MMG ROSEBERY MINE

Future Tailings Storage Feasibility Study

2/5 Dam TSF Feasibility Design Report

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**Signatures**

- **Author 1**
  - MARK DILLON
- **Reviewer 1**
  - KEITH SEDDON
- **Author 2**
  - ARUN MUHUNTHAN
- **Reviewer 2**
  - BEHROOZ.G.NEMJAD

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ATC Williams Pty Ltd
ABN 64 005 931 288
Melbourne T: +61 3 8587 0900  Perth T: +61 8 9355 8700  Brisbane T: +617 3352 7222
www.atcwilliams.com.au

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EXECUTIVE SUMMARY

MMG Limited (Rosebery Mine) requires a new Tailings Storage Facility (TSF) to store tailings beyond 2017 when the current facility (Bobadil TSF) is filled to its design capacity for the remaining Life of Mine (LoM). This report presents the design of the new facility.

The new TSF will be constructed within the current 1/2/5 Dam area and will encompass the existing structures as shown on Figure 6.1.

The facility has been designed as a 2 stage structure, with Stage 1 providing storage for 3 Mt and Stage 2 providing for a further 2 Mt. A nominal tailings production rate of 800,000 tpa has been adopted for the design which indicates a life for this new TSF of 6.25 years.

Studies completed to provide design parameters for the new TSF include:

- A geotechnical investigation of the area which consisted of the following activities:
  - Drilling of 18 cored boreholes to depths of between 10 m and 20 m;
  - Insitu permeability test - packer tests and falling head permeability tests;
  - Excavation of 43 test pits to identify near surface conditions; and
  - Cone penetrometer testing (CPTu) - 5 CPTu’s on 5 Dam and 3 CPTu’s downstream of the existing 2 Dam.

- A suite of geotechnical tests and geochemical tests undertaken to identify soil and rock design parameters.

- Pipe Loop testwork to identify rheological parameters for mechanical design.

In addition to the above, previous hydrogeological studies completed by Coffey [Ref. 17] were reviewed to confirm the hydrogeological parameters determined from the packer and falling head tests completed.

Geochemical testing indicates that the tailings are potentially acid forming (PAF) and hence the design needs to incorporate elements to manage both current and potential acid mine drainage (AMD) associated with the tailings.

In addition the rehabilitated 1 Dam is known to have environmental issues associated with impacted groundwater. Impacted seepage is also known to be affecting the Stitt River downstream of the eastern embankment of the existing 5 Dam and downstream of the existing 2 Dam.

The development of the TSF therefore needs to address both geotechnical and geochemical criteria.

The new TSF will be constructed across 1 and 5 Dams and will also be formed as a downstream raise to the existing 2 Dam embankment. The tailings storage concept is relatively simple, although complex in nature due to past developments at the site. The site is considered a brownfield site due to previous tailings storage and waste storage activities. Foundation preparation will vary across the site in response to the current site conditions. The design will include:

- A western embankment, constructed across the old 1 Dam area;
- A northern embankment constructed against the downstream face of the existing 2 Dam;
An eastern embankment constructed across the eastern side of 5 Dam. The embankment will be constructed across tailings and on natural ground (towards the southern end). The northern and eastern embankments are continuous;

- Construction of a geosynthetic liner across 5 Dam and 1 Dam to address seepage concerns;
- Foundation improvement consisting of cement/bentonite cut-off walls and grouting of foundation rock;
- A seepage collection drain adjacent to the downstream toe of the eastern and northern embankments to manage near surface seepage;
- A seepage collection pond, to be constructed downstream of the northern embankment to capture seepage and pump it back into the TSF;
- A screening wall adjacent to the Murchison Highway to both provide a vegetation screen to the South Rosebery residents and to act as a diversion wall in the highly unlikely event of failure of the western embankment;
- A clean water diversion drain along the southern side of the TSF to reduce rainfall inflows to the TSF;
- A diversion drain and spillway between the western and northern embankments to direct the flood waters to the north of the site resulting from an unlikely spillway discharge during operations and from the flood resulting from highly unlikely failure of the western embankment;
- Access roads and pipelines;
- Construction of site infrastructure (wash down facilities, security fencing and site offices;
- Internal clearing of borrow areas; and
- Tree felling within the impoundment area of the TSF.

The embankments will be constructed from rockfill with a bituminous liner placed against the upstream face of the embankments. The liner will be joined to the proposed cover liners for 1 and 5 Dam and will be anchored into cut-off walls or grout curtains. Rockfill will be sourced from on-site quarries located at the diversion/spillway cut and the rock outcrops between the existing 2 and 5 Dams and from a proposed quarry located within the south east portion of the TSF (Stage 1). The Stage 2 quarry will be located within the footprint of the facility.

The setting of the TSF and the geochemistry of the tailings means that the TSF will have a HIGH Consequence Category when assessed in accordance with ANCOLD guidelines [Ref. 9].

Deposition will be via the sub-aqueous technique so as to minimise the onset of potential acidic conditions and to obviate dust issues from the tailings. The TSF will be operated with a nominal 2 m water cover which a stochastic water balance indicates will be achievable, with minor seasonal fluctuations. During the drier summer months the gravity decant will be set so as to maintain the 2 m water cover whilst in the winter months the water depth will fluctuate to around 2.2 m.

Tailings delivery and water return pipelines will be installed within the existing pipeline corridor between the Rosebery mill and the TSF.

The closure concept for the TSF is a permanent water cover. A stochastic water balance for closure concept indicates that a minimum permanent water cover of 2 m, which is considered appropriate to minimise oxidation of the tailings, is possible at the site. The Stage 2 spillway (which has been designed to pass the design storm event will remain operational during closure and will discharge clean water to the Stitt River.
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1 INTRODUCTION

This report presents the results of the Feasibility Design of a new Tailings Storage Facility (TSF) for the MMG Limited Rosebery Mine in Tasmania.

The new facility will incorporate the rehabilitated 1 Dam, and the presently decommissioned 2/5 Dam TSF and will comprise a new TSF and associated Seepage Collection Pond (SCP). The study has included the review of previous tailings storage options studies, investigation of the preferred site, analyses, design, costing and documentation for a two stage facility.

The locality of the Rosebery Mine is shown on Figure 1.1.

2 BACKGROUND

2.1 Previous Studies

MMG has been investigating options for future tailings storage at Rosebery for a number of years. The current Bobadil TSF will reach its design capacity in the first half of 2017, and as such a facility needs to be designed to cater for the balance of the Life of Mine (LOM) production.

Options studies for future tailings storage for the Rosebery Mine were initially completed by ATC Williams (ATCW) in 2006 and 2008. These options considered five sites and for mine lives of 10 years and 20 years post Bobadil closure. This study did not consider 2/5 Dam.

Concept and options studies associated with modifying the 2/5 Dam were completed by ATCW in 2008 and 2009. These studies were centred on closure and management of seepage at 2/5 Dam. In 2010 an Options Study specifically targeted at recommissioning and expanding the 2/5 Dam was completed. This study also included a cost estimate comparison of the 2/5 scheme with the preferred option from the 2008 study, the South Marionoak site.

A Prefeasibility Study (PFS) of the 2/5 Dam site was completed in 2014. This report considered the general layouts presented in the 2010 study but included thickened tailings discharge as an option. Two embankment cross-sections were considered, these being either a rockfill/low permeability zoned embankment or Geocomposite lined upstream face on a rockfill embankment. The most efficient arrangement was identified as being to deposit conventional tailings sub-aerially from a synthetically lined embankment constructed downstream of the current 2 Dam embankment.

Relevant reports produced by ATCW covering these previous studies include the following:

- January 2008, “A Preliminary Study into Future Tailings Storage Options Study Including South Marionoak Site”, [Ref. 2].
- April 2009, “Tailings Storage Facility - Feasibility Study”, [Ref. 3].
- June 2009, “Proposal for an Options study for recommissioning 2/5 Dams, Discussion and cost estimate” [Ref. 5].
- June 2010, “A Preliminary Study for Future Tailings Storage Options - 2/5 Dam Options Study” [Ref. 6].
- October 2014, 2/5 Dam: Pre-Feasibility Study for Future Tailings Storage Options [Ref. 7].
3 OPTIONS STUDY POST PFS

3.1 General

As mentioned in Section 2, the final selected future TSF site is an expansion of the historic 2/5 Dam and incorporating the 1 Dam area.

The embankment alignments and deposition methodology of selected option from the PFS were further assessed as part of this design. These further options are described in the following sections. It should be noted that the options were assessed iteratively rather than in isolation, i.e. the options were revisited after each component was completed to identify the impact on other aspects of the study.

3.2 Power line Trade-off Study

Subsequent to the PFS the interaction of the existing 132 KV transmission tower/line easement (HV) and the proposed embankment location was considered further. The aim of this trade-off study was to ascertain whether the cost of constructing a higher embankment, on a smaller footprint was more cost effective than constructing the design proposed during the PFS and relocating the power lines.

The trade-off study identified the preferred option is that reducing the footprint of the storage by:

- Moving the northern embankment such that the ultimate embankment toe is not within the 30 m power line exclusion zone and hence relocation of the HV power lines is not required;
- The alignment of the eastern embankment across 5 Dam was relocated to be aligned with the existing eastern embankment; and

In addition to the trade-off study, the Western embankment, located on the old 1 Dam has been moved in a westerly direction over the existing sports fields, closer to the Murchison Highway, thereby avoiding the waste fill area to the south west.

In order to maintain storage capacity, the overall embankment height was increased by 2 m compared to the PFS preferred option.

3.3 Deposition Methodology

It was identified that dust may be an issue for the community given the proximity of the TSF to the Rosebery township. A trade-off study was completed to assess options available to minimise dust whilst still meeting the mine’s objectives. Two options were considered, sub-aerial and sub-aqueous deposition.

The trade-off study considered:

- The impact on storage volume of a lower dry density of the tailings of 1.2 t/m³ for the sub-aqueous deposition versus 1.5 t/m³ for sub-aerial deposition;
- The potential for oxidation of the tailings;
- Dust; and
- Deposition;

The trade-off study identified that sub-aqueous deposition was preferred on the basis of dust and potential for oxidation of the tailings. In order to provide the same storage capacity it was...
identified that the embankment height would need to be increased by approximately 1 m when compared to the Power line trade-off study findings.

4 FEASIBILITY STUDY BATTERY LIMITS

The battery limits for this design are:

- Tailings pipes and pumps ex concentrator;
- Return water from the TSF to the Effluent Treatment plant (ETP); and
- All civil design associated with the embankments and water management.

The mechanical aspect of the design are not considered in detail within this report and will be reported separately, however a summary of these aspects is provided to demonstrate the ability of the design to both receive tailings and to remove water from the system.

5 STUDY PARAMETERS

5.1 Overview

Since completion of the Prefeasibility study, the criteria and assumptions for the detailed design of the TSF have evolved in stages. In January 2015 it became apparent that the required TSF capacity would be reduced as the LOM has been better defined.

In addition, geotechnical investigations carried out in early 2015 have been completed, which allows better definition of the foundation conditions.

5.2 Tailings Tonnage

The LOM data provided by MMG is summarised in Table 5.1.

<table>
<thead>
<tr>
<th>TABLE 5.1</th>
<th>LOM TAILINGS DATA</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tonnes milled</td>
<td>879,288</td>
</tr>
<tr>
<td>Total Concentrate Produced</td>
<td>203,540</td>
</tr>
<tr>
<td>Tailings Produced</td>
<td>675,748</td>
</tr>
</tbody>
</table>

Based on current estimates, the Bobadil TSF will reach capacity in early 2017, therefore the LOM suggests a required capacity for the 2/5 Dam TSF of tonnage of 2.97 Mt to 2021, allowing for all of the period 2017 to 2021.

In this Feasibility Study, and used within the trade-off study, the capacity of the TSF has been staged as a two stage facility with the following storage capacities:

- Stage 1 to cater for current 2017 - 2021 budget LOA of 3 million tonnes (Mt),
- Stage 2 to cater for a provisional additional 2 Mt.
5.3 Tailings Discharge Rate

For this Feasibility study an average tailings production rate of 800,000 tpa has been adopted. The tailings will be discharged at a solids concentration of 30% Cw (or a nominal pumping rate of 91 t/hr assuming 8,790 hours per year).

5.4 Consequence Category

In order to ensure regulatory compliance with Environmental Protection Agency (EPA) and Assessment Committee for Dam Construction (ACDC), a review of the Consequence Category assessment has been completed for the 2/5 Dam TSF. The assessment of the consequence of failure is a risk based process, undertaken in accordance with the requirements of the ANCOLD Guidelines [Ref. 8 and 9].

The location of population and infrastructure in the area downstream of the proposed facility has been assessed on the basis of the Rosebery (3637) Base Topographic 1:25,000 map sheet, LIDAR survey, recent aerial photographs of the area and a drive through the general area.

The western flank of the facility is located very close to the Murchison Highway and South Rosebery. The Stitt River Park is located to the north west of the western flank of the facility.

The Stitt River flows around the eastern and northern flanks of the facility. Along the eastern flank the Stitt River is located within 50 m of the embankment whilst to the north the river is some 50 m to 200 m from the northern embankment toe.

The Rosebery raw water supply pump station is located approximately 200 m north of the facility.

South Rosebery is very close to the western embankment. A preliminary Failure Impact Assessment (FIA) indicated that under a “sunny day failure” scenario that some houses within South Rosebery could be inundated. A detailed FIA (refer Section 9) has been completed in order to identify engineering measures required to protect the South Rosebery residences. The FIA indicates that in the case of the adopted design neither permanent residences within South Rosebery, nor downstream would be affected in the event of a dam failure originating from the western flank of the facility.

The relevant categories from References 8 and 9 have been conservatively assessed taking into consideration the proposed engineering measures adjacent to the western embankment. An assessment of the Severity of Damages and Losses is presented in Figure 5.1. The highest damage and loss severity level is Major. Population at Risk (PAR) has been conservatively assessed as PAR = \( >1 \text{ to } <10 \) on account of the proximity of the Murchison Highway and Stitt River Park. This results in a hazard category of “HIGH C”.

The Consequence Category of a dam dictates the level of safety as applied to the operation, maintenance and surveillance requirements, together with the required detail of engineering design and the recurrence intervals of rainfall events and earthquakes for flood capacity and seismic stability analyses.
5.5  Flood Management

5.5.1  Overview

The flood management design for the TSF has been based upon the ANCOLD guidelines [Ref. 8]. There are two components to flood management:

- Freeboard; and
- Emergency spillway capacity.

5.5.2  Freeboard

With respect to the freeboard design criteria, ANCOLD [Ref. 8] require either a:

- Semi-qualitative risk analyses; or
- Minimum allowances specified within the guideline.

For this design a stochastic water balance has been undertaken to assess the maximum operating level to which to which the freeboard requirements are applied. Section 13 presents the water balance whilst Section 14.2 presents the freeboard assessment. These two criteria determine the required crest elevation for given required storage capacity.

5.5.3  Emergency Spillway

With respect to the emergency spillway design criteria, ANCOLD [Ref. 8] require High Consequence Category facilities to be able to pass runoff from the critical duration storm with a minimum Average Return Interval (ARI) of 1 in 100,000 years (AEP 0.000001) plus an allowance for a 1:10 AEP wind wave run-up, or alternatively, the Probable Maximum Flood (PMF). For this design we have adopted the PMF event. The parameters adopted are presented in Section 5.9.

5.6  Seismicity

For the purposes of the seismic analysis of the embankments, in the first instance a deterministic, prescribed approach to design has been adopted, which is the method recognised as the most widely used to date.

Seismic activity, like rainfall, is more probable at some locations than others and thus it is described in terms of probability and recurrence interval. No site specific seismic study is available for 2/5 Dams. However, it is considered that the results of investigations completed for the nearby Bobadil TSF by Seismology Research Centre (SRC) in 2002 [Ref. 10] and by GHD in 2007 [Ref. 11] are the best estimates for this study.

The SRC report [Ref. 10] provides an estimate similar to that reported in the Australian Earthquake Hazard Map in 2012 [Ref. 12] whilst the GHD [Ref. 11] seismic report presents much higher seismic risks on account of the inclusion of local faults. Review of the GHD study indicates that slightly larger earthquakes have been considered compared to those reported by SRC. The GHD data presented is inconsistent with the 2012 issue of the Australian Earthquake Hazard map, hence the data provided in the 2002 SRC report has been adopted for this study.
The ANCOLD dam design guidelines [Refs. 8 and 13] define this approach to design, with two levels of earthquake motion as follows:

- **Operating Basis Earthquake**: the OBE is for the purposes of evaluating the serviceability of the dam, rather than its safety. It is an earthquake which could reasonably be expected to occur during the life of the dam, and should only result in minor, easily repairable damage. The dam and appurtenant structures should remain functional after the occurrence of earthquake shaking not exceeding the OBE.

The ANCOLD guidelines description of the OBE reports deterministically derived earthquakes with a typical ARI range of 1 in 200 to 1 in 1,000 years. For Rosebery, the OBE acceleration coefficient is based upon a 10 percent chance of exceedance in 100 years, which is closely equivalent to an ARI of 1 in 1,000 years.

- **Maximum Design Earthquake**: the MDE will produce the maximum level of ground motion for which the dam should be designed or analysed. It is a minimum requirement that the impounding capacity of the dam be maintained when subjected to that seismic load. The ANCOLD guidelines summarise MDE deterministic, prescribed design methods used by various international dam authorities. From this summary, accepted practice for High Hazard dams is typically to adopt a probabilistic ARI of 1 in 10,000 years.

The horizontal peak ground acceleration (PGA) adopted from the SRC report [Ref. 10] for both the OBE and MDE events are as presented in Table 5.2, where ‘g’ refers to the acceleration due to gravity.

### Table 5.2
**SUMMARY OF SEISMIC DESIGN CRITERIA**

<table>
<thead>
<tr>
<th>Design Event</th>
<th>OBE</th>
<th>MDE</th>
</tr>
</thead>
<tbody>
<tr>
<td>Average Recurrence Interval</td>
<td>1 in 1,000 yrs</td>
<td>1 in 10,000 yrs</td>
</tr>
<tr>
<td>Peak Ground Acceleration</td>
<td>0.09 g</td>
<td>0.22 g</td>
</tr>
</tbody>
</table>

#### 5.7 Embankment Stability

Embankment stability has been assessed using the following minimum factors of safety (FS):

- Static loading - FS ≥ 1.5
- Static loading, end of construction - FS ≥ 1.3
- Seismic loading (serviceability) - FS ≥ 1.05
- Seismic loading (safety) - FS ≥ 1.0
- Post Liquefaction - FS ≥ 1.1

#### 5.8 Climate Data

##### 5.8.1 Rainfall

A 100 year rainfall record was developed based on the available rainfall records in the region. Data was sourced from the Bureau of Meteorology and recent site records.
There is only intermittent historical rainfall data available for Rosebery. As the available data was not continuous and did not cover all years, the Waratah weather station data was used as the base case and was modified through development of statistical correlations with the Rosebery (97011) weather station. A summary of the development of this modified Rainfall Database is presented in Table 5.3.

### TABLE 5.3
RAINFALL DATABASE

<table>
<thead>
<tr>
<th>Date</th>
<th>Weather Station / Factored Weather Station</th>
</tr>
</thead>
<tbody>
<tr>
<td>From</td>
<td>To</td>
</tr>
<tr>
<td>1/01/1908</td>
<td>31/12/1912</td>
</tr>
<tr>
<td>1/01/1912</td>
<td>30/06/1976</td>
</tr>
<tr>
<td>30/06/1976</td>
<td>31/08/1977</td>
</tr>
<tr>
<td>31/08/1977</td>
<td>31/05/1986</td>
</tr>
<tr>
<td>1/06/1986</td>
<td>30/06/1993</td>
</tr>
<tr>
<td>1/07/1993</td>
<td>30/09/1994</td>
</tr>
<tr>
<td>1/10/1994</td>
<td>31/08/1995</td>
</tr>
<tr>
<td>1/09/1995</td>
<td>1/02/2002</td>
</tr>
<tr>
<td>1/02/2002</td>
<td>31/12/2002</td>
</tr>
<tr>
<td>31/12/2002</td>
<td>31/07/2004</td>
</tr>
<tr>
<td>1/08/2004</td>
<td>31/12/2008</td>
</tr>
</tbody>
</table>

On this basis, the following precipitation data have been derived for the model:

- Average Annual Precipitation 1,985 mm/year
- Maximum Precipitation 2,941 mm/year (occurred in 1968)
- Minimum Precipitation 1,330 mm/year (occurred in 1950)

The data has been collated to assess the average monthly rainfall for this study. The average monthly rainfall is presented in Figure 5.2.

