominal cell pressures of 75, 150, 300 kPa for each specimen, except for Sample 1, where the test at 300 kPa could not be carried out because the sample had reached a strain of 20% before the third stage could begin. Each test specimen was compacted to 95% standard compactive effort at optimum moisture content. A summary of the interpreted test is data for each sample presented below.

Table 15: Sampled 4 CU Triaxial Test Results

Parameter	Result
Initial Moisture Content (%)	9.3
Wet Density (t/m^3)	2.14
Dry Density (t/m^3)	1.96
Estimated Voids Ratio	0.36
Estimated Saturation Ratio	70
Strain at Failure (%)	7.36
Deviator Stress at Failure	477
Modulus of Elasticity E_s (Mpa)	56.7

Table 16: Sampled 5 CU Triaxial Test Results

Parameter	Result
Initial Moisture Content (%)	16.2
Wet Density (t/m^3)	1.88
Dry Density (t/m^3)	1.62
Estimated Voids Ratio	0.64
Estimated Saturation Ratio	67
Cohesion	24.1
Internal Angle of Friction	16.5
Strain at Failure (%)	7.68
Deviator Stress at Failure	484
Modulus of Elasticity E_s (Mpa)	23.2

Table 17: Sampled 6 CU Triaxial Test Results

Parameter	Result
Initial Moisture Content (%)	15.6
Wet Density (t/m^3)	1.86
Dry Density (t/m^3)	1.61
Estimated Voids Ratio	0.65
Estimated Saturation Ratio	64
Strain at Failure (%)	10.46
Deviator Stress at Failure	578
Modulus of Elasticity E_s (Mpa)	30

The results are consistent with what one might expect from recompacted soils. The Volume Change Curves indicate that the specimens dilate initially, and the moisture content, density and void ratios are consistent with the UU results.

Moduli of elasticity were calculated from the test data and the results appear to be internally consistent and reliable.

6.5 Soil Compressibility

In elastic analysis, settlement is most sensitive to the selection of input parameters for soil compressibility, that is modulus of elasticity (E) and Poisson's ratio (μ). The analysis uses values of E and μ which are regarded as representative of the foundation under consideration. Undrained moduli are used to calculate immediate settlement and drained moduli are used to calculate long-term settlement, including creep.

Compressibility parameters for the in-situ soils were evaluated from in-situ and laboratory testing as well as visual and tactile assessments of the materials encountered in test pits and boreholes.

The range of moduli values assigned to each horizon is based on experience with similar soils, and correlations with field assessments. The recommended drained moduli of elasticity (E) for each interpreted horizon are summarised in Table 18.

Table 18: Summary of Inferred Soil Foundation Elastic Moduli

Structure	Layer	Consistency	Depth to base (m)	E (MPa)		
				Lowest	Expected	Highest
TSF Embankment	Alluvial Cover – Sandy SILT	Soft - Firm	0.2	10	20	30
	Conglomerate – Sandy SILT	Stiff	1.2	35	50	70
	Bedrock - SILTSTONE	Stiff – V. Stiff	>3	75	85	100

Note: ">" indicates the base of layer was not encountered.

6.6 Settlement Analysis

Standard elastic settlement analysis has been used to examine the potential settlements of the embankments. The method takes into consideration the layered soil profile by using the variation in moduli of elasticity with depth and allows for pre-consolidation. Standard elastic settlement analysis is based on the equation:

$$S = \frac{q \times B \times (1 - \mu^2) \times i}{E}$$

where: S = settlement.

q = increase in effective pressure.

B = width or diameter of footing.

μ = Poisson's ratio.

i = influence factor.

E = modulus of elasticity.

The influence factor (i) takes into account the shape of the footing or embankment and the thickness of the various soil horizons. Factors for footings of various dimensions and layer thickness ratios are published by Harr (Ref. 7) and Lee, White and Ingles (Ref. 8).

6.6.1 Loadings for Analysis

Estimated settlements have been calculated for the TSF embankment using interpreted design parameters, foundation geometries and loadings typically expected during construction and operation. Additional settlement analyses will need to be carried out as part of the detailed design, if the embankment geometries and layout change and/or foundation loads, sizes and founding depths vary from those described herein.

The maximum load applied to the embankment is assumed to be 400 kPa as a result of effective overburden (embankment to 19.74 m high). For the purpose of the analysis the average load is assumed to be 250 kPa.

6.6.2 Settlement Analysis Results

Total and differential settlements were estimated for the embankments assuming a range of moduli of elasticity as presented in Table 11. Total settlement is the expected maximum settlement of each embankment, and differential settlement is the potential difference in settlement across the embankment caused by differential loads and foundation conditions. These settlements are based on the expected bearing pressures and foundation geometries. The estimated settlements are summarised in Table 19.

Table 19: Estimated Settlement of TSF Embankment

Embankment	Bearing Pressure	Most Compressible		Expected Compressibility		Least Compressible	
	Total (kPa)	Total (mm)	Differential (mm)	Total (mm)	Differential (mm)	Total (mm)	Differential (mm)
TSF Embankment	250	40	5	30	5	20	5

6.7 Slope Stability Assessment Methodology

Slope stability assessment was undertaken assuming a uniform slope of 1 (V) : 3 (H) upstream and downstream batters. The target static stability factor of safety (FoS) is 1.50, and the maximum allowable degree of saturation in the slope to achieve this was assessed.

