ATC WILLIAMS

REPORT

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Costerfield Gold Mine

Brunswick West Tailings Storage Facility Investigation and Design

Credible Failure Mode Assessment

109014.15-R03-1
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EXECUTIVE SUMMARY

This report presents the Credible Failure Mode Analysis (CFMA) for the Brunswick West Tailings Storage Facility (TSF) at Mandalay Resources Costerfield Operations (MRCO) in Costerfield, Victoria. This assessment has been conducted to support the Brunswick West TSF detailed design in conjunction with the dam break assessment.

This assessment had been undertaken following the recommendations of ANCOLD, and has been conducted to identify potential failure modes and progression paths that would result in a catastrophic failure of the TSF leading to a release of tailings and water, and quantify the likelihood of failure. The likelihood of failure will then be used to inform the level of risk associated with each failure mode.

The CFMA involves a review and assessment of potential failure modes for the TSF embankment. For the purposes of this study, a critical failure was considered to be one that could result in loss of impoundment material and / or initiate a catastrophic failure of the embankment that would directly place mine personnel or persons downstream at risk. The aim of the CFMA is to classify all potential failure modes, as well as assess the overall level of risk of each failure mode in consideration of the potential consequences of failure, as identified from the dam break assessment conducted to support the Brunswick West TSF design.

The CFMA assessment involves the initial identification of all potential failure modes that may exist, including the documentation of the failure progression path and the controls in place to mitigate a potential failure. Following the identification, a qualitative screening process is undertaken to rule out failure modes that are considered non-credible, followed by a quantitative assessment to estimate the likelihood or probability that a failure may occur.

The potential failure modes that were identified in the qualitative screening assessment as potentially credible (to be included in the quantitative assessment) are as follows:

- Geotechnical Piping through embankment - Cracking caused by loss of support from downstream shoulder.
- Geotechnical Piping through embankment - Cracking caused by loose/poorly compacted layers in core.
- Geotechnical Piping caused by transverse seismic cracking.
- Geotechnical Piping into foundations
- Embankment Overtopping due to loss of spillway capacity
- Embankment Overtopping due to crest scour from pipeline burst
- Embankment Overtopping due to poor deposition management - Spillway Blockage
- Embankment Overtopping due to poor deposition management - Decant Blockage
- Embankment Overtopping due to poor deposition management - Over deposition
- Embankment Overtopping due to build-up of excess tailings bleed water
- Embankment Overtopping due to higher than expected operating pond levels
- Embankment Overtopping due to single/multiple large storms that exhaust freeboard and exceeds spillway capacity
- Embankment Overtopping due to reduced spillway capacity from seismic induced crest settlement
- Embankment Overtopping due to scour from failure of Spillway Erosion Protection Rip-Rap
- Embankment Instability due to incorrect material characterisation – Embankment fill materials
- Embankment Instability due to incorrect material characterisation - Foundation materials
- Embankment Instability due to high phreatic surface

- Embankment Instability due to inadequately constructed embankments
- Embankment erosion failure due to cumulative static settlement and seismic deformation

The estimated probability each failure modes for each potentially credible failure mode, along with the assessed consequences of failure (assessed as part of the dam break assessment) are summarised in **Table ES1**.