#### 5.8.2 Evaporation

The weather stations used to develop the rainfall database do not record evaporation. Pan evaporation data has been derived from BoM evaporation maps for Australia as average monthly evaporation. The derived average monthly pan evaporation is presented in Table 5.4.

### TABLE 5.4
AVERAGE MONTHLY EVAPORATION

<table>
<thead>
<tr>
<th>Month</th>
<th>Measured Pan Evaporation (mm/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>January</td>
<td>3.5</td>
</tr>
<tr>
<td>February</td>
<td>3.5</td>
</tr>
<tr>
<td>March</td>
<td>2.3</td>
</tr>
<tr>
<td>April</td>
<td>1.3</td>
</tr>
<tr>
<td>May</td>
<td>0.9</td>
</tr>
<tr>
<td>June</td>
<td>0.7</td>
</tr>
<tr>
<td>Month</td>
<td>Measured Pan Evaporation (mm/day)</td>
</tr>
<tr>
<td>----------</td>
<td>-----------------------------------</td>
</tr>
<tr>
<td>July</td>
<td>0.7</td>
</tr>
<tr>
<td>August</td>
<td>1.1</td>
</tr>
<tr>
<td>September</td>
<td>1.5</td>
</tr>
<tr>
<td>October</td>
<td>2.1</td>
</tr>
<tr>
<td>November</td>
<td>2.8</td>
</tr>
<tr>
<td>December</td>
<td>3.1</td>
</tr>
<tr>
<td>Annual Total</td>
<td>712 (mm/annum)</td>
</tr>
</tbody>
</table>

The average daily evaporation, expressed as a monthly total is also presented in Figure 5.2.

### 5.9 Design Storm Events

A study was completed by the Australian Government Bureau of Meteorology Hydrometeorological Advisory Services (BoM) in 2008 to provide an assessment of extreme rainfall for the South Marionoak catchment area [Ref. 14]. It is considered that the BOM study completed in 2008 is appropriate for this design as the South Marionoak site is located within the same valley, is at a similar elevation to, and is located approximately 3 km north west of the 2/5 Dam site. The study completed by BoM provides estimates for extreme storm events up to 72 hours. It can be assumed that the extreme rainfall data presented in Reference 14 is equivalent to the PMP.

The design storm events for the 2/5 Dam TSF site (refer Section 5.5) are:

- 1:100,000 ARI critical duration storm - 500 mm (in 3 hours)
- 1:10 AEP wind - 15 km/hr
- 1:10 AEP wind wave runup - 0.35 m

Diversion drains are located to the south of the TSF (clean water diversion) and seepage collection drains. These drains have been designed for a 1:10 AEP critical duration storm event in accordance with Australian Rainfall and Runoff (ARR) [Ref. 15]. This 1:10 AEP has been adopted on the basis that

An Intensity-Frequency-Duration (IFD) plot for the Rosebery site has been developed and is presented as Figure 5.3. The figure includes the PMP data from the BOM study [Ref. 14].

### 6 SITE CONDITIONS

#### 6.1 Site Description

The site is located approximately 1 km from Rosebery and adjacent to South Rosebery on the west coast of Tasmania.

The site currently consists of the following:

- 1 Dam - located along the western fringe of the proposed facility. This historical dam has been reclaimed and converted into a sports ground.
- 2 Dam - this covers the majority of the site and currently has a water cover. This dam currently forms part of the Rosebery sewerage scheme. A seepage collection dam and associated infrastructure is located downstream of the 2 Dam. An area downstream of the 2 Dam embankment is swampy.
• 5 Dam - this dam is located along the eastern side of the proposed TSF. It also has a water cover and is part of the Rosebery Sewerage scheme.
• There is a stormwater diversion drain that collects runoff from Mount Read (to the south of the site). The stormwater drain transfers runoff around the eastern side of 5 Dam and discharges runoff into the Stitt River downstream of the 2 Dam embankment.
• The Stitt River runs around the eastern and northern portions of the site. The river passes very close to the eastern embankment of 5 Dam and the western abutment of 2 Dam.
• There are high voltage power lines running across the site.

The site is generally covered by scrub in low lying areas and mature trees. The scrub is generally low lying in swampy areas downstream of the 2 Dam whilst tall timber is located on higher ground. Historical photographs indicate that the majority of the vegetation is regrowth.

A general arrangement plan is presented as Figure 6.1.

6.2 Geology

The Mineral Resources of Tasmania 1:25,000 scale Geological Map of Rosebery (Sheet 3637) indicates that the 2/5 Dam lies between a series of thrust faults, running in a north-south orientation as they pass the site.

To the north, west, south and beneath the western half of 2 Dam the governing geology comprises middle Cambrian age felsic volcanic and pyroclastic rock. Beneath, and to the immediate north and south of 5 Dam, the governing geology is the middle Cambrian (slightly pre-dating the felsic volcanic) volcaniclastic metasediments (mainly sandstone, with siltstone and mudstone). Both of these have been locally eroded and subsequently deposited over by the Quaternary glacial deposits. To the south of the 2/5 Dam complex, a deposit of middle Cambrian shale (younger than both the other Cambrian deposits) daylights.

6.3 Geotechnical Investigation

6.3.1 Overview

Geotechnical investigations of the site were conducted over the period December 2014 to March 2015. The report of the investigation is presented under separate cover as [Ref. 16].

6.3.2 Fieldwork

Field investigations have included borehole drilling, in-situ permeability testing, Cone Penetrometer Testing (CPTu), test pit excavations and bulk sampling for subsequent laboratory characterisation of potential construction materials. The main aims of the geotechnical investigations were to:

(i) Investigate the nature and condition of foundation materials beneath the east, north and west embankment footprints;
(ii) Estimate the approximate permeability of the near surface sediments and rock-mass foundations;
(iii) Assess topsoil and foundation stripping requirements, and detailed excavation conditions in associated civil works areas such as spillways and access road alignments; and
(iv) Locate suitable quarry areas for embankment rockfill materials.
A total of 18 boreholes have been drilled to depths ranging from 5 m to 28 m along the alignments of the proposed embankments and within potential rockfill quarry areas. Rock core was recovered from the boreholes, and in situ permeability tests conducted at regular intervals.

An extensive program of test pits were excavated to refusal or the extent of the excavator in the near-surface soils and weathered rock within the embankment footprints.

Investigation of the tailings within 5 Dam was conducted using CPTu techniques at five locations to identify the eastern wall foundation conditions. In addition, 3 CPTu tests were carried out at the downstream toe of the existing 2 Dam embankment (proposed northern wall).

Detailed reporting of the geotechnical investigation program and results may be found in [Ref. 16]. A brief summary of the major findings is presented in the following sections.

6.3.3 West Embankment Foundation Conditions (Existing 1 Dam)

The area encapsulating and closely surrounding the proposed Western Embankment presented the greatest variation in materials found within the site.

The embankment footprint covers part of the rehabilitated 1 Dam, which is now used as a cricket oval, natural ground to the south of the 1 Dam and rock foundations adjacent to the northern abutment. Historical data indicates that tailings were deposited within the area and then capped using waste rock from mine operations. Test pits confirmed the use of waste rock for rehabilitation. A pocket of landfill was identified between 2 Dam and 1 Dam but its extents could not be adequately defined. The landfill is located on the upstream side of the Western Embankment.

Boreholes drilled within 1 Dam identified the depth to tailings varying from approximately 4 m from the surface. The tailings thickness varies from approximately 0.5 m to 9 m. The depth to natural rock also varies, being at surface at the northern abutment and varying up to 18.8 m below surface through the rehabilitated 1 Dam area. These depths are consistent with the ground conditions identified by Coffey [Ref. 17]. The rock is moderately weathered to fresh and was found to have a coefficient of permeability in the range of $1 \times 10^{-6}$ m/s to $1 \times 10^{-7}$ m/s.

6.3.4 North Embankment Foundation Conditions (Downstream of Existing 2 Dam)

The near surface geology downstream of the existing 2 Dam embankment and within the area of the seepage collection pond consists of either fill material overlying silt and bedrock or surface exposure of rock.

Towards the western abutment the rock is moderately weathered and located at ground surface and to depths of 5 m. Approximately 100 m from the existing western abutment the depth to rock increases dramatically to about 18.5 m depth. Around the existing seepage collection pond the depth to rock was found to be between approximately 14.5 m and 18.5 m. Towards the eastern abutment of the existing 2 Dam embankment the depth to rock varies from between 0.2 m and 4.0 m. Rock permeability’s are consistent with those identified along the alignment of the proposed western embankment.

Where deep soils were encountered, these generally consisted of a mixture of fill consisting of old tailings and glacial deposits (silt to cobble size) overlying organic silts. The organic content of the silt decreases with depth becoming highly weather to moderately weather felsic rock at depth.
The maximum depth of silt was found to be approximately 17 m near the existing seepage collection pond. Insitu testing of the silt material, comprising SPT and CPTu testing indicates the material is loose.

The variation in fill materials and depths found within the current swamp area (between the eastern abutment and existing seepage collection pond, as marked on Figure 6.2, with depths of fill in the range of from 0.8 m to 4.0 and is underlain by either organic silt, silt or natural rock, in various weathering stages. This area has previously been subjected to a flow of tailings from a past failure. This is still visible from a layer of oxidised tailings on the surface.

6.3.5 East Embankment Foundation Conditions (Existing 5 Dam)

The proposed eastern embankment is an upstream raise on the existing 5 Dam. Borehole drilling, CPTu testing and test pit excavations were completed on the 5 Dam and the felsic rock outcrop between 2 and 5 dam and to the south of 5 Dam.

Test pits and boreholes indicated that the depth to felsic rock is relatively shallow and is generally within 0.5 m of the surface. The rock is moderately to slightly weather near surface and is fractured.

The CPTu testing indicated the tailings area soft and saturated to the full depth of the tailings.

6.3.6 Rock Mass Permeability

The Cambrian age felsic rock beneath the western and northern embankment locations and the Cambrian volcaniclastic metasediments beneath the eastern embankment location are considered to have an overall low permeability. The mechanism of seepage flow is through generally regularly spaced defects.

Constant head packer test results generally produced minor takes over the full depth range (5 m to 28 m). Approximate permeability estimates based on these takes are in the range $10^{-5}$ to $10^{-6}$ m/sec, with no discernible trend between boreholes.

6.3.7 Groundwater

Groundwater level around the site varied. Within rock, and in areas to in the southern portion of the site it is confined to the joints and defects within the rock mass.

Where deep deposits of glacial material are present in the southern portion of the site, the groundwater is generally within a couple of metres of the ground surface.

In the area of the western embankment, the groundwater is generally within 1 m of the ground surface.

6.3.8 Embankment Construction Materials

6.3.8.1 Rockfill

Based on the core obtained from the proposed quarry boreholes, the sites are expected to yield suitable rockfill for embankment construction.

Quarry production methods involving deep, cross-ripping of the upper profile, followed by carefully designed blasting should produce a well-graded rockfill with typical maximum particle sizes in the range 200 mm to 500 mm.
6.3.8.2 Granular Materials

Specific granular materials with particle size less than 50 mm nominal size are required for the liner protection. This material will be produced from quarried rockfill by crushing and screening.

The by-products from this process can be utilised elsewhere in the works. The oversize material will be blended with general rockfill for construction of the embankments.

6.3.9 Rockfill Geochemical Characterisation

Quarry areas identified for rockfill production are presented as Figure 6.3.

Samples of core from each of these areas, as well as surface sampling of rock exposes were recovered for geochemical characterisation. Characterisation of the proposed quarry material was carried out by Earth Systems (ES) [Ref. 18]. The testing completed indicates:

- Volcanic metasediments are non-acid forming (NAF)
- Shale, encountered at depth to the south of 5 dam is potentially acid forming (PAF).

In addition to the testing and analyses undertaken by ES, samples of rock core from the proposed spillway area were tested. The results indicate that this material is NAF.

Material that is reported as NAF will be geochemically suitable for use as rockfill which is the felsic volcanic and metasediments. The shale will not be suitable for use as construction material and hence the quarry design will need careful consideration to avoid these materials.

6.4 Hydrogeology

Coffey [Ref. 17] undertook a hydrogeological study of the existing site in 2012. Based on the Coffey finding, and supplemented with information gathered as part of the geotechnical investigation of the site by ATCW in 2015 [Ref. 16], the hydrogeological conditions of the site can be summarised as:

- The main hydrogeological units underlying the site are a composite of unconsolidated surface materials including river gravel, silt and clay and glacial deposits, as well as reworked and or mine affected materials such as landfill, dam walls and embankments. These units overly consolidated bedrock.
- Shallow groundwater exists near the 2-Dam embankment, present in the Quaternary aged river gravels.
- Groundwater flow direction and gradient were noted as likely to be highly dependent on groundwater recharge as well as changes in river height.
- The shallow hydrogeology of the area is complicated due to the heterogeneity of subsurface geology over short distance, the potential interaction and seepage from the dams, and interactions with the cut-off drain and Stitt River.

Based on the findings reported by Coffey [Ref. 17] and the results from the recent ATCW investigation [Ref. 16], the average permeability of the various hydrogeological units is:

- Average permeability of shallow aquifers (sediments) $= 1 \times 10^{-6}$ to $1 \times 10^{-8}$ m/s
- Average permeability of deeper aquifers (bedrock) $= 1 \times 10^{-5}$ to $1 \times 10^{-6}$ m/s
7 TAILINGS CHARACTERISATION

7.1 Physical Properties

7.1.1 General

Samples of ex-concentrator tailings were shipped to ATCW NATA accredited laboratory in Melbourne in February and April 2015. Test work included basic characterisation tests (particle density, plasticity, particle size distribution, pH, Electrical conductivity and Total dissolved Solids) and specialist tailings testing (initial and shrinkage limit density, consolidation, segregation threshold and rheology).

The test work carried out by ATCW was completed in three stages. The basic characterisation test work was carried out on samples in their as-received condition. Basic characterisation and specialist testing was carried out on segregated split samples. Rheology tests, consisting of pipe loop testing was carried out on the February and April samples to assess pipe friction losses at various solids concentrations.

The segregated split samples were prepared to assess separately the engineering characteristics of coarse and fine tailings. This was completed to identify the likely average settled dry density of the tailings for the case where they are deposited below water (sub-aqueous deposition).

A report on the laboratory test work conducted on the tailings by ATCW is presented in Appendix A. The following is a summary of the tailings physical properties resulting from the test work.

7.1.2 Tailings Classification

- Particle Specific Gravity (G) : 3.11 - 3.13 t/m$^3$
- Particle Size Distribution : $P_{80} \sim 100 \mu$m
- Atterberg Limits : Liquid Limit = 17 - 18
  Plasticity Index = 0 - 1
- Unified Soil Classification : Low plasticity Silt or sandy SILT (ML)
- Segregation Threshold : 59% solids

7.1.3 Tailings Density

Tailings will be deposited into the TSF using the sub-aqueous method. Discharge of this nature results in segregation of the tailings as the material settles through water. To estimate the average dry density of the deposited tailings the tailings were segregated into coarse and fine samples. These were then tested to identify the average parameters. The adopted tailings design density parameters are based on laboratory test results and are presented below:

- Coarse split tailings fraction sub-aqueous dry density - 1.56 t/m$^3$,
- Fine split tailings fraction sub-aqueous dry density - 0.96 t/m$^3$,
- Average sub-aqueous dry density - 1.26 t/m$^3$

A conservative sub-aqueous dry density of 1.25 t/m$^3$ has been adopted for design purposes.
In addition to the above testing, a sample of total tailings was tested to identify the shrinkage limit density. This is carried out to determine the maximum dry density that could result from evaporative drying. The shrinkage limit density was found to be 1.83 t/m$^3$.

### 7.2 Tailings Geochemistry

Geochemical test work on the tailings solids and liquor ex-concentrator has not specifically been carried out for this study. However, geochemical testing of samples recovered from the 5 Dam during the geotechnical works was conducted by Earth Systems (ES) for Pitt & Sherry (P&S) [Ref. 19]. In addition, geochemical assessment of tailings samples was conducted in 2014 by RGS Environmental as part of Bobadil closure studies [Ref. 20].

The ES test work provides geochemical information on the historical tailings within 5 Dam, whilst the RGS study results of recently deposited tailings within Bobadil are considered representative of the current tailings.

The results of the geochemical testwork indicate the tailings are potentially acid forming.

### 8 TSF PLANNING AND DESIGN

#### 8.1 Required TSF Capacity

Based upon a total tailings tonnage of 5 Mt at an overall insitu dry density of 1.25 t/m$^3$ at the end of filling, the required life of mine TSF capacity is 4.0 million m$^3$ (Mm$^3$)

#### 8.2 Tailings Storage Concept

The tailings storage concept is relatively simple, although in practice there are complexities in nature due to past developments at the site. The design makes use of the existing characteristics of the site. Overall site layout plans of the 2/5 Dam TSF are shown on Figure 8.1 (Stage 1 Layout) and Figure 8.2 (Stage 2 layout). Figures 8.3 to 8.5 provide typical sections and details.

Tailings will be pumped from the Rosebery Concentrator to the site via the existing pipeline corridor as shown on Figure 8.6. New pipelines will be installed for this purpose. Discharge will be via single point discharge from floating pipes extended into the facility. Discharge will only occur from a single point at any time. Deposition will continue at a specific point until a set tailings elevation is achieved, at which point discharge will be changed to the next floating pipeline. This process will be repeated until the impoundment stage is filled. The tailings elevation of the relatively uniform surface will remain nominally 2 m below the water surface across the facility, although it is likely that isolated low and high points will occur. Routine bathymetry surveys will need to be completed to identify these low areas so that targeted tailings deposition can occur to fill these areas towards the end of the life of the facility. The water depth will be maintained by adding stop logs to the decant so that the decant inlet is progressively raised in line with the increase in tailings elevation.

Decant water will be discharged from the TSF via a gravity decant located between the western and northern embankments as shown on Figure 8.1 and 8.2. The decant will consist of an inverted box culvert as shown on Figures 8.7 and 8.8 for Stage 1 and 2 respectively. Typical sections and details of the decant as presented on Figure 8.9. The decant will discharge water via gravity to the Effluent Treatment Plant (ETP) which is located within the processing area. The decant pipeline will be located within the same corridor as the tailings delivery pipeline as shown on Figure 8.6. Operation of the decant is discussed in Section 8.2.
Containment along the southern side of the TSF is almost entirely provided by topography. Containment along the northern, western and eastern sides will be provided by embankments, constructed in two stages. Stage 2 will be constructed using the downstream construction technique. The embankments will be constructed with rockfill sourced from on-site quarries, with an upstream geosynthetic lining system installed to minimise seepage from the system. The embankments will be constructed in two Stages, with Stage 1 construction to a crest elevation of RL 170 m and Stage 2 Crest at RL 173 m. The Stage 1 crests will be nominally 10 m higher than the existing 2 Dam embankment crest. Civil works required to construct the facility include:

- A western embankment, constructed across the old 1 Dam area;
- A northern embankment constructed against the downstream face of the existing 2 Dam;
- An eastern embankment constructed across the eastern side of 5 Dam. The embankment will be constructed across tailings and on natural round (towards the southern end). The northern and eastern embankments are continuous;
- Construction of a geosynthetic liner across 5 Dam and 1 Dam;
- Foundation improvement consisting of cement/bentonite cut-off walls and grouting of foundation rock;
- A seepage collection drain adjacent to the downstream toe of the eastern and northern embankments;
- A seepage collection pond, to be constructed downstream of the northern embankment;
- A screening wall adjacent to the Murchison Highway;
- A clean water diversion drain along the southern side of the TSF;
- A diversion drain and spillway between the western and northern embankments;
- Access roads and pipelines;
- Construction of site infrastructure (wash down facilities, security fencing and site offices);
- Internal clearing of borrow areas; and
- Tree felling within the impoundment area of the TSF.