The following analysis techniques were used:

- Hoek & Bray (1981) – chart solution for circular failure slip with upstream tension crack
- Michalowski (2002) – chart solution for log-spiral failure slip
- Cousins (1978)– chart solution for circular failure slip, presented in Hunt (1986).

The embankment material is unlikely to be susceptible to seismic liquefaction, given its high fines content and well-compacted state. Seismic stability was assessed by considering

- What percent reduction in soil strength was required in order to achieve a post seismic FoS of unity;
- What coefficients of horizontal (kh) and vertical (kv=+/-0.5kh) acceleration were required to achieve a FoS of unity.

6.7.1 Static Stability

Results of static stability analyses are presented in Table 20 for target factor of safety (FoS) value of 1.50. These results indicate adequate stability even for the case of a part-saturated embankment.

Table 20: Static Stability Results

Analysis Method	Static FoS	Embankment Percentage Saturation
Hoek & Bray (1981)	1.50	50%
Michalowski (2002)	1.50	90%
Cousins (1978)	1.50	60%

6.7.2 Seismic Stability

Results of seismic stability analyses are presented in Table 21 for a target factor of safety (FoS) value of unity.

The strength reduction results point to a robust embankment even if marked strength reduction occurs post seismic shaking.

Simplistic pseudo-static assessment using kh and kv indicate adequate seismic stability. The peak ground acceleration (PGA) for the site is <0.06g for the Operating Basis Earthquake (OBE) and Maximum Design Earthquake (MDE) events. The kh and kv for FoS of unity are 0.10g and +/- 0.055g respectively.

Table 21: Seismic Stability Results

Assessment Method	Seismic FoS	Result
Strength Reduction	1.0	30% reduction in c' and ϕ'
	1.0	100% reduction in c' No reduction in ϕ'
Lateral Acceleration	1.0	k_h=0.10g k_v=+/- 0.5k_h

7. General Geotechnical Issues

7.1 Retaining Structures

Soil loads on retaining structures should be based on Rankine theory and may be calculated in accordance with the procedure outlined by Duncan and Seed (Ref. 10). Where backfill comprises select sand gravel with less than 15 % fines, the following parameters are considered applicable:

- Effective angle of friction (ø') = 36°.
- Cohesion (c') = 0 kPa.
- Angle of repose (β max.) = 35°.
- Bulk Unit Weight (γ) = 20 kN/m^3.

Rankine earth pressure coefficients of Ka (active) = 0.26 and Kp (passive) = 3.85 are recommended assuming that the retaining wall is vertical and sufficiently flexible, the ground behind the wall is horizontal, and zero wall friction develops. Ko (at rest) is dependent on the degree of compaction near the retaining structure. Assuming controlled backfill conditions in which heavy compaction equipment does not traffic adjacent to the retaining wall and hand compaction is undertaken in these areas, a value of 0.5 may be assumed. A higher degree of compaction could result in values of Ko of between 2.0 and a maximum value of Kp.

These parameters will vary depending upon the type of backfill material and should be reviewed on a case by case basis.

In order to provide adequate drainage and minimise lateral earth pressures it is recommended that a granular backfill material with the following properties be placed within 3m of retaining structures:

- Maximum fines content (% passing 0.075 mm) 15 %.
- Maximum particle size 50 mm.
- Minimum compaction of 92 % of modified maximum dry density (AS1289.5.2) at
- a moisture content of -3 % to +1 % of optimum moisture content.

Adequate drainage must be provided to ensure that water does not collect behind the walls. As an alternative, a geotextile drainage blanket may be installed down the back face of retaining structures, draining to a toe drain at the base of the wall. In either case, and regardless of the backfill material, reduced and careful compaction adjacent to the retaining structures, together with adequate drainage, is required to control excessive earth and hydrostatic pressures.

7.2 Earthworks

In general, subgrade preparation, road base and structural fill should be compacted to 95 % of Maximum Modified Dry Density (MMDD) at +/- 3 % of Optimum Moisture Content (OMC) in 300 mm layers with a 13-15 Tonne vibrating pad foot roller.

Additionally, depending on the moisture content of the materials, moisture conditioning (i.e. dry or wetting of the materials) may be required.

7.3 Excavatability

The test pits across the site were excavated with a Backhoe and as such most test pits were refused in the surficial soil and alluvium at depths of between 1.3 m and 2.8 m below ground level.

It is assumed that the alluvium can be excavated without the need for blasting. We expect that a dozer or excavator (D9N tracked dozer with single tine, 30 tonne excavator with single tooth ripping tine or similar) may be used in the excavation of the surficial material.

7.4 Heave Potential

There are no indications that the alluvial soils (the particle size distributions of which are clay/silt, sand and gravel-dominated) have significant heave potential.

7.5 Collapsing Soils

The alluvial soils are generally medium dense as a minimum, and often medium dense to dense. In general, the alluvial soils and duricrust are not expected to be prone to collapse.