TABLE ES1: ASSESSED POTENTIALLY CREDIBLE FAILURE MODES

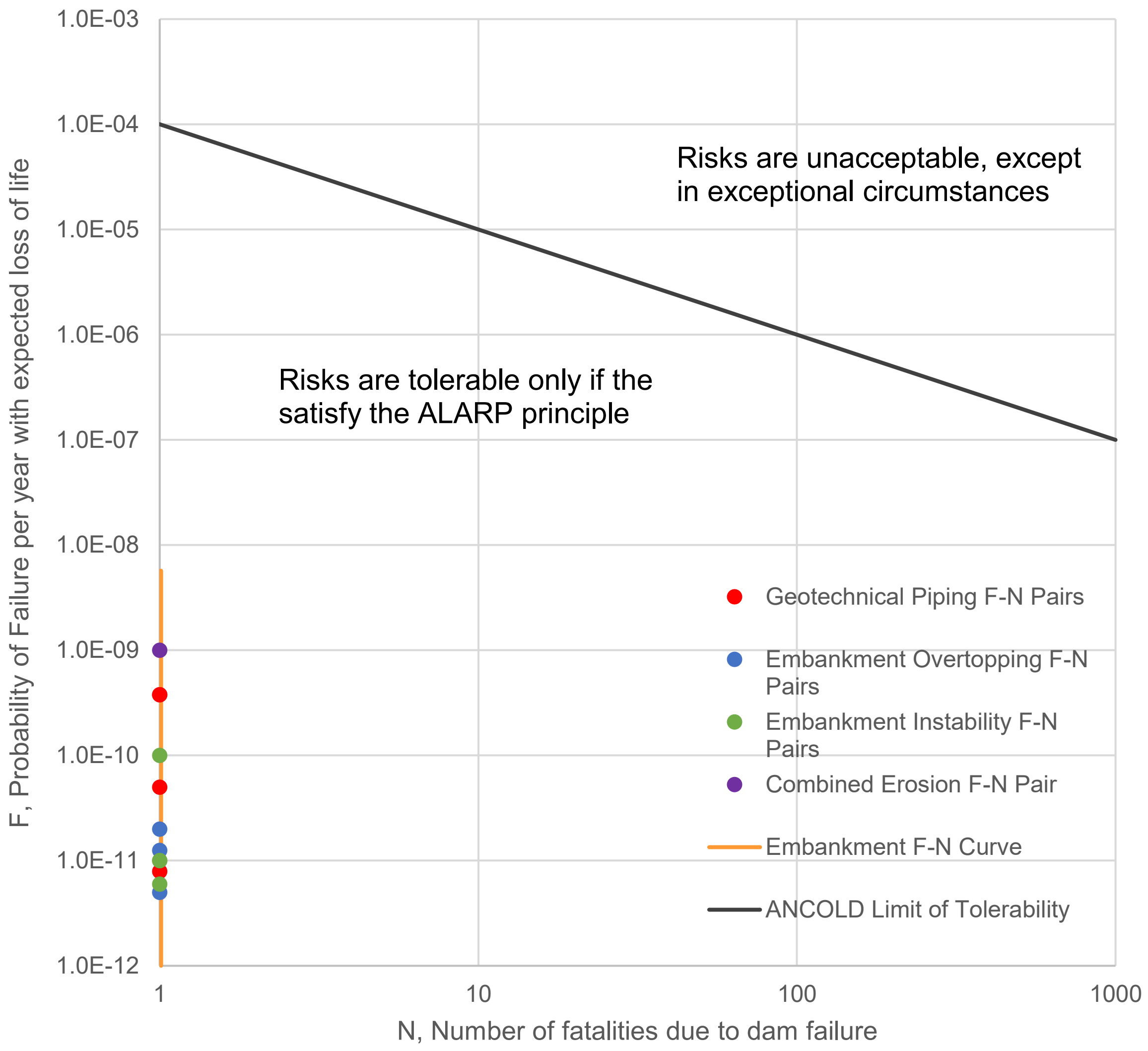
Failure Mode	Estimated Probability	Consequences of Failure (PLL)
Geotechnical Piping		
Through embankment - Cracking caused by loss of support from downstream shoulder	3.8×10^{-10}	Sunny Day Failure PLL = 1
Through embankment - Cracking caused by loose/poorly compacted layers in upstream clay zone	7.9×10^{-12}	
Through embankment – Cracking caused by transverse seismic cracking	5.0×10^{-11}	
Into foundations	3.8×10^{-10}	
Embankment Overtopping		
Loss of spillway capacity	1.3×10^{-9}	Flood Failure PLL = 0.01
Crest scour from pipeline burst	5.0×10^{-10}	
Poor deposition management - Spillway Blockage	>> 1.0×10^{-10} (1)	
Poor deposition management - Decant Blockage	>> 1.0×10^{-9} (1)	
Poor deposition management - Over deposition	>> 1.0×10^{-8} (1)	
Build-up of excess tailings bleed water	>> 1.0×10^{-10} (1)	
Higher than expected operating pond levels	>> 1.0×10^{-10} (1)	
Single/multiple large storms that exhaust freeboard and exceeds spillway capacity	>> 5.0×10^{-11} (1)	
Reduced spillway capacity from seismic induced crest settlement	>> 1.0×10^{-10} (1)	
Scour from failure of Spillway Erosion Protection Rip-Rap	2.0×10^{-9}	
Embankment Instability		
Incorrect material characterisation - Embankment fill materials	1.0×10^{-11}	Sunny Day Failure PLL = 1
Incorrect material characterisation - Foundation materials	1.0×10^{-11}	
High phreatic surface	1.0×10^{-13}	
All combined design element failures above	3.0×10^{-13}	
Inadequately constructed embankments	1.0×10^{-10}	
Inadequately prepared foundations	6.0×10^{-12}	
Combined Settlement and Erosion Failure		
Erosion failure due to cumulative static settlement and seismic deformation	1.0×10^{-9}	Sunny Day Failure PLL = 1