The site is considered a brownfield site due to previous tailings storage and waste storage activities. Foundation preparation will vary across the site in response to the current site conditions.

The 1 Dam poses an environmental risk due to the nature of the rehabilitation works completed to date, this being the placement of waste rock over tailings. Upwelling of groundwater is currently managed via a drainage network that directs groundwater to the existing seepage collection pond located downstream of 2 Dam. The 1 Dam drainage system will be removed as part of the works so to manage impacted groundwater upwelling downstream of the western embankment a bituminous membrane will be buried approximately 2 m below ground surface across the 1 Dam. The membrane will be connected to grouted rock along the north, west and south sides of 1 Dam. A ballast of 2 m will be placed above the membrane to counter uplift due to groundwater. This membrane will be connected to the embankment lining system to provide an effective seal.

The western embankment also poses a risk to South Rosebery dwellings due to its close proximity. To protect the residences and other itinerants a screening wall will be formed adjacent to the Murchison Highway and a diversion drain cut through the hill between the western and northern embankments. Both the screening wall and diversion drain have been designed to safely discharge outflow from a breach from the western embankment. The screening wall will be constructed from rockfill, topsoiled and planted with trees, whilst the diversion drain will have a base width of 15 m. Erosion protection of the side slope and base of the screening wall and at the transition into the diversion channel will be constructed. The screening wall will be constructed above the 1 Dam lining system.
The northern embankment will be constructed as a downstream raise on the existing 2 Dam. To manage seepage water from the existing facility a bituminous liner will be installed between the existing embankment and the existing downstream raise and then be placed on the upstream face of the raised embankment. Foundation conditions vary along the length of the embankment. Topsoil and surficial soils will be excavated to provide a suitable foundation at the abutments. In these areas basement rock will be exposed. This rock will be grouted to a depth of about 10 m and the liner (sandwiched between the existing embankment batter and the downstream raise) will be connected to the top of the grout curtain. Where the depth of overburden is deeper, as described in Section 6.3, a cut-off wall will be installed to a depth of around 6 m, with the liner connected to the top of the cut-off.

The downstream face of the existing 2 Dam will be benched to allow intermediate anchorage of the liner so as not to overstress the material.

The eastern embankment will be constructed across the 5 Dam and onto natural ground towards the southern abutment. Construction of this embankment will need to be staged to allow the tailings to consolidate during construction. To manage this, vibrating wire piezometers will be installed within the tailings prior to construction to allow monitoring of pore pressure during the works. In this area the lining system will be anchored into the existing 5 Dam clay core and installed below the embankment raise and wrapped onto the upstream face. A liner will also be placed across the remaining exposed surface of 5 Dam to reduce seepage from the facility.

Where the embankment is constructed on natural ground, vegetation will be cleared and shallow surficial material removed to expose a rock foundation. The rock will be grouted to reduce seepage from the facility.

Rock will be sourced from quarries developed within the footprint of the facility and from the diversion channel between the northern and western embankments.

An emergency spillway will be constructed between the western and northern embankments, with its outlet discharging into the diversion.

A clean water diversion drain will be formed to the south of the facility. This drain will divert clean water runoff from Mount Read in an easterly direction and discharge into the Stitt River. The drain alignment and typical section are presented on Figure 8.11.

The proposed closure concept for the facility is a permanent water cover, although other closure forms will be explored during the operational phase of the facility.

8.3 Beach Slope

The tailings will be deposited sub-aqueously and hence it has been assumed that as the tailings fall through the column of water overlying the tailings they will settled to a relatively uniform overall level and hence a flat tailings beach has been adopted for the purposes of calculation of storage capacity.

8.4 Embankment Staging

The required TSF Embankment crest level is a function of the tailings elevation with time, the depth of water cover and freeboard. Construction will be conducted in two stages to minimise up-front capital costs.
General layouts of the TSF at the end of Stages 1 and 2 are shown in Figures 8.1 and 8.2 respectively. The Stage Capacity Curves (SCC) for tailings deposition, in terms of rate of rise, tailings level and volume as a function of tonnage, are depicted in Figure 8.12. Also shown is the TSF Embankment staging schedule.

The embankment staging and TSF capacity schedule will be as listed in Table 8.1.

<table>
<thead>
<tr>
<th>Stage</th>
<th>TSF Embankment Crest RL (m)</th>
<th>Capacity (1)</th>
<th>Filling Life (yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Volume (Mm³)</td>
<td>Tailings (Mt)</td>
<td>Stage</td>
</tr>
<tr>
<td>1</td>
<td>170.0</td>
<td>2.4</td>
<td>3.0</td>
</tr>
<tr>
<td>2</td>
<td>173.0</td>
<td>1.6</td>
<td>2.0</td>
</tr>
<tr>
<td>Total</td>
<td>4.0</td>
<td>5.0</td>
<td></td>
</tr>
</tbody>
</table>

(1) Tonnage-volume relationship based on ATCW dry density at end of filling of 1.25 t/m³. The capacity also assumes that the Stage 1 borrow area is predominately located within the Stage 1 footprint.

Initially the rate of rise is relatively high as the borrow areas are filled and whilst filling is limited to the existing 2 Dam area. Once the entire footprint is available for storage the rate of rise reduces to an average of around 1.6 m/year as shown in Figure 8.12. It should be noted that the stage capacity curve includes allowance for quarry areas within the footprint of the TSF and these quarry voids have been taken into consideration within the storage volume.

8.5 Geomembrane Sealing Systems

8.5.1 General

A Geosynthetic Sealing System (GSS) will be constructed as described in Section 8.2. The purpose of the GSS is to provide a low permeability facing to the rockfill embankments, separate new tailings and the existing 5 Dam tailings and to manage seepage concerns associated with 1 Dam.

With regard to the geosynthetics, the embankment GSS, 1 Dam and 5 Dam cover GSS will consist of a reinforced elastomeric bitumen geomembrane suitable for placement on embankments. These geomembranes are robust and have excellent UV stability and weathering properties necessary to survive for up to 40 years uncovered. These materials have a density > 1g/cc and hence will not float in water and are less susceptible to wind uplift than traditional polyethylene geomembranes.

8.5.2 Reinforced Elastomeric Bitumen Geomembrane

The selected membrane shall consist of either:

- Silplast TERANAP 531 TP elastomeric modified bituminous geomembrane, nominal thickness 4.6 mm; or
- Axter COLETANCHE ES 3 SBS elastomeric modified bituminous geomembrane, nominal thickness 4.8 mm.

On the upstream face of the embankment the GSS will be placed against a fine crushed rock produced via quarrying on-site. Where the GSS will be buried within the embankment and across 1 Dam it shall have a 300 g/m² either side to provide cushioning to the GSS.
The chemical durability of these products lends themselves to be suitable for the proposed lining works.

8.6 Anchorage and Crest Protection

Perimeter anchorage of the GSS will consist of:

- Installation at the toe of the embankments will into the cement bentonite cut-off wall or mechanical tie-down type on the abutments and where the rock foundation is grouted. Where the GSS is to be anchored at the cut-off walls, the material will be installed vertically to a depth of nominally 1 m prior to curing of the cut-off material or will be embedded in an anchor trench backfilled with a cement-bentonite mixture.

- At rock abutments and where the foundation has been grouted the installation will either be via the mechanical tie-down method of welding to a concrete plinth formed at the top of the grouted rock. The mechanical tie down method incorporates stainless steel batten strips bolted to the rock but it should be noted that any mechanical tie-down installation shall be protected against corrosion. The plinth will be nominally 2 m wide by 0.4 m deep. The plinth will be constructed of 50 MPa reinforced concrete, with 2.5 m rock anchor dowels at 2.0 m centres.

- At the crest, the GSS will be anchored within a trench of nominal dimensions 0.5 m wide by 0.5 m deep. The anchor trench will be backfilled with fine grained rock.

Figure 8.5 provides typical anchorage details.

8.7 Seepage Collection Pond

A new Seepage Collection Pond (SCP) will be constructed downstream of the northern embankment. The primary purpose of the pond is to capture minor seepage from the downstream toe of the eastern and northern embankments as well as runoff from the existing swamp area which is known to be degraded as a result of past tailings spills. The existing seepage collection pond pump station will be relocated to the new pond to allow pumping of water back into the TSF.

The pond will be formed primarily in cut with a low height embankment located along the northern flank of the facility. A GSS lining system will be installed within the basin to minimise seepage to the environment.

The pond has been sized to have sufficient capacity to store the 1:100, 24 hour AEP rainfall event from the contributing catchment area (between the crest and seepage collection drains) with a 0.85 yield factor. This has been adopted to provide storage in the event of power outage or pump failure. The 1:100, 24 hour AEP rainfall depth was determined from the IFD presented as Figure 5.3.

The storage volume of the SCP is 15,000 m$^3$.

The location of the SCP is shown on Figure 8.1, whilst typical sections are presented on Figure 8.5.

A stochastic water balance module has been developed within the TSF water balance to assess the spill risk of the SCP. The results are presented within Section 13.3.
9 FAILURE IMPACT ASSESSMENT

9.1 Overview

A dam break assessment was completed for the western embankment due to its proximity to the Murchison Highway and to the dwellings abutting the highway and Gepp Street in South Rosebery. The dam break also considered the downstream impacts along the Stitt River.

9.2 Methodology

The assessment has been based on the sub-aqueous deposition scenario. The adopted cross section used for the assessment is:

- Stage 2 crest elevation (RL 173.0 m);
- Embankment constructed from rock fill;
- 2 m freeboard;
- 2 m pond depth; and
- 7.5 m of soft, sub-aqueously deposited tailings.

LIDAR survey provided by MMG was used for topographical purposes. It is considered that this survey is a true representation of the topography.

The volume associated with a pond depth of 2 m is approximately 1.1 GL.

The assessment considers the worst case whereby the dam is at its maximum operating level at the time of dam failure and the dam breach is located at the northern abutment. The adopted breach mechanism is settlement of the crest leading to an overtopping event resulting in crest erosion and propagating progressive failure of the embankment.

The TUFLOW two-dimensional hydrological modelling software was used to identify the flood inundation area, flood depths and flow velocities. Each case considered was first modelled in AutoCAD to obtain the adjusted terrain, which was then used as the input file for TUFLOW.

Modeled water level results were obtained and compared to the base case to assess the effectiveness of the mitigation option. The number of flood affected properties were also identified for each option.

9.3 Assessment

9.3.1 Failure hydrograph

A failure hydrograph was initially prepared for the embankment breach. This hydrograph has been calculated using the above adopted parameters. Figure 9.1 provides the failure hydrograph.

The hydrograph shows the relative water and tailings elevations and outflow with time. The hydrograph indicates that the water and some tailings in the vicinity of the breach would flow for around 8 hours. The hydrograph also indicates a peak discharge of approximately 180 m$^3$/s.
9.3.2 Cases Considered

9.3.2.1 Base Case

The base case considered for the Failure Impact Assessment (FIA) was the scenario whereby no mitigation measures were put in place downstream of the western embankment. This assessment indicated that five (5) dwellings could be inundated and that the Stitt River Park would be impacted. The model indicated that there would be no major flood impacts within the Stitt River downstream of the facility, although the constriction of the Stitt River at the Stitt River falls resulted in flooding of the football ground. The inundation area, showing the impacted dwellings for the base case is presented as Figure 9.2.

The results of the base case model indicated an unacceptable number of affected dwellings and hence various mitigation measures were investigated to reduce the number of impacted dwellings to zero.

9.3.2.2 Mitigation Options

Various mitigation options were considered to reduce Population at Risk (PAR) to as low as reasonably possible. A discussion of options considered and the modelled outcomes is presented in Table 9.1. Various iterations for the configuration of the options were completed to assess the sensitivity of the models.

<table>
<thead>
<tr>
<th>Option</th>
<th>Description</th>
<th>Outcome</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Consideration of various configurations associated with widening of the Murchison Highway road cutting to the north of the dwellings by 2.5 m.</td>
<td>These models resulted in a lower flood height between the western embankment and the dwellings but did not reduce the number of impacted dwellings or the flooding of the Stitt River Park.</td>
</tr>
<tr>
<td>2</td>
<td>Construction of a screening wall along the eastern side of the Murchison Highway and widening of the road cutting to 10 m.</td>
<td>These models resulted in two dwellings being impacted and the Stitt River Park being flooded.</td>
</tr>
<tr>
<td>3</td>
<td>As per option 2 but with a 4.5 m drain excavated between the screening wall and the road cutting.</td>
<td>These models resulted in no dwellings being impacted but the Stitt river park remained flooded. As people routinely camp at the Park during the summer months’ people are still at risk. This option was considered not viable as the excavation of a 4.5 m deep drain would intercept impacted groundwater within the 1 Dam area.</td>
</tr>
<tr>
<td>4</td>
<td>As per option 2 but with widening of the road cutting by up to 30 m.</td>
<td>This option resulted in a similar outcome to that of Option 2 but with more of the flood passing down the eastern side of the Murchison Highway and reduced flooding of the Stitt River Park.</td>
</tr>
<tr>
<td>5</td>
<td>Construction of a screening wall and cutting a diversion drain with a base width of 15 m through the hill between the western and northern embankments.</td>
<td>This option resulted in the flood being directed away from the Murchison Highway, dwellings and Stitt River Park. The downstream flood near the football oval is unaffected. This is the preferred option.</td>
</tr>
</tbody>
</table>

9.3.2.3 Final Mitigation Model - Design Basis

Option 5 described in Table 9.1 has been adopted as the design basis for the FIA. A discussion of the screening wall and diversion drain is provided in Section 8.2. The layout of the screening wall,
diversion drain and associated infrastructure is presented on Figures 8.7 and 8.8 for Stage 1 and Stage 2 development respectively.

Figure 9.3 presents the flood inundation area for the Design Basis.

10 EMBANKMENT STABILITY

10.1 Overview

The governing principle is that the Stage 1, RL 170 m and Stage 2, RL 173 m embankments have been designed in such a manner that the integrity of the structure, with respect to stability under static and seismic loading conditions, is preserved.

Stability analyses for the embankments were conducted using the following loading cases:

- static, long term analyses (effective or undrained strength, as appropriate);
- seismic analyses; and
- Post liquefaction analyses.

In each case analysed, the maximum height section was considered. Three different cross sections were considered for the analyses and are shown as Sections 1 to 3 on Figure 8.3 and Figure 8.4.

The stability analyses were undertaken with the propriety software SLOPE/W [Ref. 21], utilising the GLE (general limit equilibrium) method, satisfying both force and moment equilibrium criteria.

10.2 Seismicity

The seismic design parameters are presented in Section 5.6.

10.3 Liquefaction Potential of Tailings

The liquefaction potential of the tailings foundations within 5 Dam and silt foundation at the downstream of 2 Dam were assessed on the basis of Cone Penetrometer Testing (CPTu) undertaken during the geotechnical investigations in February 2015 [Ref 16].

Whilst no CPTu testing was conducted in the area of 1 Dam, it has been assumed that the tailings conditions will be similar to those investigated at 5 Dam.

The results obtained in the above mentioned investigations and subsequent analyses of the liquefaction potential of the tailings and silt materials, as summarised below.

The triggering PGAs (assessed to be where FS\textsubscript{L} are < 1.1) are presented in Table 10.1.
TABLE 10.1
TRIGGERING PEAK GROUND ACCELERATION

<table>
<thead>
<tr>
<th>Material</th>
<th>Yield PGA</th>
<th>ARI</th>
</tr>
</thead>
<tbody>
<tr>
<td>Tailings</td>
<td>0.07 g (M = 6.5)</td>
<td>~1 in 500 yrs</td>
</tr>
<tr>
<td>Foundation Silt (Downstream of 2 Dam)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth 0 - -6.0 m</td>
<td>0.09 g (M = 6.5)</td>
<td>-1:1,000 yrs</td>
</tr>
<tr>
<td>Foundation Silt (Downstream of 2 Dam)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Depth -6.0 - -17.0 m</td>
<td>0.17 g (M = 6.5)</td>
<td>-1:5,000 yrs</td>
</tr>
</tbody>
</table>

The ANCOLD guideline on design of dams for earthquakes [Ref. 13] states that if liquefaction is expected within the storage, the liquefied tailings load case becomes the critical case, over OBE or MDE seismic stability. Nevertheless, although the tailings and foundation silts are susceptible to liquefaction, it is considered that the embankment materials themselves are not liquefiable, by nature of the materials used in construction. Therefore, we have considered both seismic load cases as well as post liquefaction in the analyses.

10.4 Pseudo Static Stability Methodology

Embarkment design procedures generally proceed as required from initial, simplified methods to more complex, rigorous methods. Hence the purpose of the pseudo-static analyses is to determine if more detailed embankment deformation analyses are required.

Pseudo-static analyses were conducted using the OBE (serviceability) and MDE (safety) loading cases specified in Section 5.6.

OBE pseudo-static analyses were conducted using conventional, peak strength properties and no reduction factor on the earthquake acceleration, in order to assess the safety and serviceability of the embankment when subjected to the operational design earthquake load. The pseudo-static method is considered a very conservative approach to seismic stability analysis.

MDE analyses were performed using the US Army Corps of Engineers (USCE) screening method. The USCE method is an internationally recognised screening tool for seismic instability, applicable to well-constructed embankments not susceptible to liquefaction. For the purpose of this section of the analyses we have used factored peak undrained strengths or shear strengths of the tailings.

The USCE method recommends use of a seismic coefficient equal to one-half of peak ground acceleration (PGA) using undrained strength for cohesive materials and drained conditions for free draining granular materials, with a 20 percent strength reduction in all materials to allow for strain weakening during the earthquake loading.

The design objective was for negligible predicted deformations under the MDE (safety) criteria. This determination was based upon USCE Screening resultant Factors of Safety being greater than 1.0. ANCOLD 1998 [Ref. 13] indicates an FS of 1.0 indicates deformations of less than 1 m along the nominal failure “plane”.
10.5 Material Design Parameters

10.5.1 Overview

The material properties used in the static and pseudo-static stability analyses are summarised in Tables 10.2 to 10.4 and a discussion of the derivation of the properties is presented in the subsequent sections. These properties were obtained from site investigations, laboratory tests, published literature, established correlations and previous experience, as explained in the following sections.

These parameters are used in the analysis of the following design cases:

- Drained or Effective Stress Parameters for use in analysis of long-term static conditions;
- Undrained Parameters for use in total stress analysis at end of construction and seismic loading (pseudo static); and
- Liquefied Strength Parameters for post liquefaction analysis.