9. Conclusions and Recommendations

9.1 General

Based on the investigations and analyses, it has been established that it is feasible to design and construct the TSF at the proposed location.

9.2 Ground Conditions

Typical ground conditions (from surface down) encountered during the investigation of the site are summarised in Table 22, below.

Table 22: Summary of Typical Sub-Surface Profile at TSF Site

Location	Description
GL – 0.2 m	Sandy SILT [ML], soft, non-plastic, brown-grey with gravel, dry, contains roots and organics.
0.2 m – 0.6 m	Sandy SILT [ML], soft, non-plastic, pale-brown with gravel, dry, transition phase between topsoil and red-brown horizon. Loose and unconsolidated material.
0.6 m – 1.2 m	Sandy SILT [ML], stiff, non-plastic, red-brown with gravel, dry, locally very conglomeritic with occasional lateritic texture.
1.2 m – 3. 0m	Sandy SILT [ML], stiff, non-plastic, white-grey, mottled red, dry, contains quartz cobbles (excavated as rock - SILTSTONE).

- It is unlikely that significant groundwater will be encountered during construction. Where encountered, seepage rates are expected to be low due to the fine grain size of in situ soils.
- Prior to commencing earthworks, the upper 150 mm to 300 mm thick topsoil layer should be removed and stockpiled. The near surface sandy GRAVEL is suitable for re-use as general and select fill.

9.3 TSF Foundation Design

It is feasible to design and construct the TSF at the proposed location. Stability analyses indicated that the minimum global factor of safely was above 1.5 and the likelihood of large scale failure under normal operating condition is considered low.

As a result of the investigation the following recommendations can be made:

- Approximately 150 mm – 300 mm of topsoil will need to be stripped from the TSF footprint, and stockpiled at designated locations along the alignment;
- A drainage layer is essential behind all retaining structures to reduce water pressures. This could comprise a geotextile blanket or a clean sand/gravel, free of deleterious material, with a fines content below 15 %.

11. Limitations

MHA Geotechnical (MHA) has prepared this report for the development of the Tailings Storage Facility (TSF) at ACH Minerals' Ravensthorpe Gold Project in accordance with MHA's proposal dated the 5th of November 2017. This report is provided for the exclusive use of ACH Minerals Pty Ltd and their consultants for this project only and for the purposes as described in the report. Any party so relying upon this report beyond its exclusive use and purpose as stated above, and without the express written consent of MHA, does so entirely at its own risk and without recourse to MHA for any loss or damage. In preparing this report MHA has necessarily relied upon information provided by the client and/or their agents.

The results provided in the report are indicative of the sub-surface conditions on the site only at the specific sampling and/or testing locations, and then only to the depths investigated and at the time the work was carried out. Sub-surface conditions can change abruptly due to variable geological processes and also as a result of human influences. Such changes may occur after MHA's field testing has been completed.

MHA's advice is based upon the conditions encountered during this investigation. The accuracy of the advice provided by MHA in this report may be affected by undetected variations in ground conditions across the site between and beyond the sampling and/or testing locations. The advice may also be limited by budget constraints imposed by others or by site accessibility.

This report must be read in conjunction with all of the attached and should be kept in its entirety without separation of individual pages or sections. MHA cannot be held responsible for interpretations or conclusions made by others unless they are supported by an expressed statement, interpretation, outcome or conclusion stated in this report.

This report, or sections from this report, should not be used as part of a specification for a project, without review and agreement by MHA. This is because this report has been written as advice and opinion rather than instructions for construction.

The contents of this report do not constitute formal design components such as are required, by the Health and Safety Legislation and Regulations, to be included in a Safety Report specifying the hazards likely to be encountered during construction and the controls required to mitigate risk. This design process requires risk assessment to be undertaken, with such assessment being dependent upon factors relating to likelihood of occurrence and consequences of damage to property and to life. This, in turn, requires project data and analysis presently beyond the knowledge and project role respectively of MHA.

MHA may be able, however, to assist the client in carrying out a risk assessment of potential hazards contained in this report, as an extension to the current scope of works, if so requested, and provided that suitable additional information is made available to MHA.

Any such risk assessment would, however, be necessarily restricted to the geotechnical components set out in this report and to their application by the project designers to project design, construction, maintenance and demolition.

Should you have any questions regarding the content of this report, please do not hesitate to contact us directly.