Note (1) Failure modes requiring storm events larger than the PMP denoted with ">>" suffix, indicating the probability is well below this assessed value.

The estimated probabilities of failure are very low, owing to the numerous conservative design, construction and operation controls that have been (or will be) implemented for the Brunswick West TSF.

The ANCOLD Guidelines on Risk Assessment provide guidance for the tolerability of public safety risks for the general community and workers associated with the facility, and considers the relationship between the annualised probability of failure and the potential number of fatalities due to dam failure. This is presented as an F-N chart, or the ANCOLD Societal Risk Guidelines. This chart presents the individual F-N pairs for each of the failure cases considered, as well as the combined F-N curve for the Brunswick West TSF, and is presented in **Chart ES1**

CHART ES1: ANCOLD SOCIETAL RISK GUIDELINES



Note: ANCOLD provides the Limit of Tolerability line to a lower bound N value of 1. For fractional N values (i.e., N < 1), the F-N pairs have been presented at N = 1, with the probability values adjusted to compensate.

The estimated F-N pairs for the assessed failure cases and the cumulative embankment F-N curve plot significantly below the ANCOLD Limit of Tolerability for new dams, and are within the region where the risk can be considered as tolerable if they satisfy the ALARP principle.

The key ALARP considerations for the proposed Brunswick West TSF are summarised below:

- The residual societal risk for the Brunswick West TSF design is roughly 4 orders of magnitude below the ANCOLD Limit of Tolerability for new dams.

- The proposed design for the Brunswick West TSF satisfies best practice design for the facility, and has been designed to meet and exceed the minimum requirements of ANCOLD. Key risk control measures from the TSF design are summarised below;
  - Inclusion of the BGM liner to control initiation for geotechnical piping.
  - Formation of the embankment downstream slopes to 4:1 (H:V), resulting in a significant Factor of Safety against embankment instability.
  - Excavation of the emergency spillway beyond the depth of the PMP peak flood height to control water overtopping the embankment crest.
- The proposed design also incorporates two flood diversion bunds to aid in the prevention of material (tailings and/or water) inundating the Brunswick Underground Portal in the events of a dam failure to remove the PLL from the Dam Break scenario. While the additional costs to construct these bunds is not insignificant, these flood protection measures have been considered as necessary to ensure the safety of the mine workers within the underground network.
- Construction of the Brunswick West TSF is proposed to have full time construction QA/QC to ensure the design specifications are met, including foundation preparation, material specifications, and material placement and compaction.
- An Operation, Maintenance and Surveillance Manual (OMS) will be prepared and implemented prior to commissioning of the Brunswick West TSF, in accordance with the operating requirements of ANCOLD.
  - This document will describe the routine inspections required for key elements of the facility, and subsequent actions to be conducted to limit the potential progression of embankment failure.
  - Surveillance and instrumentation monitoring requirements will also be documented, to be conducted regularly, and reviewed annually as part of the dams engineer inspections.
- A Dam Safety Emergency Plan (DSEP) will also be prepared and implemented prior to commissioning of the Brunswick West TSF, in accordance with the operating requirements of ANCOLD.
  - This document will describe the procedures to be followed by personnel in the inundation zone based on trigger levels of key elements of the facility. These trigger levels are defined on a traffic-light system, from lowest (within normal operating levels) to highest (failure is imminent), and provides instruction on the required actions to take at different levels.
  - Timely evacuation of mine personnel from the Brunswick Processing Plant area when approaching the highest trigger level will further reduce the PLL in the event of a Dam Break .
  - As part of the operation of the facility, the DSEP will be regularly tested to ensure mine personnel within the inundation zone are sufficiently trained on the actions to take at different trigger levels.
- The incremental costs and level of effort associated with further engineering risk reduction works for the proposed Brunswick West TSF would be significant compared to the relatively small to negligible further risk reduction that could be achieved.