The most critical variable in any effective stress analysis is the pore pressure within each zone of the embankment. These can generally be estimated or measured in-situ. However, excess pore pressure is generated in soils during an earthquake due to rapid change in load and is difficult to quantify. Total stress (undrained) parameters are independent of pore pressure and are used in Seismic analysis as it is anticipated that there would be little or no drainage occurring during such a short duration event.
<table>
<thead>
<tr>
<th>Material</th>
<th>Drained Strength Parameters</th>
<th>Undrained Strength Parameters (1)</th>
<th>Liquefied Strength Parameters</th>
<th>γ sat (kN/m³)</th>
<th>γ unsat (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>c' (kPa)</td>
<td>Φ'</td>
<td>cu (kPa)</td>
<td>Φ_u</td>
<td>$S_u$/$\sigma_v'$</td>
</tr>
<tr>
<td>Glacial Foundation</td>
<td>5</td>
<td>30</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Tailings</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Foundation Silt - Top Layer</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(At the d/s of 2 dam)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation Silt - Middle Layer</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(At the d/s of 2 dam)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Foundation Silt - Bottom Layer</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(At the d/s of 2 dam)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Fill</td>
<td>0</td>
<td>35</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(Foundation at the downstream</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>toe of 2 dam)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Landfill</td>
<td>0</td>
<td>28</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(Upstream toe of Western</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>embankment)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Waste Rock</td>
<td>0</td>
<td>36</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>(Foundation- Western</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Embankment)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Geosynthetic Liner</td>
<td>0</td>
<td>24</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Rockfill (2)</td>
<td>Shear Strength Function $\tau_u = fn(\sigma'_{n})$ - Lep's average capped at 45°</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>

Notes.
1. $S_u/\sigma'v_0$ refers to the SHANSEP, or shear strength as a function of overburden model.
2. The values for Normal stress and shear stress for Lep’s average were entered into SlopeW to define a shear stress function. SlopeW effectively plots these figures onto a chart and defines a mathematical function $\tau_u = fn(\sigma'_{n})$. This chart is illustrated in Figure 10.1.
### TABLE 10.3
MATERIAL PARAMETERS FOR EXISTING 2 DAM EMBANKMENT

<table>
<thead>
<tr>
<th>Material</th>
<th>Drained Strength Parameters</th>
<th>Undrained Strength Parameters (1)</th>
<th>Liquefied Strength Parameters</th>
<th>$\gamma_{\text{sat}}$ (kN/m$^3$)</th>
<th>$\gamma_{\text{unsat}}$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$c'$ (kPa) $\Phi'$</td>
<td>$c_u$ (kPa) $\Phi_u$</td>
<td>$\frac{S_u}{\sigma'_{\text{v}}}$</td>
<td>-</td>
<td>20</td>
</tr>
<tr>
<td>Clay/Waste rock</td>
<td>5</td>
<td>30</td>
<td>0</td>
<td>40</td>
<td>20</td>
</tr>
<tr>
<td>Waste rock</td>
<td>0</td>
<td>35</td>
<td>0</td>
<td>35</td>
<td>20</td>
</tr>
<tr>
<td>Glacial Fill</td>
<td>5</td>
<td>30</td>
<td>$\frac{S_u}{\sigma'_{\text{v}}} = 0.5$</td>
<td>-</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$S_{u(\text{min})} = 40$ kPa</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Compacted Tailings</td>
<td>0</td>
<td>30</td>
<td>0</td>
<td>30</td>
<td>18</td>
</tr>
</tbody>
</table>

Notes:
1. $S_u/\sigma'_{\text{vo}}$ refers to the SHANSEP, or shear strength as a function of overburden model

### TABLE 10.4
MATERIAL PARAMETERS FOR EXISTING 5 DAM EMBANKMENT

<table>
<thead>
<tr>
<th>Material</th>
<th>Drained Strength Parameters</th>
<th>Undrained Strength Parameters (1)</th>
<th>Liquefied Strength Parameters</th>
<th>$\gamma_{\text{sat}}$ (kN/m$^3$)</th>
<th>$\gamma_{\text{unsat}}$ (kN/m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Zone A - Glacial Till containing some fines</td>
<td>5</td>
<td>30</td>
<td>$\frac{S_u}{\sigma'_{\text{v}}} = 0.5$</td>
<td>-</td>
<td>20</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$S_{u(\text{min})} = 40$ kPa</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Zone B - Glacial Fill (clayey silt, sandy clays, and gravelly clays)</td>
<td>5</td>
<td>30</td>
<td>$\frac{S_u}{\sigma'_{\text{v}}} = 0.5$</td>
<td>-</td>
<td>18</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>$S_{u(\text{min})} = 40$ kPa</td>
<td>18</td>
<td></td>
</tr>
<tr>
<td>Zone C - Filter (sandy gravel)</td>
<td>0</td>
<td>34</td>
<td>0</td>
<td>34</td>
<td>18</td>
</tr>
<tr>
<td>Zone D - Glacial Fill with minimum fines</td>
<td>2</td>
<td>33</td>
<td>0</td>
<td>35</td>
<td>18</td>
</tr>
</tbody>
</table>

Notes:
1. $S_u/\sigma'_{\text{vo}}$ refers to the SHANSEP, or shear strength as a function of overburden model

#### 10.5.2 Glacial Foundation

Geotechnical investigation work carried out by GHD in 2001 [Ref 22] indicated that a layer of glacial till may exist beneath the 2 Dam embankment although the relevant borehole excluded the foundation upstream of the initial starter dam. This material was reported to be predominantly a Silty Clay. No shear strength parameter was reported for this material.
Glacial till material parameters are available for the material that is present at the Bobadil TSF. At that site GHD have conservative assigned drained friction angle of 35° and cohesion of 5 kPa and a total stress parameter of $s_u/\sigma'_v = 0.5$ [Ref 23]. These parameters have been adopted for the glacial till material encountered at 2/5 Dam.

10.5.3 Tailings

Particle size distribution tests on the tailings recovered from 5 Dam show that 100% of the particles are finer than 0.6 mm and that it contains a significant quantity of low plasticity fines. As such, undrained shear strength parameters have been derived for use in the stability analyses, as it is believed that any slope failure passing through tailings will mobilise its undrained shear strength.

The strength parameter, shear strength as a function of the effective overburden stress, together with the post-liquefaction strengths, have been adopted based on the CPT results as described in the ATCW Geotechnical Investigation Report [Ref 16].

10.5.4 Foundation Silt

Particle size distribution tests on foundation silts (downstream of 2 Dam) show that 100% of the particles are finer than 0.6 mm, similar to 5 Dam. As such, undrained shear strength parameters have been derived for use in the stability analyses.

The strength parameter, shear strength as a function of the effective overburden stress, together with the post-liquefaction strengths, have been adopted based on the CPT results as described in the ATCW Geotechnical Investigation Report [Ref 16].

10.5.5 Fill

Fill material was encountered to a depth of approximately 3 m at the downstream toe of the existing 2 Dam during the recent geotechnical investigation work [Ref 16]. The fill material was found to be mixture of sand, gravel and cobbles.

Significant differences between drained and undrained strength response are not expected for these coarse grained fill materials. Consequently, drained strength parameters have been used for this material in all stability analyses.

The drained friction angle of 35° has been conservatively adopted from numerous data sources such as [Ref. 24], which correlate test results for medium to dense, angular gravels and sands.

10.5.6 Waste Rock

Waste rock material used for the rehabilitation and capping of the existing 1 Dam was found to be a mixture of gravel, cobbles and boulder size material.

Significant differences between drained and undrained strength response are not expected for this coarse grained materials. Consequently, drained strength parameters have been used for this material in all stability analyses.

The drained friction angle of 36° has been adopted for waste rockfill based on past experience with the similar materials.
10.5.7 Landfill

Landfill material was encountered to a depth of approximately 4 m at the upstream toe area of the proposed Western embankment during the recent geotechnical investigation work [Ref 16]. The landfill material was found to be a mixture of waste rock, steel, plastics, rubber and wood.

The drained friction angle of 28° has been conservatively adopted from numerous data sources such as [Ref. 25], which suggest the drained friction angle of 35° for fresh Municipal Solid Waste (MSW) and drained friction angle of 28° for degraded MSW.

10.5.8 Geosynthetic Liner

The design and installation manual for bitumen geomembranes [Ref. 26] indicates that the angle of friction of the membrane will vary depending upon the contact material. For sand and gravel the friction angle varies from 30° to 32°. For this design a cushion geotextile will be placed between the liner and the fine rockfill material. We have therefore de-rated the friction angle to approximate that of the cushion geotextile. A conservative drained friction angle of 20° has been adopted for stability analysis.

10.5.9 Rock Fill

Due to the large maximum particle size, and anticipated essentially well graded nature of the rockfill materials, significant differences between drained and undrained strength response are not expected.

Rockfill in the proposed quarry areas is generally classified as felsic volcanic sandstone and considered to be of medium quality. With regard to the shear strength of the rockfill, Leps [Ref. 27] has shown that the shear strength as expressed by its friction angle varies noticeably as the function of the effective normal stress. Figure 10.1 shows the relationship derived for rockfill for both static and pseudo-static loading conditions, together with the Leps' lower bound, average and upper-bound functions. It should be noted that the results of a shear box test conducted by Monash University for the nearby Bobadil rockfill source [Ref. 28] indicate that shear strength function of the Bobadil rockfill material is slightly higher than the Leps' average function. Hence, the adopted shear strength function is considered to be appropriate.

10.5.10 Existing 2 Dam Embankment

No detailed as-constructed records are available for the existing 2 Dam. Based on the information presented by GHD [Ref. 28], silty sand residue (tailings) was used for the construction of the starter embankment in the early 1960’s, and that a mixture of gravelly and sandy clays were used to raise the dam in 1968. Glacial gravelly materials were subsequently placed on the downstream face. A combination of clay and waste rock was used for raising the embankment in 1991. Waste rock has subsequently been placed to form a downstream buttress and included widening the crest width to 10 m and flattening the downstream slope to 1 (vertical) to 5 (horizontal). The approximate embankment profile is presented as Section 2 on Figure 8.3.

The adopted material properties for the slope stability analysis shown in Table 10.2 to 10.4 are based on the properties adopted by GHD for the stability analysis of 2 Dam [Ref. 28].

10.5.11 Existing 5 Dam Embankment

No detailed as-constructed information is available for the 5 Dam, however design drawings from 1969 have been identified. Based on the information available, the 5 Dam embankment section
consists of a zoned earth embankment with a central clay core, downstream filter and glacial outer shell (along the eastern side) and outer rockfill shell along the northern side.

The adopted material properties for the slope stability analysis shown in Table 10.2 to 10.4 are based on the shear strength characteristics of similar glacial fill material used for the Bobadil embankment [Ref. 23].

10.6 Stability Analyses

10.6.1 Overview

The embankment stability analyses were conducted on the maximum height cross section for each of the three cross sections considered. The phreatic surface adopted for the analyses was conservatively derived from a steady state seepage analysis.

10.6.2 Western Embankment (Section 1)

A summary of the various load cases considered for the Western Embankment is presented in Table 10.5. The Figure numbers referred to in the table are SLOPE/W outputs showing the critical failure surfaces for each case. These are presented in Appendix B.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Load Case</th>
<th>Crest RL (m)</th>
<th>Failure Type</th>
<th>Factor of Safety</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1</td>
<td>End of Construction</td>
<td>170</td>
<td>Upstream</td>
<td>1.33</td>
<td>B1</td>
</tr>
<tr>
<td>Stage 1</td>
<td>End of Construction</td>
<td>170</td>
<td>Downstream</td>
<td>2.30</td>
<td>B2</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static-Long-term (Prior to Filling)</td>
<td>170</td>
<td>Upstream</td>
<td>1.62</td>
<td>B3</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static-Long-term</td>
<td>170</td>
<td>Downstream</td>
<td>2.42</td>
<td>B4</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Seismic OBE - (pseudo-static analysis)</td>
<td>170</td>
<td>Downstream</td>
<td>1.40</td>
<td>B5</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Seismic MDE - (USCE Method)</td>
<td>170</td>
<td>Downstream</td>
<td>0.99</td>
<td>B6</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static - (Post Liquefaction)</td>
<td>170</td>
<td>Downstream</td>
<td>1.61</td>
<td>B7</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Static</td>
<td>173</td>
<td>Downstream</td>
<td>2.11</td>
<td>B8</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Seismic OBE - (pseudo-static analysis)</td>
<td>173</td>
<td>Downstream</td>
<td>1.17</td>
<td>B9</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Seismic MDE - (USCE Method)</td>
<td>173</td>
<td>Downstream</td>
<td>0.84</td>
<td>B10</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Seismic MDE for sliding on liner - (USCE Method)</td>
<td>173</td>
<td>Downstream</td>
<td>1.43</td>
<td>B11</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Static - (Post Liquefaction)</td>
<td>173</td>
<td>Downstream</td>
<td>1.18</td>
<td>B12</td>
</tr>
</tbody>
</table>

The seismic MDE load case for both Stage 1 and 2 returned an unacceptable FoS indicating that deformation of the crest is likely to occur. The results of the stability analyses are discussed further in Section 10.6.5 and the deformation analyses are discussed in Section 10.8.

10.6.3 Northern Embankment (Section 2)

A summary of the various load cases considered for the Northern Embankment is presented in Table 10.6. The Figure numbers referred to in the table are SLOPE/W outputs showing the critical failure surfaces for each case. These are presented in Appendix C.
### TABLE 10.6
**SUMMARY OF STABILITY ANALYSES - NORTHERN EMBANKMENT (SECTION 2)**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Load Case</th>
<th>Crest RL (m)</th>
<th>Failure Type</th>
<th>Factor of Safety</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End of Construction</td>
<td>170</td>
<td>Upstream</td>
<td>1.30</td>
<td>C1</td>
</tr>
<tr>
<td>Stage 1</td>
<td>End of Construction</td>
<td>170</td>
<td>Downstream</td>
<td>1.31</td>
<td>C2</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static-Long-term</td>
<td>170</td>
<td>Downstream</td>
<td>1.69</td>
<td>C3</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static-Long-term - Sliding on Liner</td>
<td>170</td>
<td>Downstream</td>
<td>1.67</td>
<td>C4</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Seismic OBE - (pseudo-static analysis)</td>
<td>170</td>
<td>Downstream</td>
<td>1.09</td>
<td>C5</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Seismic MDE - (USCE Method)</td>
<td>170</td>
<td>Downstream</td>
<td>0.81</td>
<td>C6</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static - (Post Liquefaction)</td>
<td>170</td>
<td>Downstream</td>
<td>1.35</td>
<td>C7</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Static</td>
<td>173</td>
<td>Downstream</td>
<td>1.53</td>
<td>C8</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Seismic OBE - (pseudo-static analysis)</td>
<td>173</td>
<td>Downstream</td>
<td>1.07</td>
<td>C9</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Seismic MDE - (USCE Method)</td>
<td>173</td>
<td>Downstream</td>
<td>0.81</td>
<td>C10</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Static - (Post Liquefaction)</td>
<td>173</td>
<td>Downstream</td>
<td>1.27</td>
<td>C11</td>
</tr>
</tbody>
</table>

The seismic MDE load case for both Stage 1 and 2 returned an unacceptable FoS indicating that deformation of the crest is likely to occur. The results of the stability analyses are discussed further in Section 10.6.5 and the deformation analyses completed is discussed in Section 10.8.

### 10.6.4 Eastern Embankment (Section 3)

A summary of the various load cases considered for the Western Embankment is presented in Table 10.7. The Figure numbers referred to in the table are SLOPE/W outputs showing the critical failure surfaces for each case. These are presented in Appendix D.

### TABLE 10.7
**SUMMARY OF STABILITY ANALYSES - EASTERN EMBANKMENT (SECTION 3)**

<table>
<thead>
<tr>
<th>Stage</th>
<th>Load Case</th>
<th>Crest RL (m)</th>
<th>Failure Type</th>
<th>Factor of Safety</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>End of Construction</td>
<td>170</td>
<td>Upstream</td>
<td>0.93</td>
<td>D1</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static-Long-term</td>
<td>170</td>
<td>Upstream</td>
<td>1.73</td>
<td>D2</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static-Long-term</td>
<td>170</td>
<td>Downstream</td>
<td>1.61</td>
<td>D3</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Seismic OBE - (pseudo-static analysis)</td>
<td>170</td>
<td>Downstream</td>
<td>1.36</td>
<td>D4</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Seismic MDE - (USCE Method)</td>
<td>170</td>
<td>Downstream</td>
<td>0.96</td>
<td>D5</td>
</tr>
<tr>
<td>Stage 1</td>
<td>Static - (Post Liquefaction)</td>
<td>170</td>
<td>Downstream</td>
<td>1.54</td>
<td>D6</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Static</td>
<td>173</td>
<td>Downstream</td>
<td>1.61</td>
<td>D7</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Seismic OBE - (pseudo-static analysis)</td>
<td>173</td>
<td>Downstream</td>
<td>1.29</td>
<td>D8</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Seismic MDE - (USCE Method)</td>
<td>173</td>
<td>Downstream</td>
<td>0.87</td>
<td>D9</td>
</tr>
<tr>
<td>Stage 2</td>
<td>Static - (Post Liquefaction)</td>
<td>173</td>
<td>Downstream</td>
<td>1.40</td>
<td>D10</td>
</tr>
</tbody>
</table>

The analyses indicates that the Stage 1 end of construction case returned an unacceptable FoS, indicating that staged construction of this embankment will possibly be required. Staging of the Stage 1 construction is discussed in Section 10.7. The analyses also indicate the seismic MDE load case for both Stage 1 and 2 returned an unacceptable FoS indicating that deformation of the crest
is likely to occur. The results of the stability analyses are discussed further in Section 10.6.5 and the deformation analyses completed is discussed in Section 10.8.

10.6.5 Results

10.6.5.1 Static Analyses

The stability analyses results indicate that under static load conditions the Stage 1 and Stage 2 embankment heights have an acceptable FS except the eastern embankment at the end of construction. The FS less than 1.0 for the eastern embankment for the end of construction analysis indicates that construction of the embankment over the tailings to the full stage height in a single stage cannot be achievable. Hence, staged construction is required for this embankment. Detailed analyses completed for Staged construction are discussed in Section 10.7.

10.6.5.2 Pseudo Static Analyses

The peak strength, un-factored earthquake load OBE condition represents a pseudo-static assessment of the serviceability of the embankment. The analyses have yielded FS results in excess of 1.0 for the critical embankment failure cases, indicating that deformations would not be expected during the loading case.

The MDE USCE screening analyses have yielded a FS less than 1.0, indicating that there will be some cumulative deformation during shaking. In accordance with the design principles outlined in ANCOLD guideline [Ref. 8], it was concluded that deformation analyses are required for all the embankments. The estimation of deformation for all of the embankments is discussed in Section 10.8.

10.6.5.3 Post Liquefaction Analyses

The post liquefaction analysis was completed to provide an indication of the impact of residual shear strength tailings and foundation silts (post liquefied strength) on the overall global stability of the embankments. The post liquefaction analysis results of FS > 1 for all embankments indicate that the post liquefied state of the foundation tailings/silt possess sufficient strength to support the embankment.

10.7 Staged Construction

The stability analyses completed for construction of Stage 1 of the eastern embankment to the required height over the tailing yielded a FS less than 1.0. Hence, staged construction is required. Calculations have been completed to estimate the possible dissipation and consolidation time required between construction stages to provide an acceptable FS > 1.

The dissipation tests completed during CPTu investigation work indicate that the coefficient of consolidation \(C_t\) ranges between 500 m²/year to 10,000 m²/year [Ref 16]. Of the calculations completed, it is estimated that on average a minimum of 50 days (based on \(C_t\) of 500 m²/year) will be required between construction stages.
The proposed staged construction sequence of the eastern embankment over tailings is as follows:

1. Place initial layers of rockfill to a maximum height of 2 m in layers with a maximum layer thickness of 400 mm.
2. Allow the tailings to consolidate for 50 days from the start of the construction/placement at any point to allow construction induced excess pore pressure to dissipate.
3. Place a further 1.5 m of rockfill (also in maximum 400 mm thick layers) with an offset bench of 5 m from the upstream edge of the initial rockfill layers.
4. Allow the tailings to consolidate for a further 50 days.
5. Place a further 1.5 m of rockfill (in maximum 400 mm thick layers).

The 50 day period between layers may be reduced on the basis of field performance and the degree of construction induced excess pore pressure. It is recommended that vibrating wire piezometers are installed ahead of construction to monitor pore pressures and hence be able to identify when construction subsequent layers can commence.

The end of construction stability analyses results are summarised in Table 10.8 whilst the graphical outputs are presented in Appendix E. The stability analysis assumes that after the time lag of 50 days, the excess pore pressure from the previous layer have dissipated (to a residual of no more than 10% of the maximum applied load) and the tailings foundation is in a normally consolidated state.