Sincerely,

For and on behalf of MHA Geotechnical,

Mitch Hanger

Director

Principal Geotechnical Engineer

BEng Civil (Hons) MIEAust

13. References

1. Giardini, D. Et al, *Global Seismic Hazard Assessment Program* – Global Seismic Hazard Map, 1999.
2. United States Geology Survey (USGS), *World Data Centre for Seismology*: Earthquake Data
3. Australian Standards. AS1289.0–1991, Methods of testing soils for engineering purposes.
4. BS 1377: Part 9:1990, Methods of test for soils for civil engineering purposes.
5. Australian Standards. AS1726–1993, *Geotechnical Site Investigations*.
6. Bowles, J. (1988). *Foundation Analysis and Design*, 4th Edition. McGraw Hill International Edition.
7. Harr (1966). Foundations of Theoretical Soil Mechanics. McGraw – Hill, New York.
8. Lee, White and Ingles (1983). *Geotechnical Engineering*. Pitman Publishing, Melbourne.
9. Ahlvin and Ulery (1962). Tabulated Values for Determining the Complete Pattern of Stresses, Strains and Deflections beneath a Uniform Circular Load on a Homogeneous Half Space. Highway Research Bulletin No 342.
10. Duncan, James M., and Seed, Raymond B., "Compaction-Induced Earth Pressures under Ko Conditions", Journal of Geotechnical Engineering, Volume 112, No. 1, January 1986, pp 1 – 22.
11. Australian Standards. AS1170.4-1993, Minimum Design Loads on Structures. Part 4: Earthquake Loads.

Appendix B

Test Pit Logs and Photographs

Sampling Methods

Sampling

Sampling is carried out during drilling or test pitting to allow engineering examination (and laboratory testing where required) of the soil or rock.

Disturbed samples taken during drilling provide information on colour, type, inclusions and, depending upon the degree of disturbance, some information on strength and structure.

Undisturbed samples are taken by pushing a thin-walled sample tube into the soil and withdrawing it to obtain a sample of the soil in a relatively undisturbed state. Such samples yield information on structure and strength, and are necessary for laboratory determination of shear strength and compressibility. Undisturbed sampling is generally effective only in cohesive soils.

Test Pits

Test pits are usually excavated with a backhoe or an excavator, allowing close examination of the in- situ soil if it is safe to enter into the pit. The depth of excavation is limited to about 3 m for a backhoe and up to 6 m for a large excavator. A potential disadvantage of this investigation method is the larger area of disturbance to the site.

Large Diameter Augers

Boreholes can be drilled using a rotating plate or short spiral auger, generally 300 mm or larger in diameter commonly mounted on a standard piling rig. The cuttings are returned to the surface at intervals (generally not more than 0.5 m) and are disturbed but usually unchanged in moisture content. Identification of soil strata is generally much more reliable than with continuous spiral flight augers, and is usually supplemented by occasional undisturbed tube samples.

Continuous Spiral Flight Augers

The borehole is advanced using 90-115 mm diameter continuous spiral flight augers which are withdrawn at intervals to allow sampling or in-situ testing. This is a relatively economical means of drilling in clays and sands above the water table. Samples are returned to the surface, or may be collected after withdrawal of the auger flights, but they are disturbed and may be mixed with soils from the sides of the hole. Information from the drilling (as distinct from specific sampling by SPTs or undisturbed samples) is of relatively low reliability, due to the remoulding, possible mixing or softening of samples by groundwater.

Non-core Rotary Drilling

The borehole is advanced using a rotary bit, with water or drilling mud being pumped down the drill rods and returned up the annulus, carrying the drill cuttings. Only major changes in stratification can be determined from the cuttings, together with some information from the rate of penetration. Where drilling mud is used this can mask the cuttings and reliable identification is only possible from separate sampling such as SPTs.

Continuous Core Drilling

A continuous core sample can be obtained using a diamond tipped core barrel, usually with a 50 mm internal diameter. Provided full core recovery is achieved (which is not always possible in weak rocks and granular soils), this technique provides a very reliable method of investigation.

Standard Penetration Tests

Standard penetration tests (SPT) are used as a means of estimating the density or strength of soils and also of obtaining a relatively undisturbed sample. The test procedure is described in Australian Standard 1289, Methods of Testing Soils for Engineering Purposes - Test 6.3.1.

The test is carried out in a borehole by driving a 50 mm diameter split sample tube under the impact of a 63 kg hammer with a free fall of 760 mm. It is normal for the tube to be driven in three successive 150 mm increments and the 'N' value is taken as the number of blows for the last 300 mm. In dense sands, very hard clays or weak rock, the full 450 mm penetration may not be practicable and the test is discontinued.

The test results are reported in the following form.

- In the case where full penetration is obtained with successive blow counts for each 150 mm of, say, 4, 6 and 7 as: 4,6,7 N=13
- In the case where the test is discontinued before the full penetration depth, say after 15 blows for the first 150 mm and 30 blows for the next 40 mm as: 15, 30/40 mm

The results of the SPT tests can be related empirically to the engineering properties of the soils.

Dynamic Cone Penetrometer Tests / Perth Sand Penetrometer Tests

Dynamic penetrometer tests (DCP or PSP) are carried out by driving a steel rod into the ground using a standard weight of hammer falling a specified distance. As the rod penetrates the soil the number of blows required to penetrate each successive 150 mm depth are recorded. Normally there is a depth limitation of 1.2 m, but this may be extended in certain conditions by the use of extension rods. Two types of penetrometer are commonly used.

- Perth sand penetrometer - a 16 mm diameter flat ended rod is driven using a 9 kg hammer dropping 600 mm (AS 1289, Test 6.3.3). This test was developed for testing the density of sands and is mainly used in granular soils and filling.
- Cone penetrometer - a 16 mm diameter rod with a 20 mm diameter cone end is driven using a 9 kg hammer dropping 510 mm (AS 1289, Test 6.3.2). This test was developed initially for pavement subgrade investigations, and correlations of the test results with California Bearing Ratio have been published by various road authorities.