As such, ATCW consider that the design of the Brunswick West TSF satisfies the ALARP principle, and that the residual risks imposed to the community and mine personnel due to the presence of the Brunswick West TSF are tolerable, and the facility meets the requirements of ANCOLD for the management and tolerability of risk.

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1 INTRODUCTION

This report presents the Credible Failure Mode Analysis (CFMA) for the Brunswick West Tailings Storage Facility (TSF) at Mandalay Resources Costerfield Operations (MRCO) Costerfield Gold Mine, located approximately 8 km north-east of the township of Heathcote in Victoria, Australia.

This study has been undertaken following the recommendations of ANCOLD, and has been conducted to identify potential failure modes and progression paths that would result in a catastrophic failure of the TSF leading to a release of tailings and water, and quantify the likelihood of failure. The likelihood of failure will then be used to inform the level of risk associated with each dam failure mode. This assessment has been conducted to support the TSF detailed design, documented in ATCW report 109014.15-R04-Rev 3 **[1]**.

These works have been undertaken following discussions between Mr Shannon Green and Mrs Annabel Meagher of MRCO, and Alexander Campbell and Craig Noske of ATCW, and in general accordance with ATCW budget variation 109014.15-L01, dated 21st September 2022.

This document presents Revision 1 of the Credible Failure Mode Assessment Report. Changes made within this document as part of the Revision 1 update are presented in Red.

2 BACKGROUND

2.1 Site Description

The Costerfield Mine is an underground gold and antimony mining operation. The site currently has two tailings storage facilities:

- Bombay TSF (currently receiving tailings), located 500 m north of the Brunswick Processing Plant, and
- Brunswick TSF (at capacity, deposition recently ceased), located to the immediate east of the Brunswick Processing Plant.

A locality plan of the Costerfield Gold Mine is presented in **Figure 1**, with the general Brunswick West TSF area shown in **Figure 2.**

The site of the new Brunswick West TSF is located approximately 500 m north-west of the Brunswick Processing Plant, within an adjacent farm paddock. The site is roughly triangular, and is confined by Crown Land to the east, MRCO infrastructure and additional farmland to the south, and Bradleys Lane to the west.

The site has a ridge at approximately RL 194.0 m in the centre of the paddock running in a south-easterly direction, from which the natural ground slopes to the north, east and south at a natural grade of up to 5%. The site currently contains two farm water dams at the east and south, the latter of which is situated within a natural drainage channel.

The site currently contains farm infrastructure, trees, and a high voltage Single Wire Earth Return (SWER) powerline. Additionally, the boundary of the Brunswick West TSF extends partially into the MRCO Low Grade Run Of Mine (ROM) pad. This infrastructure is all expected to be removed prior to commencement of construction.

2.2 Facility Investigation and Design

2.2.1 Geotechnical Investigations

To facilitate the design of the Brunswick West TSF, ATCW conducted a geotechnical investigation of the site to characterise the site and collected samples to further characterise the in-situ foundations of the TSF. The objectives of the investigations were:

- Characterise the sub-surface strata within the embankment foundations and TSF impoundments;
- Determine the in-situ properties of the natural clays and rock within the embankment foundations;
- Assess the material properties of the clays and rockfill within the proposed TSF impoundment excavation.

The results of the investigation and laboratory testing are documented in ATCW report 109014.15-R01 **[2]**.

2.2.2 Facility Design

The TSF embankments will be constructed to RL 200.0 m, which was inferred as the maximum allowable height of the facility based on the current approved elevation limit for the nearby Bombay TSF (RL 200.4 m). Construction of the Brunswick West TSF will generally consist of the following:

- Early works preparation of the site, including the removal of all unsuitable existing infrastructure and vegetation,
- Preparation of the embankment foundations, including stripping of topsoil and upper clays, and re-compaction of the in-situ clays,
- Excavation of the impoundment area to a base elevation of between RL 186.0 m and 180.0 m. Suitable clays and all excavated rockfill will be stockpiled and conditioned as required for use in embankment and clay liner construction,
- Placement and compaction of a 1.0 m thick Zone 1 clay liner and 0.3 m thick rockfill protection layer (select Zone 3A) over the base of the impoundment excavation,
- Construction of a herringbone underdrainage network over the lined base, discharging into a collection sump at the southern end,
- Construction of a 3.0 m wide compacted Zone 1B earthfill zone to act as the BGM liner subgrade up the excavated batter walls to natural surface level,
- Construction of the perimeter embankments, consisting of a downstream Zone 3B rockfill shoulder to a minimum crest width of 3.0 m, a 5.0 m wide compacted Zone 3A transition fill , and a 3.0 m wide Zone 1B earthfill subgrade on the upstream side (following the profile of the Zone 1B earthfill subgrade up the excavated batter walls),
- Construction of the inclined decant structure installed at the south-west corner of the facility,
- Excavation of an (over-crest) emergency spillway at the southern end of the facility,
- Installation of the Bituminous Geomembrane (BGM) Liner on the upstream face of the embankment and impoundment Zone 1B earthfill subgrade batters,
- Placement of suitable road-base material along the crest of the embankment,
- Construction of access ramps to the crest of the embankment,
- Construction of flood diversion bunds around the southern and western perimeter of the Brunswick Underground Portal,
- Excavation of clean water diversion drains around the downstream toe of the embankment,
- Installation of tailings delivery pipelines, decant removal pipelines and pumps,
- Installation of Groundwater Monitoring Bores (GMBMs),
- Construction of the external Return Water Pond (RWP), including
  - Excavation of suitable construction materials,
  - Construction of embankments,

- Lining with synthetic Liner.

A layout plan of the facility is provided in **Figure 3**, with typical embankment sections provided in **Figure 4**. Design of the Brunswick West TSF is documented in ACTW report 109014.15-R04-Rev 3 **[1]**.

2.2.3 Embankment Geometry

A summary of the TSF embankment geometry is presented in **Table 1,** and is presented visually in **Figure 4**.

TABLE 1: SUMMARY OF TSF EMBANKMENT GEOMETRY

Item	Criteria
Foundation Treatment	Natural surface - Stripping 0.5m topsoil and 0.5m natural clay, and re-compaction of residual clays to 98% Standard Maximum Dry Density (SMDD). Disturbed foundations (ROM Pad and Existing Farms Dams) – Stripping of all material to expose weathered rock foundations, and re-compaction of weathered rock to 98% SMDD.
Crest Elevation	RL 200.0 m (AHD)
Crest Width	6.0 m
Crest Treatment	Covered with minimum 100mm gravelly road base material, and graded into the TSF.
Upstream Batter Slope	2:1 (H:V)
Downstream Batter Slope	4:1 (H:V)
Embankment Height	Max 14.0m
Embankment Internal Geometry	From upstream to downstream; • Bituminous Geomembrane Liner (BGM) • Zone 1B - Gravelly Clay/Clayey Gravel Earthfill Subgrade – uniform 3.0m wide, extends to crest level. • Zone 3A - Weathered Rockfill/Earthfill Transition zone – 5.0 m wide to RL 199.0 m, 3.0m wide at crest level • Zone 3B - Downstream Rockfill Shoulder – 3.0 m wide at RL 199.0 m. • Zone 3A – Weathered Rockfill/Earthfill Transition Zone – 1.0m wide at RL 199.0 m to support topsoil placement

2.2.4 Embankment Materials

A summary of the Brunswick West TSF embankment construction materials are provided below. A detailed description of these materials are further discussed in the design report **[1]**.

Bituminous Geomembrane (BGM) Liner

The upstream liner is an elastomeric modified BGM of nominal minimum thickness 4.0 mm. The BGM will be installed on the upstream face of the embankment, and anchored at the crest and toe of the embankment.

Zone 1B Earthfill Subgrade

Zone 1B is an earthfill material, comprising of a mixture of Gravelly Clay, Clayey Gravel, and Residual & Extremely Weathered rock, with a maximum particle size of 30mm.

The material will be placed in 300mm (loose) layers and compacted to achieve a minimum 98% SMDD. The upstream face of the material will also be trimmed to form a smooth subgrade such that the maximum asperities are no more than 30mm.

Zone 3A Transition Fill

Zone 3A is an earthfill/rockfill mixture, comprising of a mixture of Residual Soil, Extremely and Highly Weathered siltstone and sandstone particles in a matrix of clays, silts, sands, and gravels, with a maximum particle size of 300mm.