**TABLE 10.8 END OF CONSTRUCTION STABILITY ANALYSES RESULTS**

<table>
<thead>
<tr>
<th>Structure</th>
<th>Description</th>
<th>Layer Thickness (m)</th>
<th>Factor of Safety (FoS)</th>
<th>Allowable FoS</th>
<th>Figure</th>
</tr>
</thead>
<tbody>
<tr>
<td>Eastern Embankment</td>
<td>Layer 1</td>
<td>2.0</td>
<td>1.63</td>
<td>1.3</td>
<td>E1</td>
</tr>
<tr>
<td></td>
<td>Layer 2</td>
<td>1.5</td>
<td>1.35</td>
<td>1.3</td>
<td>E2</td>
</tr>
<tr>
<td></td>
<td>Layer 3 (to RL 170 m)</td>
<td>1.5</td>
<td>1.33</td>
<td>1.3</td>
<td>E5</td>
</tr>
</tbody>
</table>

10.8 Deformation Analysis

10.8.1 Overview

As the pseudo-static analyses indicate that deformations in excess of those which are considered tolerable may occur, dynamic deformation analyses have been undertaken to assess the possible magnitude of the deformation which could be expected from the seismic event. It is considered that the deformation in the Stage 1 embankment will be smaller than the deformation that may occur in the Stage 2 embankment. Hence, deformation is estimated only for Stage 2 embankment.

10.8.2 Methodology

A simplified semi-empirical relationship for estimating permanent displacement due to earthquake induced deformation presented in [Ref. 29] was used to assess the deformation of the embankment. This method utilizes a nonlinear fully coupled stick-sliding block model to capture the dynamic performance of embankments.

10.8.3 Parameters

For the purpose of this study the following parameters (used in the estimation of the earthquake induced displacement) have been obtained from various resources.
• **Shear Wave Velocity (Vs)**

The CPT analysis results [Ref. 16] indicate that the shear wave velocity in the silt foundation varies from 50 m/s to 150 m/s. An average shear wave velocity of 75 m/s was adopted for the foundation and the embankment.

• **Spectral Acceleration (Sa)**

Spectral acceleration corresponding to estimated initial fundamental period was directly obtained from work completed by GHD in 2007 [Ref. 11].

• **Yield Acceleration**

Yield accelerations for all the embankments were estimated using SLOPE/W analysis using liquefied shear strength for tailings and silt foundation.

The estimated parameters for all the three embankments are summarised in Table 10.9.

<table>
<thead>
<tr>
<th>Parameters</th>
<th>Western Embankment</th>
<th>Northern Embankment</th>
<th>Eastern Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Yield (triggering) Acceleration (m/s²) (1)</td>
<td>0.02</td>
<td>0.050</td>
<td>0.026</td>
</tr>
<tr>
<td>Height of Sliding Mass (m)</td>
<td>26</td>
<td>38</td>
<td>27</td>
</tr>
<tr>
<td>Critical Spectral Acceleration (m/s²)</td>
<td>0.211</td>
<td>0.153</td>
<td>0.199</td>
</tr>
<tr>
<td>Average Shear Wave Velocity (m/s)</td>
<td>75</td>
<td>75</td>
<td>75</td>
</tr>
<tr>
<td>Initial Fundamental Period (s)</td>
<td>0.901</td>
<td>1.317</td>
<td>0.936</td>
</tr>
</tbody>
</table>

Note 1. The yield (triggering) acceleration is based on the post-peak strengths (i.e. liquefied strengths).

10.8.4 Results

For an earthquake of 6.5 magnitude the estimated deformations along the failure plane for each of the Stage 2 embankments are summarised in Table 10.10.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Western Embankment</th>
<th>Northern Embankment</th>
<th>Eastern Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 2</td>
<td>68</td>
<td>20</td>
<td>46</td>
</tr>
</tbody>
</table>
The predicted maximum deformation of 0.68 m on the western embankment does not exceed the total normal operating freeboard of 2.0 m. Hence, the magnitude of the estimated deformations would not be of sufficient magnitude to cause loss of storage under normal operating conditions.

10.9 Deformation for Closure

In accordance with the closure principles outlined in ANCOLD guidelines [Ref. 8], the Maximum Credible Earthquake (MCE) was used to assess post closure deformation. The MCE is defined as an earthquake geologically defined as the maximum for the site. Whilst a site specific seismic study has not been completed to identify the MCE, the GHD study for the Bobadil TSF [Ref. 11] does provide an estimate for the MCE (on the basis of the influence of local faults). Based on information contained within the GHD report a peak ground acceleration of 0.28 g has been adopted for the MCE.

The estimated deformations along the failure plane for closure with MCE using semi-empirical relationship presented in [Ref. 29] are summarised in Table 10.11.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Western Embankment</th>
<th>Northern Embankment</th>
<th>Eastern Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 2</td>
<td>108</td>
<td>34</td>
<td>80</td>
</tr>
</tbody>
</table>

It should be noted that for the above deformation analyses the value of yield acceleration was calculated using liquefied shear strength for tailings and foundation silt. Given the shape of the critical yield surfaces, it is expected that this maximum deformation will be in an essentially horizontal direction; vertical deformation is likely to be less than 50% of this value.

11 SEEPAGE ANALYSIS

11.1 Overview

The existing 2/5 Dam and 1 Dam are currently discharging impacted seepage water into the downstream environment of the dam, so engineering design for the new TSF is considering various control measures to minimise future impacts.

Recent geotechnical investigation of the ground conditions within and around the proposed dam indicates that the underlying bedrock has a relatively high permeability and the flow mechanism is fracture flow. Hence, it will not be possible to completely stop seepage but the engineering design will greatly reduce future seepage rates and the proposed depositional methodology (sub-aqueous) will greatly minimise additional contaminant loads from developing by preventing the oxidation of tailings and hence the formation of AMD.

11.2 Proposed Seepage Mitigation Measures

11.2.1 Western Embankment Area

This embankment will be founded on the old 1 Dam. Investigation within this area indicates that the foundation will consist of a mixture of historic mine waste overlying tailings. Bedrock in this
area was encountered at between 5.5 m and 16.5 m. The waste rock overlying the tailings has a relatively high permeability in comparison to the underlying tailings. The underlying bedrock was found to have a relatively high permeability of approximately $7.6 \times 10^{-6}$ m/s.

Engineering design to minimise future seepage impacts for this embankment consists of a geosynthetic lining system on the upstream face of the embankment, a geosynthetic lining system across the old 1 Dam surface and a grout curtain constructed across the northern, western and southern flanks of 1 Dam. The lining system will also be extended below the screening wall. The lining system will be anchored to the grout curtain to effectively seal the 1 Dam and hence minimise surface expression of seepage. A plan of the area is provided as Figure 11.1, whilst a typical Section is provided as Section 1 on Figure 8.3.

11.2.2 Northern Embankment

The existing 2 Dam embankment (proposed location of the northern wall) is founded on glacial and silt materials to depths of up to 17 m. The silty material below about 6 m has a relatively low permeability whilst the overlying fill and underlying bedrock has a relatively higher permeability.

For this embankment, engineering design to minimise future seepage impacts will include:

- Installation of a bentonite/cement cut-off to 6 m depth at the downstream toe of the existing embankment, founded within the low permeability silts;
- The construction of a geosynthetic lining system on the downstream face of the existing embankment which will be extended up the upstream face of the raised embankment.
- Where foundations are shallow rock, i.e. along the western abutment, a grout curtain will be installed to a depth of nominally 6 m.

Figure 11.1 shows the approximate extent of the cut-off and grout curtain, whilst a typical section of the proposed embankment cross section is presented as Section 2 on Figure 8.3.

11.2.3 Eastern Embankment

The eastern embankment will be constructed across the eastern side of 5 Dam, founding in the most part on the existing tailings but also on a natural rock foundation towards the southern end and will straddle the existing embankment at the northern end.

The tailings foundation consists of about 20 m of tailings overlying a thin veneer of glacial deposits and relatively shallow bedrock. Where the embankment is to be founded on a natural foundation this will consist of a thin veneer of glacial soils overlying bedrock. The permeability of the bedrock varies from $1 \times 10^{-6}$ m/s to $5 \times 10^{-7}$ m/s.

Where the embankment is founded on the tailings, the upstream face of the embankment will be lined with a geosynthetic membrane. The lining system will be installed through the rockfill (after the initial 1 m of rockfill has been placed and tied into the existing 5 Dam embankment clay core. This will minimise the potential for seepage expression at the current 5 Dam crest level.

The Stitt River is aligned parallel to, and within close proximity to the eastern wall of the 5 Dam. Deep seepage from the existing 5 Dam is potentially impacting the river. The increase in wall height (by approximately 8 m) will result in a higher driving head and hence deep seepage through the base of 5 Dam will increase with time. To reduce future impacts, a geosynthetic lining system will be placed across the surface of the 5 Dam and anchored into the volcanic rock that forms a natural barrier between the 2 and 5 Dams. A grout curtain will be installed within the volcanic
rock to further reduce seepage. In addition to these measures, the existing bypass drain that runs along the eastern downstream toe of the embankment will be regraded and directed to the SCP.

**Figures 8.1 and 8.2** present Stage 1 and 2 Layout Plans, whilst typical sections are presented on **Figure 8.4** and the grouting extents are shown on **Figure 11.1**.

### 11.2.4 Spillway Region

A grout curtain will be constructed through the intact rock between the western and northern embankments to a depth of approximately 10 m. This grout curtain will be tied into the grout curtain to be installed along the northern flank of the existing Dam and the grout curtain located along the downstream toe of the northern embankments western abutment. The grout curtain will provide an effective seal between the TSF and the diversion drain constructed to protect South Rosebery in the unlikely event of dam failure from the western embankment. The extent of the grout curtain is presented on **Figure 11.1** whilst typical liner details are presented on **Figure 8.4 and 8.5**.

### 11.2.5 Seepage Collection Drain

An additional measure to reduce future impacts downstream of the new TSF is the construction of a seepage collection drain around the eastern and northern portion of the site. This drain will intercept any near surface and surface flows from the eastern and northern embankments. The drain will be constructed to the north of the existing impacted area downstream of the 2 Dam.

The drain will be terminated at the SCP. A pump will be installed at the SCP to transfer any runoff and collected seepage to the TSF.

The alignment of the drain is shown on **Figure 8.1**.

### 11.3 Analyses

#### 11.3.1 Overview

Detailed steady state seepage analyses were carried out to estimate the seepage fluxes and evaluate the effectiveness of the proposed seepage control measures. The seepage assessment was conducted using the finite element numerical modelling software, SEEP/W utilising steady state condition (worst case scenario). The three embankment sections selected for the seepage analysis are shown as Sections 1, 2 and 3 on **Figures 8.3 and 8.4**.

The following limitations should be considered when interpreting the results of the seepage analyses:

- The seepage model is a simplified 2 dimensional representation of the hydrogeology at the site. In reality, the system is 3 dimensional and there will be significant spatial variations in the hydrogeological properties in the underlying foundation.
- The geological units and thickness vary across the site. Although site investigation has been carried out, there remains significant uncertainty in the geological profile and associated hydraulic properties across the site.

#### 11.3.2 Boundary Conditions

The boundary conditions applied to the models are summarised below:
A potential seepage face is applied to the seepage collection drains to simulate the recovery/flow of seepage.

A total head is applied at the location of the existing Stitt River to simulate 1 m depth of water in the river. Note that differing elevations are applied to the eastern and northern sections.

For the Stage 1 analysis a constant head boundary of RL 169 m was applied to the tailings surface to simulate the water cover surface above the tailings surface.

For the Stage 2 analysis a constant head boundary of RL 172 m was applied to the tailings surface to simulate the water cover surface above the tailings surface.

### 11.3.3 Material Parameters

All material permeability values included for the foundation in the seepage analyses have been determined either by laboratory tests or from in-situ testing completed during the geotechnical investigation. The material properties for embankment zones have been conservatively adopted based on our previous experience with the similar materials. A coefficient of permeability of \(1 \times 10^{-13}\) m/s has been adopted for the Geosynthetic liner based on the data supplied by manufacturer [Ref. 26].

The hydraulic properties adopted for the seepage modelling are summarised in Table 11.1 to Table 11.3.

#### TABLE 11.1
**HYDRAULIC PARAMETERS FOR WESTERN EMBANKMENT**

<table>
<thead>
<tr>
<th>Material</th>
<th>Horizontal Saturated Hydraulic Conductivity, (k_h) (m/s)</th>
<th>Hydraulic Conductivity Ratio (k_v/k_h)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation-Waste rock</td>
<td>(1 \times 10^{-6})</td>
<td>1</td>
<td>Adopted from Coffey [Ref. 17]</td>
</tr>
<tr>
<td>Tailings</td>
<td>(2 \times 10^{-7})</td>
<td>1</td>
<td>CPT testing and Laboratory testing.</td>
</tr>
<tr>
<td>Foundation Gravel</td>
<td>(1 \times 10^{-6})</td>
<td>1</td>
<td>Assumed</td>
</tr>
<tr>
<td>Foundation Weathered Rock</td>
<td>(8 \times 10^{-6})</td>
<td>1</td>
<td>Packer testing</td>
</tr>
<tr>
<td>Foundation Competent Rock</td>
<td>(1 \times 10^{-6})</td>
<td>1</td>
<td>Extrapolated from packer tests</td>
</tr>
<tr>
<td>Landfill</td>
<td>(1 \times 10^{-6})</td>
<td>1</td>
<td>Adopted from Ref. 25</td>
</tr>
<tr>
<td>Grout Curtain</td>
<td>(1 \times 10^{-8})</td>
<td>1</td>
<td>Target design value</td>
</tr>
<tr>
<td>Liner</td>
<td>(1 \times 10^{-13})</td>
<td>1</td>
<td>Manufacturer’s data sheet</td>
</tr>
<tr>
<td>Rockfill</td>
<td>(1 \times 10^{-5})</td>
<td>1</td>
<td>Assumed, typical</td>
</tr>
</tbody>
</table>

#### TABLE 11.2
**HYDRAULIC PARAMETERS FOR NORTHERN EMBANKMENT**

<table>
<thead>
<tr>
<th>Material</th>
<th>Horizontal Saturated Hydraulic Conductivity, (k_h) (m/s)</th>
<th>Hydraulic Conductivity Ratio (k_v/k_h)</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation-Fill</td>
<td>(1 \times 10^{-6})</td>
<td>1</td>
<td>Assumed</td>
</tr>
<tr>
<td>Tailings</td>
<td>(2 \times 10^{-7})</td>
<td>1</td>
<td>CPT testing and Laboratory testing.</td>
</tr>
<tr>
<td>Foundation Silt</td>
<td>(1.5 \times 10^{-8})</td>
<td>1</td>
<td>CPT testing and Laboratory testing.</td>
</tr>
</tbody>
</table>
### TABLE 11.3
HYDRAULIC PARAMETERS FOR EASTERN EMBANKMENT

<table>
<thead>
<tr>
<th>Material</th>
<th>Horizontal Saturated Hydraulic Conductivity, ( k_h ) (m/s)</th>
<th>Hydraulic Conductivity Ratio ( k_v/k_h )</th>
<th>Source</th>
</tr>
</thead>
<tbody>
<tr>
<td>Foundation Bedrock</td>
<td>5 X 10^{-6}</td>
<td>1</td>
<td>Packer testing</td>
</tr>
<tr>
<td>Cut-Off</td>
<td>1 X 10^{-8}</td>
<td>1</td>
<td>Assumed, typical</td>
</tr>
<tr>
<td>Waste Rock</td>
<td>1 X 10^{-6}</td>
<td>1</td>
<td>Assumed, typical</td>
</tr>
<tr>
<td>Glacial Fill</td>
<td>5 X 10^{-8}</td>
<td>1</td>
<td>Assumed, typical</td>
</tr>
<tr>
<td>Compacted Tailings</td>
<td>5 X 10^{-8}</td>
<td>1</td>
<td>Assumed, typical</td>
</tr>
<tr>
<td>Liner</td>
<td>1 X 10^{-13}</td>
<td>1</td>
<td>Manufacturer’s data sheet</td>
</tr>
<tr>
<td>Rockfill</td>
<td>1 X 10^{-5}</td>
<td>1</td>
<td>Assumed, typical</td>
</tr>
</tbody>
</table>

#### 11.3.4 Seepage Analysis Results

The steady state seepage analyses were conducted with the pond at its maximum level (worst case scenario) for each of the three embankment cross sections considered. In addition to the seepage analyses of the proposed embankment raises of Stage 1 and Stage 2, seepage analyses have also been carried out for the existing conditions and for the future raises without additional seepage control measures. The purpose of the additional seepage analyses is to evaluate the effectiveness of the proposed seepage control measures. The seepage results for the each of the embankments are discussed in the following sections.

#### 11.3.4.1 Western Embankment

The steady state seepage analyses results for the Western embankment is summarised in Table 11.4. The graphical output of the seepage analysis results are presented in Appendix F.
### TABLE 11.4
SEEPAGE FLUXES (m3/day/unit width) - WESTERN EMBANKMENT

<table>
<thead>
<tr>
<th>Receiving End</th>
<th>Western Embankment (1)</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stage 1 (Pond at RL 169 m)</td>
<td>Stage 2 (Pond at RL 172 m)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>With Control Measures</td>
<td>Without (2) Control Measures</td>
<td>With Control Measures</td>
<td>Without (2) Control Measures</td>
</tr>
<tr>
<td>Total</td>
<td>0.2</td>
<td>0.4</td>
<td>0.3</td>
<td>0.5</td>
<td></td>
</tr>
<tr>
<td>Seepage Collection Drain</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Stitt River, Groundwater and Environment</td>
<td>0.2</td>
<td>0.4</td>
<td>0.3</td>
<td>0.5</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**
1. Seepage flux is measured at the grout curtain located to the west of the screening wall.
2. No additional seepage control measures other than the liner on the upstream face.

The seepage estimates presented in Table 11.4 indicate that the proposed control measures for the western embankment reduce the seepage to the environment by approximately 50%.

The graphical outputs presented in Appendix F indicate that the proposed measures control the phreatic surface within the foundation and restrict the flow of seepage into the existing contaminated tailings zone.

11.3.4.2 North Embankment

The steady state seepage analyses results for the Northern embankment is summarised in Table 11.5. The graphical output of the seepage analysis results are presented in Appendix F.

### TABLE 11.5
SEEPAGE FLUXES (m3/day/unit width) - NORTHERN EMBANKMENT

<table>
<thead>
<tr>
<th>Receiving End</th>
<th>Northern Embankment</th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stage 1 (Pond at RL 169 m)</td>
<td>Stage 2 (Pond at RL 172 m)</td>
<td></td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>With Control Measures</td>
<td>Without (3) Control Measures</td>
<td>With Control Measures</td>
<td>Without (3) Control Measures</td>
</tr>
<tr>
<td>Total</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>0.3</td>
<td></td>
</tr>
<tr>
<td>Seepage Collection Drain</td>
<td>-</td>
<td>0.1</td>
<td>-</td>
<td>0.1</td>
<td>-</td>
</tr>
<tr>
<td>Stitt River, Groundwater and Environment</td>
<td>0.2</td>
<td>0.1</td>
<td>0.2</td>
<td>0.1</td>
<td>0.3</td>
</tr>
</tbody>
</table>

**Notes:**
1. No additional seepage control measures other than the liner on the upstream face of the future raises.
The seepage estimates presented in Table 11.5 indicate that the seepage control measures adopted for the proposed design minimise the current embankment seepage to the environment by 50%. No additional seepage to the environment from the future raises is expected.

11.3.4.3 Eastern Embankment

The steady state seepage analyses results for the Eastern embankment is summarised in Table 11.6. The graphical output of the seepage analysis results are presented in Appendix F.