Soil Descriptions

Description and Classification Methods

The methods of description and classification of soils and rocks used in this report are based on Australian Standard AS 1726, Geotechnical Site Investigations Code. In general, the descriptions include strength or density, colour, structure, soil or rock type and inclusions.

Soil Types

Soil types are described according to the predominant particle size, qualified by the grading of other particles present:

Type	Particle size (mm)
Boulder	>200
Cobble	63 - 200
Gravel	2.36 - 63
Sand	0.075 - 2.36
Silt	0.002 - 0.075
Clay	<0.002

The sand and gravel sizes can be further subdivided as follows:

Type	Particle size (mm)
Coarse gravel	20 - 63
Medium gravel	6 - 20
Fine gravel	2.36 - 6
Coarse sand	0.6 - 2.36
Medium sand	0.2 - 0.6
Fine sand	0.075 - 0.2

The proportions of secondary constituents of soils are described as:

Term	Proportion	Example
And	Specify	Clay (60%) and Sand (40%)
Adjective	20 - 35%	Sandy Clay
Slightly	12 - 20%	Slightly Sandy Clay
With some	5 - 12%	Clay with some sand
With a trace of	0 - 5%	Clay with a trace of sand

Definitions of grading terms used are:

- Well graded - a good representation of all particle sizes;
- Poorly graded - an excess or deficiency of particular sizes within the specified range;
- Uniformly graded - an excess of a particular particle size;
- Gap graded - a deficiency of a particular particle size with the range.

Cohesive Soils

Cohesive soils, such as clays, are classified on the basis of undrained shear strength. The strength may be measured by laboratory testing, or estimated by field tests or engineering examination. The strength terms are defined as follows:

Description	Abbreviation	Undrained shear strength (kPa)
Very soft	vs	<12
Soft	s	12 - 25
Firm	f	25 - 50
Stiff	st	50 - 100
Very stiff	vst	100 - 200
Hard	h	>200

Cohesionless Soils

Cohesionless soils, such as clean sands, are classified on the basis of relative density, generally from the results of standard penetration tests (SPT), cone penetration tests (CPT) or dynamic penetrometers (PSP). The relative density terms are given below:

Relative Density	Abbreviation	SPT N value	CPT qc value (MPa)
Very loose	vl	<4	<2
Loose	l	4 - 10	2 -5
Medium dense	md	10 - 30	5 - 15
Dense	d	30 - 50	15 - 25
Very dense	vd	>50	>25

Soil Origin

It is often difficult to accurately determine the origin of a soil. Soils can generally be classified as:

- Residual soil - derived from in-situ weathering of the underlying rock;
- Transported soils - formed somewhere else and transported by nature to the site; or
- Filling - moved by man.

Transported soils may be further subdivided into:

- Alluvium - river deposits
- Lacustrine - lake deposits
- Aeolian - wind deposits
- Littoral - beach deposits
- Estuarine - tidal river deposits
- Talus - scree or coarse colluvium
- Slopewash or Colluvium - transported downslope by gravity assisted by water. Often includes angular rock fragments and boulders.

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 01	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.685685
Logged By:	Harvey Morcom	Northing:	120.206652

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.2	TOPSOIL	SANDY SILT, [ML], soft, non-plastic, brown grey with gravel, dry.	Roots and organics	100		Bulk
					200		
					300		
					400		
					500		
0.2	0.75	ALLUVIUM	SANDY SILT, [ML], stiff, non-plastic, red brown, with gravel, dry.	Indurated, excavated as rock.	600		Bulk
					700		
					800		
					900		
					1000		
0.75	1.6	ALLUVIUM	SANDY SILT, [ML], stiff, non-plastic, white grey motled red orange	Siltstone, excavated as rock.	1100		Bulk
					1200		
					1300		
					1400		
					1500		
1.6	EOH	REFUSAL			1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m.

Groundwater recorded at ___________ m on the _____/______/_______.

Co-ordinate System:________________________, Zone: _______________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 01 – 1.6m to refusal, 3 soil horizons in a clockwise rotation starting top right; TOPSOIL, ALLUVIUM (red brown sandy silt) and ALLUVIUM (white mottled sandy silt).