**TABLE 11.6**

SEEPAGE FLUXES (m3/day/unit width) - EASTERN EMBANKMENT

<table>
<thead>
<tr>
<th>Receiving End</th>
<th>Eastern Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Existing Conditions</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>Total</td>
<td>0.5</td>
</tr>
<tr>
<td>Seepage Collection Drain</td>
<td></td>
</tr>
<tr>
<td>Stitt River, Groundwater and Environment</td>
<td>0.5</td>
</tr>
</tbody>
</table>

Notes:
1. No additional seepage control measures other than the liner on the upstream face of the future raises.
2. Seepage flux is measured on the basis of constructing a seepage collection drain at the downstream toe of the eastern flank of the 5 Dam and installation of a lining system across the existing tailings surface of 5 Dam along with a grout curtain located within the rock between 2 and 5 dams.

The seepage estimates presented in Table 11.6 indicate that the seepage control measures adopted for the proposed embankment raises minimise the current embankment seepage to the environment by 80%. No additional seepage to the environment from the future raises is expected.

12 SETTLEMENT OF TAILINGS

12.1 General

The Eastern embankment and Western embankment are proposed to be constructed over the existing tailings dams. The geotechnical investigation results indicate that that tailings in the existing dams are saturated and in a normally consolidated state. Hence, tailings will experience settlement under the embankment load. The amount of total settlement is critical for the design of effective geosynthetic lining system. For the purpose of the liner design, the primary and secondary consolidation settlements are calculated using well established theories.

12.2 Consolidation Settlement

The magnitude of primary consolidation settlement under embankment loading is calculated using one dimensional consolidation theory using laboratory determined primary compression index (Cc).
No specific consolidation test has been carried out for the tailings in the existing dams, however consolidation test completed for on current tailings samples supplied by MMG has been conservatively used to estimate $C_c$. The estimated $C_c$ for the fine fraction of tailings is 0.132.

For estimation of secondary consolidation settlement, the secondary compression ratio is conservatively assumed as 0.05 $C_c$ [Ref. 30].

The estimated total settlement for Stage 1 and Stage 2 construction of the Western and Eastern embankments are summarised in Table 12.1. It should be noted that the majority of the settlement will occur during construction of the dam.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Western Embankment</th>
<th>Eastern Embankment</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1</td>
<td>26</td>
<td>45</td>
</tr>
<tr>
<td>Stage 2</td>
<td>32</td>
<td>57</td>
</tr>
</tbody>
</table>

### 12.3 Post Seismic Settlement

Post seismic settlement due to liquefaction has been calculated based on critical state soil mechanics theory. If the material undergoes full liquefaction, the effective stress will move to the Critical State Line (CSL) due to pore pressure increase. On dissipation of excess pore pressures the maximum settlement that the material can experience would be associated with normal consolidation along a new compression line, assumed to be parallel but offset from the initial normal consolidation line.

No triaxial testing has been carried out for 1 Dam or 5 Dam tailings. For the purpose of this assessment, a CSL, established with similar tailings (Bobadil TSF tailings) has been used. Based on an initial void ratio of 1.3 (for both 1 Dam and 5 Dam tailings) the estimated settlement for the Stage 2 embankment load is 0.4 m for the Western Embankment and 0.6 m for the Northern Embankment.

### 12.4 Likely Post Closure Settlement

The total possible post closure settlement is due to vertical deformation during cyclic loading during an MCE event, and post seismic settlement due to liquefaction. Based on the assessments discussed in Section 10.9 and Section 12.3 the expected maximum vertical deformation for this dam is around 1.0 m. Hence, the minimum freeboard of 2 m to the embankment crest and 1 m to the spillway invert should be considered for the detailed closure design.

### 12.5 Elongation of Liner

The estimated maximum elongation of the geosynthetic liner based on consolidation, seismic and post seismic settlements is around 10%.

The bituminous geomembrane proposed for this design has excellent stress strain behaviour characteristics and elongation at failure is reported as 60-70%. Hence, it is considered that the proposed bituminous geomembrane is suitable for this design.
13 TAILINGS MANAGEMENT

13.1 Delivery and Distribution

Tailings management for the 2/5 Dam TSF will be relatively straight-forward. Tailings will be discharged via single spigot offtakes from a nominal 200 mm HDPE perimeter main that will be located on each of the embankments and a portion of the southern flank of the facility. The offtakes will be of the same diameter as the ring main and will be extended nominally 200 m onto the dam and will be supported by floats.

The outlet of these floating pipelines will be secured in place by a buoy arrangement within the dam. The buoys will be set-out so that relatively even distribution of tailings within the dam can occur. The active discharge pipe outlet will be moved on a regular basis using a boat so that the tailings do not build-up and become exposed above the water surface. A detailed operational plan will developed prior to commissioning the dam.

Figure 13.1 shows the general layout of the perimeter ring main and the floating discharge pipelines.

13.2 Pipeline Flushing

A dedicated pipeline flushing system will be designed to allow flushing of the perimeter main and discharge pipes. A valving arrangement will be constructed between the north and west embankments to allow flushing of the pipelines located on the crest of the walls in between depositional cycles.

A flushing pump will be fixed to a cradle located within an inverted culvert positioned adjacent to the decant structure. The pump will be able to be raised and lowered for maintenance purposes and in response to the increase with time of the tailings elevation.

The location of the pumping system is shown on Figure 8.7 and typical sections and details are presented on Figure 8.9 and 8.10.

14 WATER BALANCE

14.1 Methodology

The water balance model has been developed in two stages (TSF and Seepage Collection Pond) to address the following are the key objectives:

(i) Determine the probable likelihood the TSF spillway will be engaged;

(ii) Assess the water level fluctuation and hence the statistical range of expected water levels in the TSF due to the seasonal impacts of the Rosebery regional climate;

(iii) Determine the probable performance of the TSF decant system; and

(iv) Determine the probable performance of the seepage Collection Pond.

The model was run on a daily mass balance approach over the life of mine, using actual and synthetically derived daily climatic records. Inputs into the TSF system include rainfall, tailings bleed and catchment runoff, with losses being seepage and evaporation and outputs being decant...
return. Inputs to the Seepage Collection Pond (SCP) include rainfall, seepage and catchment runoff, with losses being evaporation and seepage return to the TSF.

A flow diagram of the model is presented as Figure 14.1 whilst further discussion on the inputs and outputs is presented in the following sections.

Based on an annual production rate of 800,000 tonne of tailings and a total tonnage of 5 Mt, it is estimated that deposition will occur over approximately 6.25 years.

14.2 Modelling Protocols

14.2.1 General

As mentioned above, the model is divided into two components, being the TSF and SCP. The inputs, outputs and losses are described below:

(i) Inflows to the system, consisting of:
   - Rainfall - direct into pond and runoff from the southern catchment area (the area between the clean water diversion drain and the TSF) and eastern catchment area (the area between the TSF embankment and the seepage collection drain);
   - Pump back from the seepage pond downstream of the northern embankment; and
   - Tailings water within the slurry.

(ii) Losses from the system, consisting of:
   - Seepage losses from the TSF pond;
   - Infiltration from the southern and eastern catchments;
   - Seepage from the TSF;
   - Retained interstitial water within the settled tailings; and
   - Evaporation.

(iii) Outflows, consisting of:
   - Decant return to the ETP.

The model sums the inflows and outflows from the TSF system to identify the available water for discharge to the ETP. The model assesses the change in pond volume, and hence pond height prior to any discharge of water from the system.

The SCP sums the inflows from seepage and rainfall run-off and assesses water availability for return to the TSF.

14.2.2 Climate Data

The basis for the climate dataset used in the water balance model is presented in Section 5.8.

As mentioned previously, the water balance has been run for three climatic cases over the LOM. The data derived from the rainfall database and used in the model are:

In order to cover the range of possible rainfall scenarios during normally expected climatic conditions over the life of mine, three sets of data representing the medium, maximum and minimum for 6.25 year periods have been used. These data periods are presented in Table 14.1.
### TABLE 14.1
CLIMATIC PERIODS

<table>
<thead>
<tr>
<th>Description</th>
<th>Period</th>
<th>Maximum Annual Rainfall for Period (mm)</th>
<th>Total Rainfall for Period (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Maximum 6.25 year rainfall</td>
<td>1968-1974</td>
<td>2,941 (1968)</td>
<td>13,934</td>
</tr>
<tr>
<td>Average 6.25 year rainfall</td>
<td>1918-1924</td>
<td>2,501 (1923)</td>
<td>12,163</td>
</tr>
<tr>
<td>Minimum 6.25 year rainfall</td>
<td>1933-1939</td>
<td>2,157 (1939)</td>
<td>10,734</td>
</tr>
<tr>
<td>Maximum rainfall year</td>
<td>1968</td>
<td>2,941</td>
<td></td>
</tr>
<tr>
<td>Average annual rainfall</td>
<td>N/A</td>
<td>1,985</td>
<td></td>
</tr>
<tr>
<td>Minimum rainfall year</td>
<td>1950</td>
<td>1,330</td>
<td></td>
</tr>
</tbody>
</table>

The water balance uses the average daily evaporation data presented in Section 5.8.

14.2.3 Rainfall Runoff

Rainfall has been modelled for the following model components:

- Direct rainfall onto the TSF Pond;
- Direct rainfall onto the SCP;
- Rainfall onto the southern catchment area between the TSF and the clean water diversion drain; and
- Rainfall onto the eastern catchment area between the TSF embankment and the seepage collection drain.

Rainfall is unfactored on the TSF and SCP surfaces but is routed through the Australian Water Balance (AWA) model for runoff from the surrounding catchments.

14.2.4 Evaporation

Evaporation is applied to the TSF and SCP. A pan factor of 1.0 has been applied to evaporation.

14.2.5 Bleed Water

Bleed water is the difference between the water in the tailings slurry as it leaves the tailings tank, and the retained water within the tailings at the initial settled density. Bleed water is considered to arrive instantaneously at the surface of the tailings, where it becomes part of the pond water. Bleed is a function of the tailings production rate, the tailings discharge solids content, and the tailings initial settled density.

The estimate of bleed water reporting to the pond is presented in Table 14.2.
TABLE 14.2
BLEED WATER ESTIMATE
(BASED ON A PRODUCTION RATE OF 800,000 DRY TONNES PER ANNUM)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solids discharge rate</td>
<td>2,191 t/day</td>
</tr>
<tr>
<td>Discharge solids concentration</td>
<td>30% Cw</td>
</tr>
<tr>
<td>Tailings water</td>
<td>5,114 m³/day</td>
</tr>
<tr>
<td>Initial settled density</td>
<td>0.5 t/m³</td>
</tr>
<tr>
<td>Water retained within tailings</td>
<td>1,843 m³/day</td>
</tr>
<tr>
<td>Bleed Water reporting to the pond</td>
<td>3,271 m³/day</td>
</tr>
</tbody>
</table>

14.2.6 Seepage and Impacted Runoff Return

Rogue seepage from the Northern and Eastern embankments and runoff from the critical duration 1:10 year ARI rainfall event is collected downstream of the TSF in the SCP. This water is then pumped back into the TSF pond. Historical water transfer from the existing seepage collection pond to the ETP indicates that the pump capacity is approximately 35 L/s. This pump capacity has been adopted in the model. A comparison of different pump capacities is shown below in Section 14.3.

The lining system for the proposed TSF will minimise seepage to the extent that only rogue seepage will need to be captured in the seepage collection drains. This seepage has been calculated on the basis of a liner coefficient of permeability of 1 x 10^-9 m/s, which results in a seepage value of approximately 600 m³/day.

The catchment area between the embankment and Seepage Collection Drain is approximately 14.5 ha and results in an average runoff value of approximately 506 m³/day.

The SCP has been designed to store a 1:100 year storm event of 24 hours duration. Based on the Intensity Frequency Duration (IFD) curves (Figure 5.3), a maximum catchment yield of 85%, a storage volume of 15,000 m³ has been adopted for the pond. This value generated from the IFD curves is higher than the maximum daily runoff based on the 100 year rainfall record, and thus represents a more conservative case.

14.2.7 Decant Return to the ETP

Routing of decant water from the surface of the TSF was based on developing relationships between water height and flow, based on a decant width of 0.9 m. Water return to the ETP will be via a gravity decant system. Based on the height differential between the decant invert and the point at which the decant pipe enters the ETP, decant flow has been limited to 60 L/s. It should be noted that this is less than the current tailings stream into the ETP so there should not be any major impact on the current ETP protocols.

14.2.8 Seepage Loss from the TSF

Seepage loss through the permeable floor of the TSF has been estimated on the basis that the permeability of the tailings is lower than the underlying fractured rock and hence limits seepage losses. Even with the installation of grout curtains and cut-off walls, deep seepage will still occur. This seepage is lost to the system and mixes with the base flow of groundwater from Mount Read and surrounds.
Seepage losses have been calculated on the basis of the tailings coefficient of permeability of $1 \times 10^{-7}$ m/s (derived from geotechnical investigative work on 5 Dam), a hydraulic gradient of 1 and the surface area of the TSF pond (which varies with time).

A sensitivity analyses has also been carried out to consider the effects on pond volume of a lower tailings coefficient of permeability. This is further discussed in Section 14.3.

### 14.3 Results

The key outputs from the water balance model are:

- an estimate of the daily discharge of decant water from the TSF to the ETP;
- pond water levels; and
- pond water levels within the SCP.

The pond water levels was used as a semi qualitative method of identifying the required freeboard for the facility, whilst the water storage within the SCP was used to assess whether there would any release to the downstream environment of potentially impacted water.

#### 14.3.1 Decant Water Discharge

A summary of the average daily discharge of decant water is presented in Table 14.3.

<table>
<thead>
<tr>
<th>Case</th>
<th>Average Decant Discharge (m³/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum</td>
<td>2,376</td>
</tr>
<tr>
<td>Average</td>
<td>2,785</td>
</tr>
<tr>
<td>Maximum</td>
<td>3,350</td>
</tr>
</tbody>
</table>

Daily discharge of decant water for each of the three climate cases is presented as Figures 14.2 to 14.4. The three figures present the daily discharge (limited by pipe flow) and the average daily discharge per year for each of the climate cases respectively.

As shown in Figures 14.2 to 14.4, decant return to the ETP will occur most of the year. Of note is that there is little or no decant return during the initial three months and then again for a period around year 3 for each of the three climate cases. This is on account of the borrow areas developed within the dam footprint filling prior to the average tailings elevation increasing further with time.

#### 14.3.2 Tailings Coefficient of Permeability Sensitivity Analysis

Decant discharge was also analysed under a scenario in which seepage losses from the TSF is halved and therefore the pond retains more water. Figure 14.5 presents this scenario for the maximum climate case. As shown on this Figure, the decant operates at maximum capacity for approximately 70% of the life of mine. In the corresponding climate case where seepage loss is not reduced (Figure 14.4), the decant flow operates at maximum capacity for approximately 44% of the time.

#### 14.3.3 Tailings Level and Pond Water Level

The water balance model tracks the increase in tailings elevation on a daily basis as well as the water level within the dam. The decant inlet is set at 2 m above tailings elevation to mimic the
adopted minimum water pond and represents the addition of stop boards to the decant. In reality, stop boards will be added in increments as the tailings level rises, but for the purpose of this water balance it is assumed that the increase in the decant inlet mimics the rise in the tailings beach.

The model indicates that the pond water levels are very similar for all 3 climate cases, mainly on account of decant water return. On this basis, only the average climate case has been presented in Figure 14.6.

14.3.4 Seepage Collection Pond Water Volume and Level

The water balance model tracks the SCP volume on a daily basis for the three climate cases considered and a seepage return pump rate of 35 L/s.

The model indicates that the volume of the SCP does not exceed the 15,000 m³ capacity for any of the three climate cases as shown in Figure 14.7. Figure 14.8 presents the pond depth with time. Figure 14.8 indicates that the pond depth does not exceed 2 m, which is the depth below the invert of the spillway. This Figure also depicts that the pond is never empty which is a function of the dead space within the base of the pond.

14.3.5 Seepage Collection Pond - Water Return Rate

Daily return water to the decant pond for each of the 3 climate cases has been modelled through the water balance. The average daily seepage return water is presented below in Table 14.4.

<table>
<thead>
<tr>
<th>Case</th>
<th>Daily Return Water (m³/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Minimum Climate Case</td>
<td>1,055</td>
</tr>
<tr>
<td>Average Climate Case</td>
<td>1,129</td>
</tr>
<tr>
<td>Maximum Climate Case</td>
<td>1,256</td>
</tr>
</tbody>
</table>

14.3.6 Seepage Return Pump Sensitivity Analysis

The results presented in Sections 14.3.4 and 14.3.5 are based upon a seepage pump capacity of 35 L/s. A sensitivity analyses has been carried out to assess the influence that a reduced seepage pump rate has on spill risk from the SCP for the design volume of 15,000 m³. To analyse the probability of spill risk, the entire 100 year rainfall data was utilised. Table 14.5 presents the modelled pump capacities with average pump back rate and daily overflow probability. Pump capacities above 25 L/s present a very low probability of overflow. As pump capacity approaches 10 L/s, the daily probability of overflow increases to 99%.
TABLE 13.5
SEEPAGE RETURN PUMP SENSITIVITY

<table>
<thead>
<tr>
<th>Pump Capacity (L/s)</th>
<th>Daily Overflow Probability</th>
</tr>
</thead>
<tbody>
<tr>
<td>35</td>
<td>0.00%</td>
</tr>
<tr>
<td>30</td>
<td>0.00%</td>
</tr>
<tr>
<td>25</td>
<td>0.03%</td>
</tr>
<tr>
<td>20</td>
<td>0.19%</td>
</tr>
<tr>
<td>15</td>
<td>1.60%</td>
</tr>
<tr>
<td>12</td>
<td>56.50%</td>
</tr>
<tr>
<td>10</td>
<td>99.00%</td>
</tr>
</tbody>
</table>

15  WATER MANAGEMENT

15.1 Decant Operation

As discussed in Section 8.3, the topography of the 2/5 Dam TSF, coupled with the tailings deposition methodology, allows decant water management to occur from an inclined decant constructed on the rock cut face adjacent to the spillway inlet as shown on Figure 8.7. Figure 8.9 shows typical sections for the decant.

The decant system will consist of gravity discharge from the TSF impoundment to the ETP. The decant will consist of inverted box culvert sections, nominally 900 mm in width, with a 500 mm diameter HDPE PN10 pipe constructed within a rock cutting through the northern abutment of the Western Embankment. The pipe trench will be backfilled with a cement/bentonite mixture between the decants and the Murchison Highway. The pipeline will pass under the Murchison Highway in a culvert and then be installed on the ground surface to the ETP.

Water discharged into the pipeline will be discharged via gravity to the ETP. The decant inlet (top of the inverted culvert sections) will progressively be raised by placing stop boards against the legs of the culverts. The stop boards will be sealed to the culvert legs to reduce the risk of tailings solids carry over into the decant pipe. The inlet of the inclined decant will be progressively raised as the tailings level within the facility rises. The concept of sub-aqueous deposition is centred around maintaining a 2 m water cover above the tailings level.

It will be an important operational aspect to manage the tailings deposition in a manner which maintains an even distribution of tailings and not allow a build-up of tailings in the area of the decants.

15.2 Freeboard

15.2.1 General

As discussed in Section 5.5 the TSF must have sufficient capacity above the normal operating pond level to store the following rainfall events without spilling:

- Extreme storm storage allowance of a 1 in 100, 72 hr rainfall event; and
- Contingency allowance, comprising wave run-up of 1 in 10 AEP average wind and an additional freeboard of 0.5 m.
15.2.2 Extreme Storm Storage

The flood from the southern diversion drain catchment (located south of the diversion drain) has been excluded from the extreme storage calculation, as it will be captured and diverted around the TSF. The design catchment area (including tailings surface) for the extreme storage is 58.4 hectares. The rainfall depth for the 1 in 100 AEP, 72-hour rainfall event is 215 mm. This represents a volume of water required to be stored of approximately 125,800 m$^3$ and the equivalent depth on the TSF is 0.28 m.

15.2.3 Contingency Allowance - Wave Run-up Estimates

15.2.3.1 General

The wave run-up calculations are based on the method presented in [Ref. 31]. The calculations have been based on the following assumptions:

- Water levels within the dam are at the maximum operational pond level at the end of each stage, i.e. 2 m water depth; and
- The Embankment will have a “smooth” upstream face consisting of a geosynthetic liner (i.e. wave run-up will be more pronounced than for a rockfill-faced embankment)

15.2.3.2 Predominant Wind

Wind data for the period August 2011 to January 2015 was obtained from the Rosebery Mine Weather Station. The data was analysed in order to determine the appropriate wind speed and predominant wind direction for use in the wave run-up calculations.

Due to the orientation of the TSF, it was identified that the predominant wind direction is from a North-Westerly direction are likely to result in the largest waves being generated (i.e. perpendicular to the Eastern Embankment and at the opposite end of the storage from the spillway).

The maximum wind speeds are effectively instantaneous data, representing wind gusts of short duration. It is considered that short duration wind gusts will not generate any significant waves. Hence, it is deemed more appropriate to utilise the results of the “average” wind speeds (potentially sustained over longer durations enabling waves to generate over the fetch) for the purpose of this design.

The 1 in 10 AEP wind speed was obtained by interrogating the available site specific data using the Log-Normal probability distribution. A plot of the ARI versus average wind speed is presented in Figure 15.1. The plot indicates that the 1 in 10 AEP average wind is 15 km/h.

15.2.3.3 Results

The TSF fetch distance for winds from a North-Westerly direction is calculated to be 620 m. For the design wind of 15 km/h, the maximum wave run-up is 0.35 m against the eastern embankment. It is unlikely that wave run-up of this magnitude will occur adjacent to the spillway due to the predominant wind direction. We have therefore assumed that any wave runup that may occur will be contained within the additional freeboard allowance for uncertainty in adopted values.
15.2.4 Summary

The freeboard and storm water containment storage requirements for the TSF below the spillway invert level is summarised in Table 15.1.

<table>
<thead>
<tr>
<th>Design Criteria</th>
<th>Required Freeboard above Maximum Operating Level (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Emergency Storm Storage</td>
<td>0.28</td>
</tr>
<tr>
<td>Contingency Allowance</td>
<td>0.50</td>
</tr>
<tr>
<td>Total</td>
<td>0.78</td>
</tr>
</tbody>
</table>

The storage-rating curves for the water storage above the tailings surface with the tailings at full supply levels are presented as Figure 8.12 which includes Stage 1 and Stage 2 development. The water balance indicates that on average, a 2 m depth of water will be present above the tailings surface. The maximum operating elevation for the two stages of dam development are presented in Table 15.2.

<table>
<thead>
<tr>
<th>Stage</th>
<th>Maximum Tailings Elevation (RL)</th>
<th>Maximum Operating Pond Level (RL)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1</td>
<td>166.22</td>
<td>168.22</td>
</tr>
<tr>
<td>Stage 2</td>
<td>169.22</td>
<td>171.22</td>
</tr>
</tbody>
</table>

15.3 Emergency Spillway

15.3.1 Overview

Spillway design has conducted using the numerical reservoir routing approach. The critical rainfall duration is the storm event which produces the peak spillway discharge, which is a function of the flood attenuation provided by the pond geometry.

15.3.2 Design Criteria

The design criteria for the emergency spillway is outlined in Section 5.5.

It is considered that the Spillway design criteria and design data are very similar for Stage 1 and Stage 2 designs. Hence, for the purpose of this study spillway design for both stages are considered to be same.

The design rainfall used for the spillway is described in Section 5.9.
15.3.3  Derivation of Design Flood

The inputs in the derivation of the design flood for a given duration are the design rainfall event and the design inflow hydrograph. The design inflow hydrograph for a particular duration storm is the summation of individual components hydrographs from the pond and the natural surface catchment.

The total catchment area of the TSF is approximately 45 ha and the contributing catchment area of the hillside to the south of the facility is approximately 56 ha. As the catchment is ungauged, the flood hydrograph has been modelled using the synthetic unit hydrograph technique. A synthetic unit hydrograph was constructed using the key parameters of the catchment (time of concentration, area, average slope), in accordance with the method described by Cordery and Webb [Ref. 32], and in Australian Rainfall and Runoff (ARR) [Ref. 15].

15.3.4  Spillway Flood Routing

Once the design flood for particular storm duration had been derived, spillway sizing was undertaken by routing the flood through the storage. This was accomplished using the storage indication method, a direct numerical procedure which is described in ARR [Ref. 15].

A diversion drain will be constructed to the south of the TSF. For the purpose of the design it has been assumed that the drain is not operational and hence the drains catchment area has been included in the calculations. It has conservatively been assumed that there is no attenuation of rainfall on the TSF as the deposition methodology is sub-aqueous and that prior to the PMP, the supernatant pool is at the spillway invert level, that being at RL 169 m for Stage 1 and RL 172 m for Stage 2. For both the raises the spillway level is at 1.0 m below crest height. The input parameters for flood routing through the TSF are summarised in Table 15.3. This is considered a conservative design case, given that the normal operating level for the facility will be approximately 1.2 m lower that assumed in the analyses.

Therefore, the flood wave passing through storage is both delayed and attenuated as it enters and spreads over the pool surface. The surcharge storage is gradually released over the spillway. The outflow depends on the spillway configuration, as well as on the surcharge storage characteristics. In order to perform satisfactorily, the spillway configuration must be able to pass the critical duration design AEP flood without overtopping of the embankment crest.

**TABLE 15.3**  
SPILLWAY FLOOD ROUTING INPUT PARAMETERS

<table>
<thead>
<tr>
<th>Case</th>
<th>Parameter</th>
<th>Stage 1</th>
<th>Stage 2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment Crest RL</td>
<td>(m, AHD)</td>
<td>170.0</td>
<td>173.0</td>
</tr>
<tr>
<td>Spillway Invert RL</td>
<td></td>
<td>169.0</td>
<td>172.0</td>
</tr>
<tr>
<td>Storage RL at commencement of Flood</td>
<td></td>
<td>169.0</td>
<td>172.0</td>
</tr>
<tr>
<td>Catchment</td>
<td>(ha)</td>
<td>45.0 TSF</td>
<td>45.0 TSF</td>
</tr>
<tr>
<td>Available Surcharge Storage</td>
<td>(ML)</td>
<td>439</td>
<td>439</td>
</tr>
</tbody>
</table>

Floods from storms of increasing duration were progressively routed through the storage, until a peak outflow was obtained. The spillway configuration (i.e. width) was considered satisfactory only if the spillway capacity was greater than the critical peak outflow. The results of the final (successful) routing trial for a 20 m wide spillway are summarised in Table 15.4. This summary
shows that the critical storm event is of 3 hour duration event. During this storm event the spillway, which is 1.0 m deep, will flow at 0.77 m depth. However, the analyses have been based on the pond level being at the invert of the spillway at the commencement of the rainfall event. ANCOLD guidelines [Ref. 8] suggest that the PMP rainfall should be applied to the normal operating pond elevation and hence the result presented in Table 15.4 is conservative. A plot of the inflow-outflow hydrograph for the critical 3 hr storm duration is presented in Figure 15.2.

### Table 15.4
**Spillway Flood Routing Results**

<table>
<thead>
<tr>
<th>Peak Outflow</th>
<th>Discharge (m³/s)</th>
<th>Flow Height (m)</th>
<th>Remaining Freeboard (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Spillway Capacity</td>
<td>35.5</td>
<td>1.00</td>
<td>0</td>
</tr>
<tr>
<td>1 hr storm</td>
<td>11.6</td>
<td>0.48</td>
<td>0.52</td>
</tr>
<tr>
<td>2 hr storm</td>
<td>19.6</td>
<td>0.69</td>
<td>0.31</td>
</tr>
<tr>
<td>3 hr storm</td>
<td>23.4 (Critical)</td>
<td>0.77</td>
<td>0.23</td>
</tr>
<tr>
<td>4 hr storm</td>
<td>22.3</td>
<td>0.75</td>
<td>0.25</td>
</tr>
</tbody>
</table>

15.3.5 Spillway Design

The adopted spillway configuration for both the Stage 1 (RL 170 m) and Stage 2 (RL 173 m) crest elevations will have the same base width of 20 m, and will be constructed as part of each stage of embankment construction. The spillway crest will be constructed at 1.0 m below the design crest height, being RL 169.0 m and RL 172.0 m, for the Stage 1 and Stage 2 works respectively.

Based on the flood routing results, the spillway will flow at approximately 0.77 m deep at the peak discharge for the critical PMP flood, giving 0.23 m remaining freeboard. As previously noted, the analysis has assumed that the supernatant pool is at the invert level of the spillway at the commencement of the storm. It is considered extremely unlikely that a PMP rainfall event will coincide with the supernatant pool operating at maximum level, therefore the critical flow depth is unlikely to be attained and hence the spillway design has a degree of inbuilt conservatism.

The Stage 1 spillway will have an invert of RL 169.0 m and will be cut through rock between the western and northern embankments. The rock either side of the spillway will be excavated to RL 173 m, equivalent to the Stage 2 crest elevation. The spillway will be raised during Stage 2 by construction of a 3 m high reinforced concrete weir within the Stage 1 spillway channel.

It is assumed that the base of the spillway will be founded in rock; as suggested by a borehole drilled in this area. Where this is not the case, near the outlet for example, the invert of the spillway will be lined with suitably sized erosion protection rock.

15.4 Diversion Drains

15.4.1 Clean Water Diversion

During the operational phase of the facility, stormwater from the southern hill (Mt Read) will be diverted to minimise the volume of surface run-off into the TSF from the surrounding, undisturbed ground. To facilitate this, a diversion drain will be constructed along the southern side of the facility. The drain will follow the natural topography at an average grade of 1:100 (V:H).
Figure 8.11 shows the proposed layout of the drain around the facility. The drain will discharge clean run off water towards the east, discharging into the Stitt River near the Henty track. Erosion protection will be provided where the drain discharges to the Stitt River.

Peak flows have been calculated using a geographic specific method for the West Coast of Tasmania, developed by the Tasmania Hydro-Electric Commission and outlined in Australian Rainfall and Runoff [Ref. 15].

The drain has been designed to discharge a 1:10 year AEP critical duration storm event, which is considered appropriate considering the design life of the facility is approximately 6 years. The design cross section has been based on the assumption that the bed material for the drain is rock and that the peak flow velocity should not exceed 1.2 m/s to minimise the potential for scour of the drain bed. The drain has also been designed so that the 1:10 year AEP storm event can be safely discharged.

The diversion drain will have a trapezoidal cross sectional area, with a base width of 2 m and 1:0.6 (V:H) side slopes on the upslope side and 1:1 (V:H) on the downslope side. The drain will be constructed on the uphill side of an access track, with a small (nominally 0.5 m high) safety bund on the downstream side of the drain.

The impact of failure of the drain (via an overtopping or breach event during a rainfall event exceeding 1:10 AEP) would some impact on the TSF. In the event of the drain failing the catchment area upstream of the point of failure would be directed into the TSF. This is unlikely to have major impact as the TSF has sufficient capacity to attenuate the additional water and then discharge it over a period of time through the gravity decant.

Upon closure of the facility, transverse drains may be excavated across the perimeter access track so that runoff is diverted towards the TSF. The need for this at closure will be the subject of future detailed closure design.

15.4.2 Seepage Collection Drain

A seepage collection drain will constructed downstream of the eastern and northern embankments. This drain has been design to enable discharge of a 1:10 AEP critical duration storm event. The reason for this is that the runoff from the design storm will dwarf seepage emanating from the facility. The design cross section is similar to the clean water diversion. The drain cross section consists of a 1 m wide base, 1:1 (V:H) side slopes and a nominal minimum depth of 0.5 m. An access track will be formed on the downstream side of the drain. The alignment of drain is shown on Figure 8.1, whilst typical sections are presented on Figures 8.3 and 8.4.

16 SURVEILLANCE AND MONITORING

16.1 Monitoring

Monitoring requirements for the 2/5 Dam TSF will be relatively straight-forward due to the configuration of the storage. There is a system of sub-regional groundwater monitoring bores installed around the proposed TSF [Ref. 17]. Some of these monitoring bores will be decommissioned as they fall within the footprint of the proposed facility. Additional monitoring bores will be installed downstream of the facility as part of the overall groundwater monitoring strategy. It is proposed that these will be monitored on a monthly basis.
The purpose of the existing and proposed additional bores will be to:

- Initially, provide data on the presence, depth, quality and flow direction of groundwater in the area.
- During TSF operation, provide an indication of broad scale groundwater fluctuations, such that any impacts on the groundwater regime brought about by the TSF can be monitored.

Given that the embankments will be constructed of free-draining rockfill with a synthetic lining on the upstream face, a phreatic surface will not develop within the embankments, even if a concentrated leak in the lining system were to occur.

Standpipe piezometers will hence not be required in the completed embankments, as any seepage would flow along the foundation, and accumulate in the Seepage Collection Pond downstream of the northern embankment. The Seepage Collection Pond will be inspected as part of routine weekly surveillance.

Survey monuments will be installed on the crest and downstream berms of each stage of the embankments to monitor settlement of the fill materials. It is envisaged that these monuments will be surveyed on a quarterly basis.

Weirs will be installed at strategic locations within drains located downstream of the embankments. This is to allow monitoring of seepage flow and rainfall runoff. It is envisaged that these will be monitored on a monthly basis.

16.2 Surveillance

Surveillance requirements for the TSF will involve routine daily, weekly and monthly inspections, as well as mandatory annual audits in accordance with ANCOLD guidelines [Ref. 8]. The focus of such surveillance will be as follows:

Daily Inspections -
Focus on operational issues to do with the TSF, including inspections of the tailings pipelines, discharge point management, Seepage Collection Pond and decant system operation.

Weekly Inspections -
Focus on issues that may develop over time and may impact on the safety of the TSF or the environment. These include detailed inspections of the embankment and all appurtenant structures, tailings deposition and decant pond level.

Monthly Inspections -
Focus on surveillance of all monitoring installations, except settlement monuments that will be surveyed quarterly.

Annual Audits -
These are conducted by a qualified dam engineer, and focus on the identification of deficiencies by visual examination of the TSF embankments and all appurtenant structures, as well as a review of all surveillance and monitoring data.
16.3 Operation and Maintenance Manual

An Operation and Maintenance Manual will be prepared for the TSF in accordance with the regulatory requirements for High Consequence Category tailings facilities. This will include instructions and forms to cover all necessary monitoring, daily, weekly and monthly routine inspections and surveillance activities.

Tailings deposition and decant management procedures will also be documented. The Manual will contain a section on emergency response, and will be an integral part of the overall risk management plan for the facility itself and the mine overall. A detailed inundation plan will need to be prepared and be included as part of the emergency response plan.

The manual will supersede the current Operation and Maintenance Manual that details inspection and surveillance activities associated with the Bobadil TSF and existing 2/5 facility.

17 CONSTRUCTION

17.1 General

Construction of the 2/5 Dam TSF and associated SCP can be separated into access construction, clearing works, foundation works associated with improving the foundation conditions, earthworks associated with embankment construction and diversion drains. These construction activities could potentially be undertaken concurrently.

17.2 Timeframe

Foundation preparation works as well as water management at the site during construction will be the main management consideration. Due to the nature of the site; high and regular rainfall; construction will need to be programmed such that the potential for turbid water leaving the site is minimised. This will involve:

- Construction of the SCP and permanent diversion drains as soon as practicable after commencing Stage 1 construction;
- Where possible construction of temporary sumps for dewatering excavations;
- Installation of silt fences in certain areas;
- Installation of wash down facilities at the site entrance;
- Minimising the extent of cleared areas during initial works; and
- Construction of stilling basins.

In addition to the above, it will be necessary install a means of returning water from the facility to the ETP because the existing infrastructure will be decommissioned. This can either be in the form of constructing the new decant and return water pipeline at the commencement of works or installation of temporary pumps and pipes to achieve this.

Due to the complex nature of the site and the requirement for foundation improvement and cut-off walls associated with the western and northern embankments, and construction staging associated with the eastern embankment construction sequencing will be critical to completion of the works. The following provides an overview of the construction sequence for each embankment:
• Early works:
  o Removal of existing site power and installation of new overhead wires to the proposed SCP.
  o Demolition of infrastructure, including pump stations and pipework.
  o Upgrade site access, including installation of wash down facilities, waste material storage, and development of site amenities, potable water and power.
  o Removal of water from the surface of 5 Dam.
  o Installation of security fencing.
  o Install vibrating wire piezometers within the tailings of 5 Dam.

• Ancillary works:
  o Construct decant and flushing water infrastructure.
  o Construct decant return pipeline.
  o Construct clean water diversion drain south of the TSF.

• Western Embankment:
  o Clearing works associated with the dam alignment, screening wall, diversion channel and spillway area.
  o Removal of 1 Dam drainage.
  o Grouting of rock around the western embankment.
  o Placement of pioneer layer of rockfill above excavation of overburden across 1 Dam.
  o Install GSS across the pioneer layer and 1 Dam area and anchor to grouted rock.
  o Placement of rockfill ballast to GSS layer across 1 Dam area.
  o Construction of screening wall.
  o Construction of rockfill embankment.
  o Installation of upstream GSS face lining.

• Northern Embankment:
  o Clear and grub vegetation from the footprint of the embankment.
  o Construct the SCP and modify existing drainage to direct surface runoff and seepage to the new facility. Construction to include installation of pumps and connection to power.
  o Backfill the existing seepage collection pond.
  o Construct the initial 2 m of the downstream buttress.
  o Install slurry cut-off wall and grout rock foundations.
  o Install dewatering sumps at the existing downstream toe of 2 Dam.
  o Shape and bench the downstream slope of the existing 2 Dam.
  o Place bedding layer on the shaped downstream face of the 2 Dam. Bedding to be placed to the first bench.
  o Install GSS between the existing downstream toe and first bench of the downstream slope of the 2 Dam and anchor at first bench.
  o Complete the downstream buttress and place rockfill to the first bench height.
  o Install bedding layer and GSS to the existing 2 Dam embankment crest, anchoring the GSS at intermediate benches and the existing crest.
  o Place rockfill to construct the embankment to full height.
  o Install GSS lining system on the upstream face between the existing embankment crest and new embankment crest.

Rockfill and bedding material for this embankment will be constructed from material produced by quarrying, crushing and screening (as required) from the diversion drain and spillway excavation, augmented by rockfill from the proposed quarry between 2 and 5 Dams.
• Eastern Embankment:
  o Clear and grub vegetation from embankment alignment, quarry areas and seepage collection drains.
  o Form seepage collection drains and associated access tracks.
  o Staged construction of embankment where the foundation is tailings, inclusive of installation of base GSS and associated bedding material.
  o Grouting of rock foundations
  o Construction of embankment where the foundation is natural ground (this can be undertaken between construction stages above).
  o Install GSS lining system on the upstream face of the completed embankment.
  o Grouting of the rock within the quarry between 2 and 5 Dam.
  o Installation of GSS lining across the surface of 5 Dam and anchor to the grouted rock within the quarry between 2 and 5 Dams.
  o Place ballast above GSS lining system.

Rockfill and bedding material for this embankment will be constructed from material produced by quarrying, crushing and screening (as required) from the proposed quarry between 2 and 5 Dams.

The works will likely be commenced towards the end of 2015 or early 2016. It will be crucial that all of the embankments have been commenced during the summer months of 2016 and that the GSS layers across 1 Dam, on the downstream face of the existing 2 Dam and the GSS lining located within the eastern embankment are completed prior to winter 2016. The seepage collection drains, SCP and clean water diversion drain should all be completed prior to winter 2017. Bulk rockfill can take place during the 2016 winter period, with final placement of the upstream GSS lining system completed during the 2016/2017 summer period. The civil works will need to be completed by the end of the first quarter of 2017 to allow sufficient time to install and commission the tailings distribution system. It should be noted that the tailings distribution system between the concentrator and the TSF site can be installed independently of the dam civil works.