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 02	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.6864494
Logged By:	Harvey Morcom	Northing:	120.2067161

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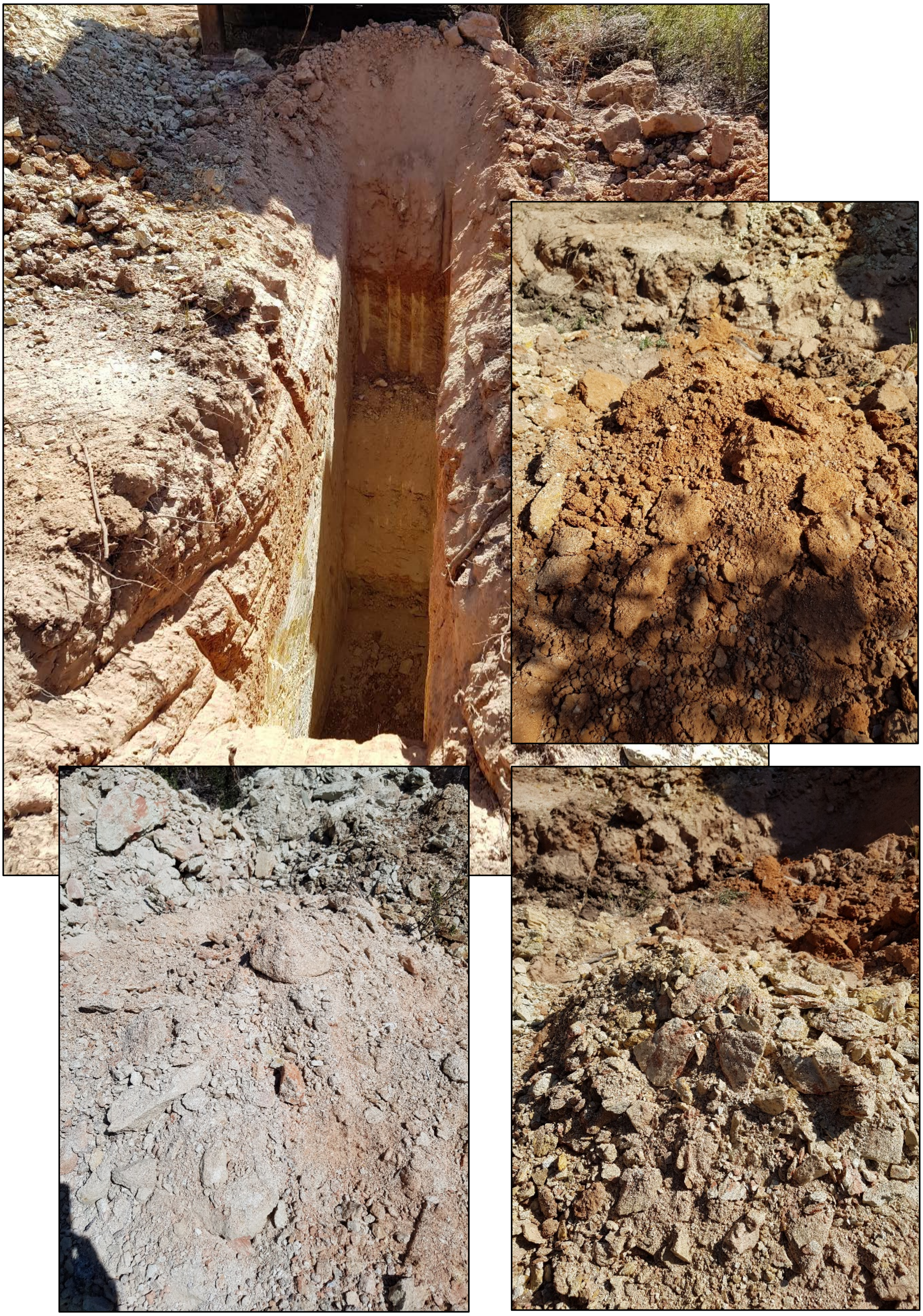
Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	SANDY SILT, [ML], soft, non-plastic, brown grey with gravel, dry.	Roots and organics	100		N/S
					200		
					300		
					400		
					500		
0.1	0.5	ALLUVIUM	GRAVELLY SILT, [ML], firm, non-plastic, red brown, with gravel, dry.	Indurated, excavated as rock.	600		Bulk
					700		
					800		
					900		
					1000		
0.5	1.1	ALLUVIUM	GRAVELLY SILT, [ML], firm, non-plastic, red brown, with gravel, dry.	Siltstone, excavated as rock.	1100		Bulk
					1200		
					1300		
					1400		
					1500		
1.1	2.7	ALLUVIUM	CLAY, [CL], soft, low-medium plasticity, grey white, dry.		1600		Bulk x2
					1700		
					1800		
					1900		
					2000		
2.7	EOH	TERMINATION			2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m.
Groundwater recorded at ____________ m on the _____/______/_______.
Co-ordinate System:_________________________, Zone: _________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	



Test Pit TP 02 – 2.7m to termination, 3 soil horizons in a clockwise rotation starting top right; TOPSOIL, ALLUVIUM (red brown sandy silt) and ALLUVIUM (white mottled sandy silt).

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 03	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.6873212
Logged By:	Harvey Morcom	Northing:	120.2068039

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	SANDY SILT, [ML], soft, non-plastic, brown grey with gravel, dry.	Roots and organics	100		N/S
					200		
					300		
					400		
					500		
0.1	0.7	ALLUVIUM	SILT, [ML], soft, non-plastic, grey white, with gravel and sand, dry to moist.	Indurated, excavated as rock.	600		Bulk
					700		
					800		
					900		
					1000		
0.7	3	ALLUVIUM	clayey SILT, [ML], very stiff, non-plastic, grey white motled red, dry.	Indurated, excavated as rock.	1100		Bulk x2
					1200		
					1300		
					1400		
					1500		
3	EOH	TERMINATION			1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m.