17.3 Construction Considerations

17.3.1 Specialist Sub-Contractors

The design has been prepared on the basis of the complex site conditions. These conditions mean that aspects of the works will need to be completed by specialist sub-contractors. Activities that will require specialist contractors include:

• Quarry rock production - drill and blast, crushing and screening.
• GSS installation.
• Pipeline contractors.

17.3.2 Suitability of Materials for Construction Purposes

Proposed embankment construction materials are:

• Zone 3A - Rockfill - bedding material and upstream zone.
• Zone 3B - Rockfill

The investigation of the potential borrow area identified durable NAF felsic rock which, once quarried, crushed and screened (as required) will be suitable from a geotechnical and geochemical perspective.
Potential Zone 3A Material

Zone 3A material will be used as bedding for the GSS lining system as well as the upstream embankment. Zone 3A material will need to be produced from quarried rock. This material will be well graded with a maximum particle size of 50 mm.

Potential Zone 3B Material

Zone 3B material will consist of run of quarry rock and will be used for bulk embankment construction. The material shall have a maximum nominal particle size of 300 mm. Oversize material from production of Zone 2 material may be blended with this material.

17.3.3 Quality Control

17.3.3.1 Engineering Supervision

The site conditions are complex and will require the works to be carried out under full time engineering supervision. Suitably qualified site engineers will be required to provide technical oversight as well as control quality control of liner supervision.

17.3.3.2 Rockfill

Due to the nature of the works, formal quality control testing for density of the rockfill is not possible. Construction therefore will need to be completed to a method specification. Rockfill (Zone 3A and 3B) will be placed in a maximum loose layer thickness of 500 mm, or as identified via trial pads. This layer thickness may need to be reduced for the eastern embankment on the basis of performance of the embankment during construction.

Water will need to be added during the works to lubricate the rockfill. Water addition of up to 5% may be required.

17.3.3.3 Grouting Works and cut-off walls

Quality control for these activities will be governed by the specialist sub-contractor engaged for these works.

17.3.3.4 GSS Lining System

Due to the nature and complexity of the GSS lining system, these works will need to be carried out under full time engineering supervision. It is envisaged that the lining contractor will manage the installation with independent engineering verification required. A detailed report of liner installation, including material tracking, panel plans, install plans and, non-destructive and destructive testing will be required.

17.3.4 Mitigation of Sediment Transportation During Construction

During construction, the use of silt traps, sediment basins and other means of trapping sediment in cleared areas will be employed. The use of sediment control devices will be included within the Construction Environmental Management Plan (CEMP). The CEMP will provide the basis of the parameters to be monitored / managed and will provide a list of performance criteria by which conformance is measured.

There will be numerous traffic movement into and out of the site on a regular basis during the works. To manage the risk of sediment transportation outside of the site, a permanent wash down
facility will be installed at the site entry that is suitable for adequately removing sediment from light vehicles and road trucks. In addition, a heavy vehicle wash down pad will be constructed inside the works area to remove sediment from earthmoving equipment prior to leaving the site.

No vehicle or earthmoving equipment will be permitted to leave the site without being washed down.

17.3.5 Mitigation of Dust During Construction

Dust will potentially be an issue during the works, more so during the drier summer months. Water carts will be required to manage dust by:

- Routinely spraying haul roads; and
- Spraying quarry areas during high winds.

17.3.6 Introduced Species Management

The introduction of weeds, pests and diseases to the site during construction will be managed by ensuring that earthmoving equipment is cleaned before arriving, i.e. earth and vegetation is cleaned from equipment and treated with phytoclean prior to floating to site. Equipment will also need to be cleaned prior to leaving site.

Waste will be managed on site by providing bins and receptacles to minimise pests and vermin on site.

These items will also be included in the CEMP.

18 CLOSURE AND REHABILITATION

18.1 Overview

Tailings management for the 2/5 Dam is primarily aimed at dust minimisation and AMD mitigation. At cessation of operations the TSF will have a water cover of approximately 2 m. The closure concept for the 2/5 Dam TSF will incorporate a full water cover to minimise the potential for longer term oxidation of the tailings.

18.2 Closure Design Objectives

As discussed in Section 7.2, tailings will be potentially acid forming. Closure must therefore ensure that a suitable protective cover be placed over the tailings to inhibit the adverse environmental impacts of such potential acid generation. The cover must therefore achieve certain objectives:

(i) Control and minimise the transport mechanism either upwards or downwards for oxidation products within the tailings, and
(ii) Be able to safely discharge excess clean water runoff from the closure landform to the environment.
18.3 Closure Concept

The closure concept consists of:

- the flooding of the dam post closure so that a minimum permanent water cover depth of 2 m can be achieved;
- decommissioning the decant structure once acceptable water quality objectives have been met;
- breaching the clean water diversion drain (refer to Section 18.5 for rational);
- converting the area downstream of the northern embankment into a wetland to provide passive treatment of possibly impacted runoff;
- if required, addition of fill to embankment crests to address long term and seismic deformation settlements; and
- removal of all tailings pipelines.

It is considered that a minimum permanent water cover of 2 m will achieve object (i) above.

For the water cover closure concept it is assumed that at closure the maximum tailings elevation will be RL 169 m (the design maximum tailings elevation), which is 3 m below the Stage 2 spillway invert level of RL 172 m and 4 m below the Stage 2 crest elevation (RL 173 m). This would result in a 3 m deep water pond before discharge of water to the environment.

The Stage 2 spillway has been designed to safely pass the closure design storm of Probable Maximum Precipitation (PMP) as required by ANCOLD [Ref. 8]. A study conducted by BOM [Ref. 14] indicated that the 1:100,000 Annual Exceedance Period (AEP) storm is equivalent to the PMP. The spillway will therefore meet objective (ii) above.

After closure, the wetland area will provide a stilling area for rainfall runoff and any minor seepage from the northern and eastern embankment. It is anticipated, although not modelled, that the wetland will reach its field capacity soon after closure and thereafter will be a flow through system. The wetland would be designed (during detailed closure design) to cater for the runoff resulting from a design storm of between 1:10 AEP to 1:100 AEP but would be dependent on the volume of seepage emanating from the facility. A design storm runoff to seepage factor of at least 10 is considered appropriate for sizing of the wetland but less than the volume resulting from a 1:10 AEP design storm.

It should be noted that upon closure of the facility, water from the TSF will continue to be decanted to the ETP until water quality was determined to be suitable for direct discharge to the Stitt River, after which the decant will be decommissioned and water discharged via the Stage 2 spillway into the diversion drain located between the western and northern embankment and from there directly into the Stitt River.

The Closure Concept is presented as Figure 18.1.

18.4 Progressive Rehabilitation

The tailings will be deposited sub-aqueously, as a result, progressive rehabilitation of the TSF, other than possible removal of pipework no longer required will not be possible until discharge of tailings is complete.

Construction of the wetland area would be deferred for a period of time after cessation of tailings discharge to assess the seepage base flow so that correct sizing of the wetland can be determined.
18.5 Closure Water Balance

18.5.1 General

A stochastic closure water balance has been developed to identify whether there is sufficient rainfall at the site to meet the closure concept of a minimum permanent water depth of 2 m.

The aim of the closure water balance is to:

- Evaluate the water contained within the TSF to identify whether the tailings retain a 2 m cover of water.
- Identify likely spillway flow.

18.5.2 Methodology

Similar to the operational water balance, the closure water balance has been developed using the computer program GoldSim Software and uses the mass balance approach.

18.5.3 Model Components

The model is divided into three components:

- Inflows, consisting of:
  - Rainfall - direct onto the TSF water cover surface and runoff from the southern catchment area.

- Losses, consisting of:
  - Seepage losses through the tailings;
  - Infiltration from the southern catchment; and
  - Evaporation.

- Outflows, consisting of:
  - Spillway flow.

18.5.4 Climate Datasets

18.5.4.1 Synthesised Precipitation Data

The Stochastic Climatic Library (SCL) computer program has been used to generate a 1,000 year synthetic daily precipitation dataset for use in the water balance. The 100 year daily precipitation database used for the operational water balance was used as the basis for the synthetic dataset. The statistical characteristics of the 100 year dataset including mean, variance, skewness and long-term persistency were calculated by the SCL program. These statistical characteristics were then used as a basis to produce a synthetic dataset with the same statistical characteristics as the original precipitation data set.

Using SCL, the 100 year data set was replicated 10 times in order to generate 1,000 years of synthetic data. Each individual replicate set will have different characteristics than the original input set, but the average of all the replicate sets combined will have the same statistical characteristics as the original set.
On this basis, the following precipitation data have been derived for the SCL model:

- Average Annual Precipitation: 1,985 mm/year
- Maximum Precipitation: 3,114 mm/year
- Minimum Precipitation: 1,078 mm/year

18.5.4.2 Evaporation Data

The evaporation dataset used for the closure water balance is the same as that used for the operational water balance (refer Section 14).

18.5.5 Model Inputs and Losses

18.5.5.1 Rainfall

Rainfall has been modelled for the following model components:

- Direct rainfall onto the TSF water surface; and
- Rainfall onto the southern catchment area.

Rainfall is unfactored on the TSF water surface but is routed through the AWB model for runoff from the surrounding catchment as described below.

18.5.5.2 Evaporation

Evaporation is applied to the TSF water surface only. A conservative pan factor of 1.0 has been applied to evaporation.

18.5.5.3 Runoff from the Southern Catchment Area

Runoff has been modelled for two cases as follows:

- Case 1 - The clean water diversion is retained, thus the external catchment area runoff reporting to the TSF is between the TSF and the clean water diversion drain.
- Case 2 - The clean water diversion drain is breached at closure, thus the external catchment area runoff reporting to the TSF consists of the entire Southern catchment area.

These cases give external catchment areas of 13 Ha and 71 Ha respectively.

Runoff from this catchment has been modelled using the AWB model.

18.5.5.4 Seepage Loss through the Tailings

Seepage losses through the tailings have been based on the tailings coefficient of permeability derived from:

- CPT testing performed on the tailings at the existing 5 Dam in February 2015, further details of which can be found in ATCW geotechnical report [Ref. 16].
- Rowe Cell testing of tailings samples from the South Marionoak study in 2008 [Ref. 3].

The sensitivity of the tailings coefficient of permeability has been based on the above test results has been assessed with the water balance. The range of coefficient of permeability from the above testing is summarised in Table 18.1.
TABLE 18.1
TAILINGS COEFFICIENT OF PERMEABILITY TEST RESULTS

<table>
<thead>
<tr>
<th>Testing Type</th>
<th>Coefficient of Permeability (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT (2015)</td>
<td>$6.1 \times 10^{-7} - 7.7 \times 10^{-8}$</td>
</tr>
<tr>
<td>Rowe Cell (2008)</td>
<td>$1.1 \times 10^{-7} - 6 \times 10^{-8}$</td>
</tr>
</tbody>
</table>

Based on these results, tailings coefficient of permeability ($k$) values between the range of $1.1 \times 10^{-7}$ m/s and $2.0 \times 10^{-8}$ m/s have been modelled within the water balance, where $k = 1.1 \times 10^{-7}$ m/s is considered the lower bound and hence most conservative case.

However, the field coefficient of permeability achieved for the adopted sub-aqueous tailings deposition method will need to be confirmed through permeability testing closer to the end of the operational life via insitu testing techniques.

The coefficient of permeability value of $1.1 \times 10^{-7}$ m/s results in an average seepage loss of $2,500 \text{ m}^3/\text{day}$ for Case 1 (with diversion drain) and $4,000 \text{ m}^3/\text{day}$ for Case 2 (diversion drain breached). The lower seepage loss of Case 1 is a result of the pond being empty for a significant portion of the mine life for this permeability value.

18.5.6 Model Runs

The water balance was run using the 1,000 years of generated synthetic rainfall data. The model was run for a 100 year cycle (realisation). In total, 10 realisations were used to model the 100 year long runs, spaced 100 years apart in order to cover the entire range of synthetic data. For example, realisation 1 models years 1-100, realisation 2 models years 101-200, realisation 3 models years 201-300 and so forth.

18.5.7 Results

18.5.7.1 General

The key outputs from the water balance model are:

- TSF water cover depth; and
- An estimate of the likely spillway flow.

The former is to identify whether the closure design objective can be met and the latter for water released to the environment.

18.5.7.2 TSF Pond Water Depth

As discussed above, various tailings coefficient of permeability values have been investigated to observe the effect on water depth. Greater tailings coefficient of permeability ($1.1 \times 10^{-7}$ m/s) will result in higher seepage losses, and thus a reduced pond volume. Conversely, lower tailings coefficient of permeability ($2.0 \times 10^{-8}$ m/s) will result in lower seepage losses and an increase in pond volume.
These values have been trialled over the two cases of the diversion drain being retained in closure and the diversion drain being breached at closure. Figure 18.2 presents the results of the case where the clean water diversion drain is in place. Figure 18.3 presents the results of the case where the diversion drain has been breached. The results of these figures are summarised below:

Case 1 (with diversion drain retained in closure):

- $2.0 \times 10^{-8}$ - Tailings maintain a minimum permanent water cover greater than 2 m depth, and hence meet the closure concept objectives.
- $6.0 \times 10^{-8}$ - Tailings maintain a permanent water cover, however this only averages 1.5 m depth, and hence does not meet the closure concept objectives.
- $1.1 \times 10^{-7}$ - Tailings periodically dry out.
- $2.0 \times 10^{-7}$ - Tailings periodically dry out.

Case 2 (diversion drain breached at closure):

- $6.0 \times 10^{-8}$ - Tailings maintain a minimum permanent water cover greater than 2 m depth.
- $1.1 \times 10^{-7}$ - Tailings maintain a minimum permanent water cover greater than 2 m depth 96% of the time over the 100 year model. After initial raising of the water level at closure, the permanent water depth remains greater than 2 m in all instances except for two years. On these two occasions the minimum permanent water depth is approximately 1.9 m, suggesting that this case generally meets the closure objectives.
- $2.0 \times 10^{-7}$ - Tailings periodically dry out.

From these results it can be concluded that the model is extremely sensitive to small variations in the tailings coefficient of permeability.

Depending upon the findings of insitu testing, additives to the tailings stream (modified tailings) may be required in the latter stages of the mine life to reduce the tailings coefficient of permeability.

Depending of the measured coefficient of permeability, the additives to the tailings stream would possibly be clay type particles at an addition rate of 3% to 5% in order to reduce the coefficient of permeability to around $1.1 \times 10^{-7}$ m/s. The clay type particles could be slurred natural clay soils added to the tailings tank (upstream of the tailings pumps) or possibly powdered bentonite. The requirements for additives and, if required, addition rates would need to be determined at least 2 years prior to mine closure to allow sufficient time to modify tailings infrastructure (tailings tank, assess pump power, PLC, etc.) so that a minimum depth of modified tailings of 500 mm can be placed across the tailings surface.

The addition of fine grained material to improve the engineering parameters of tailings, specifically the coefficient of permeability of tailings has been proven in laboratory tests [Ref. 3]. The addition of clay the particles occupies the void space between the tailings particles and hence reduced the overall coefficient of permeability.

18.5.7.3 Spillway Discharge

Spillway discharge has been assessed for the case where the clean water diversion drain has been breached as it is considered that the case where the drain is retained at closure will not achieve the concept objectives. A summary of the average daily spillway flow for all of the modelled cases is presented in Table 18.2.
### TABLE 18.2

**AVERAGE SPILLWAY FLOW**

<table>
<thead>
<tr>
<th>Diversion Case</th>
<th>Tailings Coefficient of Permeability (m/s)</th>
<th>Average Spillway Flow (m³/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Clean Water Diversion Drain Retained at Closure</td>
<td>2.0 x 10^{-7}</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>1.1 x 10^{-7}</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>6.0 x 10^{-8}</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>2.0 x 10^{-8}</td>
<td>1,087</td>
</tr>
<tr>
<td>Clean Water Diversion Drain Breached at Closure</td>
<td>2.0 x 10^{-7}</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>1.1 x 10^{-7}</td>
<td>398</td>
</tr>
<tr>
<td></td>
<td>6.0 x 10^{-8}</td>
<td>2,184</td>
</tr>
</tbody>
</table>

The lower bound case of 1.1 x 10^{-7} m/s and the clean water diversion drain breached is presented as Figure 18.4.

The spillway has been designed to meet ANCOLD criteria, this being to be able to pass the Probable Maximum Precipitation (PMP) rainfall event for a High Consequence Category dam during operations. This design criteria is also applicable to the closure case. The spillway inlet dimensions are 20 m wide by 1 m deep. For this cross section, the maximum flow depth for the above discharge is less than 0.1 m.

### 18.6 Detailed Design of TSF Closure System

The above closure system is conceptual for the purposes of this Report, and is considered to be commensurate with current best practice and regulatory requirements for equivalent project configurations and climatic conditions.

Analyses for long term and seismic (MCE) deformation will need to be determined and additional fill placed, as required, to ensure deformation does not affect the hydraulic capacity of the structure.

The results of the water balance modelling show that it is possible for a permanent water cover to be achieved and maintained at a minimum permanent depth of 2 m under certain conditions.

With respect to the detailed design of the final capping system for the TSF, commitment will be made to undertake the following:

- Undertake permeability testing of the tailings at multiple locations towards the end of the operational life of the TSF in order to more accurately assess whether a 2 m water depth wet closure is feasible.

In addition to the above, alternate closure options, such as dry closure will be investigated during the operational phase of the TSF. This would include, but not be limited to the following:

- Numerical infiltration modelling and/or capping field trials which would be conducted as part of the closure of the Bobadil facility (current TSF used to store tailings). This would provide a better understanding of the tailings characteristics and the types of cover materials that would be required for this type of closure scenario.
The purpose of this type of modelling and/or field trials would be to determine the composition of a dry capping solution; depth of the capping layers and material requirements. The depth of the cover must be sufficient to limit infiltration into the underlying tailings, whilst providing a generally stable moisture regime within the cover year-round.

- Monitor developments in capping research and regulatory requirements. The design of closure systems have evolved significantly over the last 20 years. As new techniques are trialled, demonstrable evidence of their satisfactory performance takes years to develop.

It is hence considered sound design policy, given that closure of the facility TSF will not commence for at least a further 7 years, to conduct ongoing research of relevant closure experiences, and to monitor the advancements in the subsequent regulatory requirements. This will provide valuable input to the numerical modelling and/or field trials outlined above.

19 QUANTITIES

19.1 Civil Works Quantities

Schedules of estimated quantities for the two stages of the TSF and SCP construction works are presented in Appendix G. The schedules include all embankments, civil works and mechanical works items described in this Report.

A summary of quantities relating to civil earthworks construction is presented in Table 18.1.

| TABLE 18.1 |
| SUMMARY OF EMBANKMENT CIVIL EARTHWORKS QUANTITIES |

<table>
<thead>
<tr>
<th>Stage</th>
<th>Rockfill (m$^3$)</th>
<th>Geosynthetic Liner (m$^2$)</th>
<th>Spillway/Diversion Channel Excavation (m$^3$)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Stage 1</td>
<td>814,500</td>
<td>174,000</td>
<td>183,500</td>
</tr>
<tr>
<td>Stage 2</td>
<td>245,000</td>
<td>12,000</td>
<td>0</td>
</tr>
<tr>
<td>Total</td>
<td>1,059,500</td>
<td>186,000</td>
<td>183,500</td>
</tr>
</tbody>
</table>

It is evident that the start-up (Stage 1) construction of the TSF embankment works will require approximately 666,500 m$^3$ of rockfill. Rockfill will be sourced from the excavation of the spillway and diversion drain as well as from dedicated quarry areas between the existing 2 and 5 Dams. Total quantities to complete Stage 2 involve the placement of 225,500 m$^3$ of material which will be sourced from quarry areas located towards the southern portion of the TSF impoundment.

20 CLOSURE

Your attention is drawn to the “Conditions of Investigation and Report” which appear after the document history and status page of this report.
REFERENCES