Groundwater recorded at ____________ m on the _____/______/_______.

Co-ordinate System:_________________________, Zone: _________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 03 – 3.0m to refusal, 2 soil horizons; TOPSOIL and ALLUVIUM (red brown sandy silt) and ALLUVIUM (white mottled sandy silt).

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 04	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.6883154
Logged By:	Harvey Morcom	Northing:	120.2069192

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.25	TOPSOIL	sandy gravelly SILT, [ML], firm, non-plastic, brown grey with gravel, dry.	Roots and organics	100		N/S
					200		
					300		
					400		
					500		
0.25	1	ALLUVIUM	SILT, [ML], firm, non-plastic, grey white red motling, with gravel and sand, dry.	Indurated, locally cemented.	600		Bulk
					700		
					800		
					900		
					1000		
1	3.1	ALLUVIUM	SILT, [ML], stiff, non-plastic, grey white, dry.	Indurated, excavated as siltstone.	1100		Bulk
					1200		
					1300		
					1400		
					1500		
3.1	EOH	TERMINATION			1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m.

Groundwater recorded at ___________ m on the _____/______/_______.

Co-ordinate System:________________________, Zone: ________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 04 – 3.1m terminated, 2 soil horizons; TOPSOIL and ALLUVIUM (red brown sandy silt) and ALLUVIUM (white mottled sandy silt).

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 05	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.688971
Logged By:	Harvey Morcom	Northing:	120.2069675

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	sandy gravelly SILT, [ML], firm, non-plastic, brown grey with gravel, dry.	Roots and organics	100		N/S
					200		
					300		
					400		
					500		
0.1	0.8	ALLUVIUM	SILT, [ML], firm, non-plastic, light brown, trace gravel with sand, dry.	Gravelly / sandy	600		Bulk
					700		
					800		
					900		
					1000		
0.8	1.8	ALLUVIUM	gravelly SILT, [ML], firm, low plasticity, red brown, trace gravel, dry.	Indurated, excavated as rock.	1100		Bulk
					1200		
					1300		
					1400		
					1500		
1.8	2.9	ALLUVIUM	SILT, [ML], stiff, non-plastic, grey white, dry.		1600		Bulk
					1700		
					1800		
					1900		
					2000		
2.9	EOH	TERMINATION			2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m.

Groundwater recorded at ____________ m on the _____/______/_______.

Co-ordinate System:_________________________, Zone: _________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 05 – 2.9m terminated, 4 soil horizons; TOPSOIL and 3 grades of ALLUVIUM; light brown silt, red brown gravelly silt and grey white sandy silt.

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 07	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.6889939
Logged By:	Harvey Morcom	Northing:	120.2084628

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	sandy gravelly SILT, [ML], firm, non-plastic, brown grey with gravel, dry.	Roots and organics	100		N/S
					200		
					300		
					400		
					500		
0.1	0.6	ALLUVIUM	SILT, [ML], soft, non-plastic, grey, with gravel and sand, dry.	roots and organics, gravelly / sandy, loose unconsolidated soil	600		Bulk
					700		
					800		
					900		
					1000		
0.6	2.7	ALLUVIUM	SILT, [ML], stiff, non-plastic, grey white, dry.	Indurated, excavated as siltstone.	1100		Bulk
					1200		
					1300		
					1400		
					1500		
2.7	EOH	TERMINATION			1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m.

Groundwater recorded at ___________ m on the _____/______/_______.

Co-ordinate System:________________________, Zone: ________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 06 – 2.7m terminated, 3 soil horizons; TOPSOIL and 2 grades of ALLUVIUM; predominantly composed of the grey white siltstone, however with darker grey gravelly material.

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 07	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.6889501
Logged By:	Harvey Morcom	Northing:	120.2091173

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	sandy gravelly SILT, [ML], firm, non-plastic, brown grey with gravel, dry.	Roots and organics.	100		N/S
					200		
					300		
					400		
					500		
0.1	0.2	ALLUVIUM	SILT, [ML], soft, non-plastic, dark grey, with sand, dry.	Gravel clasts, loose, unconsolidated.	600		Bulk
					700		
					800		
					900		
					1000		
0.2	2.5	ALLUVIUM	SILT, [ML], stiff, non-plastic, white, dry to moist.	Indurated, excavated as siltstone.	1100		Bulk x2
					1200		
					1300		
					1400		
					1500		
2.5	EOH	TERMINATION			1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to ____________ m.

Groundwater recorded at ___________ m on the _____/______/______.

Co-ordinate System:________________________, Zone: ________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 07 – 2.5m terminated, 3 soil horizons; TOPSOIL and 2 grades of ALLUVIUM; 0.2m of dark grey surficial top soil blending into the typical white siltstone.

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 08	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.6889116
Logged By:	Harvey Morcom	Northing:	120.2096444

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	sandy gravelly SILT, [ML], firm, non-plastic, brown grey with gravel, dry.	Roots and organics.	100		N/S
					200		
					300		
					400		
					500		
0.1	0.4	ALLUVIUM	sandy SILT, [ML], soft, non-plastic, grey, with sand, dry.	Unconsolidated.	600		Bulk
					700		
					800		
					900		
					1000		
0.4	2.2	ALLUVIUM	SILT, [ML], stiff, non-plastic, white, dry to moist.	Indurated, excavated as siltstone.	1100		Bulk
					1200		
					1300		
					1400		
					1500		
2.2	EOH	TERMINATION			1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m.

Groundwater recorded at ____________ m on the _____/______/_______.

Co-ordinate System:_________________________, Zone: _________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 08 – 2.2m terminated, 3 soil horizons; TOPSOIL and 2 grades of ALLUVIUM; 0.4m of dark grey surficial top soil blending into the white siltstone.

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017	
Test Pit ID:	TP 09	Date Finished:	20/11/2017	
Contractor:	Gary	Bucket Width:	0.55m	
Machine:	JCB	Easting:	-33.6888324	
Logged By:	Harvey Morcom	Northing:	120.2104839	

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	sandy gravelly SILT, [ML], firm, non-plastic, brown grey with gravel, dry.	Roots and organics.	100		N/S
					200		
					300		
					400		
					500		
0.1	0.2	ALLUVIUM	sandy SILT, [ML], firm-stiff, non-plastic, grey, with sand, dry.	Quartz clasts, gravelly.	600		Bulk
					700		
					800		
					900		
					1000		
0.2	2.2	ALLUVIUM	SILT, [ML], stiff, non-plastic, white, dry to moist.	Indurated, excavated as siltstone.	1100		Bulk x2
					1200		
					1300		
					1400		
					1500		
2.2	EOH	TERMINATION			1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m.
Groundwater recorded at ____________ m on the _____/______/_______.
Co-ordinate System:__________________________, Zone: _________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
CONCRETE	PEAT	OH	very soft	non-plastic	orange	silt	dry to moist
BITUMEN	CLAY	OL	soft	low plasticity	yellow	sand	moist
FILL	SILT	CH	firm	low - medium	brown	gravel	wet
BASSENDEAN SAND	SAND	CL	stiff	medium plasticity	purple	cobbles	moist to wet
SAND FROM TAMALA LST	GRAVEL	MH	very stiff	medium to high	green	OM	saturated
TAMALA LST	COBBLES	ML	hard	high plasticity	white	BR	
GUILDFORD FORMATION	BOULDERS	SW	Coarse Grain:	Coarse Grain:	cream	Fines:	
ALLUVIUM	Scondary:	SP	very loose	Fine	grey	<=15% "Trace"	
COLLUVIUM	clayey	SC	loose	Medium	black	15-30% "With"	
AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	

Test pit TP 09 – 2.2m terminated, 3 soil horizons. TOPSOIL and 2 grades of ALLUVIUM; grey surficial top soil blending into the white siltstone.

TEST PIT LOG

Job No:	P02-17	Date Started:	20/11/2017
Test Pit ID:	TP 10	Date Finished:	20/11/2017
Contractor:	Gary	Bucket Width:	0.55m
Machine:	JCB	Easting:	-33.6888137
Logged By:	Harvey Morcom	Northing:	120.2109811

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Depths (From)	Depths (To)	Main material	Material Description	Comments	DCP Depth (mm)	DCP Blows/100m	Laboratory Samples
0	0.1	TOPSOIL	sandy gravelly SILT, [ML], firm, non-plastic, brown grey with gravel, dry.	Roots and organics.	100		N/S
					200		
					300		
					400		
					500		
0.1	2.5	ALLUVIUM	SILT, [ML], stiff to very stiff, non-plastic, white orange mottling, dry to moist.	Clayey silt, less granular, excavated as rock.	600		Bulk X2
					700		
					800		
					900		
					1000		
2.5	EOH	TERMINATION			1100		
					1200		
					1300		
					1400		
					1500		
					1600		
					1700		
					1800		
					1900		
					2000		
					2100		
					2200		
					2300		
					2400		
					2500		
					2600		
					2700		
					2800		
					2900		
					3000		

NOTES AND COMMENTS

Many small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m and few small (1 - 2 mm) / medium (2 - 10 mm) / large (>10 mm) roots to _____________ m.

Groundwater recorded at ____________ m on the _____/______/_______.

Co-ordinate System:_________________________, Zone: _________________________.

Origin	Soil Name	Group	Consistency	Plasticity/Grain size	Colour	With/Trace	Moisture
TOPSOIL	Primary	Pt	Fine Grain:	Fine grain:	red	clay	dry
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AEOLIAN	silty	SM	medium dense	Coarse Grain:	blue	>30% "Secondary"	
SWAMP DEPOSIT	sandy	GW	dense	Additional:	Additional:	Coarse:	
LATERITE	gravelly	GP	very dense	Unifor, gap graded, poorly graded. Rounded, sub rounded, sub angular, angular, flaky, platy	Can be modified using pale, dark and motled	<=5% "Trace"	
	cobbly	GC				5-12% "With"	
	bouldery	GM				>12% "Secondary"	