The material will be placed in 300mm (loose) layers and compacted to achieve a uniform, compacted mass. The level of compaction will be determined on site from field trials to determine the compactive effort required.

Zone 3B Rockfill

Zone 3B is a general rockfill mixture, comprising of durable, highly to slightly weathered siltstone and sandstone, and must not degrade under placement. The material will exclude material that is primarily clay or silt, wet, or contains high organic content. The maximum particle size is 400mm, with minimum fines content.

The material will be placed in 600mm (loose) layers and compacted to achieve a uniform, compacted mass. The level of compaction will be determined on site from field trials to determine the compactive effort required.

2.3 Operating Procedures

Prior to commencement of tailings deposition, an Operation, Maintenance and Surveillance (OMS) Manual will need to be prepared for the Brunswick West TSF, in accordance with ANCOLD Guidelines **[4]**.

As a minimum, the OMS should include the hydraulic performance criteria, and instructions to cover all necessary monitoring, daily and weekly routine inspections and surveillance activities. Tailings deposition, decant and return water management procedures will also be documented.

This CFMA has been prepared on the assumption that a suitable document will be prepared and adhered to, which has been accounted for in the assigned probabilities or failure mode controls relating to operations. As such, this document should be reviewed upon finalisation of the OMS Manual to ensure that the information presented herein is appropriate.

An overview of what will be incorporated into the OMS Manual is provided in the design report **[1]**. For the purpose of this document, this information has been assumed to be implemented at the time of commissioning the TSF, and is summarised below:

- Tailings Delivery and Return Water:
  - Tailings deposition will primarily occur from a single spigot at the northern end of the facility, with 4-6 additional spigots along the eastern and western embankments to aid in beach development.
  - Decant water will be collected via an inclined decant structure, comprising of thick-walled HDPE pipelines with regularly spaced slots cored in, and wrapped in a UV stabilised geotextile to filter tailings. Decant water will collect in a buried reinforced concrete sump within the excavation, and be returned via a submersible pump activated by automated water level sensors.
  - Tailings delivery and return water pipelines will be aligned such that they do not cross the emergency spillway.
  - In high risk areas, to minimise the risk of material flowing off site in the event of pipe burst, tailings delivery and return water pipelines will be sleeved in additional HDPE pipeline.

- Surveillance and Monitoring:
  - Routine monitoring will include reconciliation of tailings discharge tonnages and solids concentrations, tailings beach head and toe levels, decant structure and pump performance, water levels and return water rates, groundwater monitoring, tailings beach density and shear strength profiles.
  - Routine surveillance will consist of daily, weekly, monthly, intermediate (annual) and comprehensive (after first year and then 2 yearly) inspections. The level of detail of each type of inspection will increase with reducing frequency. Routine surveillance will generally comprise of visual inspections of the following;
    - Tailings and return water pipelines.
    - Deposition spigots.
    - Tailings Beach (level, overall shape).
    - Decant Pond (size, location, depth, clarity).
    - Decant structure.
    - Embankment crest and batter conditions.
    - BGM liner condition.

A Dam Safety Emergency Plan (DSEP) will also be developed for the storage to document the procedures to be followed based on trigger levels for the following scenarios;

- Major mechanical or electrical failure.
- Extreme rainfall.
- Seismic activity.
- Significant embankment movement/slippage.

These trigger levels are defined on a traffic-light system, from lowest (within normal operating levels) to highest (failure is imminent), and provides instruction on the required actions to take, with directives to evacuate the Brunswick Processing Plant area and Brunswick Underground if approaching the highest level. This DSEP will be regularly reviewed, and evacuation procedures tested to ensure the mine personnel within the inundation area are trained on the actions to take.

2.4 Instrumentation and Monitoring

Instrumentation for the Brunswick West TSF will consist of Groundwater Monitoring Bores that are to be installed at five locations around the toe of the embankment, coinciding with low points in the natural topography. These will provide measurements to aid the assessment of the impact (if any) of the TSF to the groundwater.

Embankment settlement and movement, and tailings beach development will be monitored via regular aerial surveys.

3 CONSEQUENCE CATEGORY

3.1 Overview

The consequence category assigned to a dam is used as a measure for rating the potential impacts resulting from dam failure.

An assessment of the consequence category for the Brunswick West TSF has been undertaken in accordance with the ANCOLD Guidelines for Consequence Categories **[3],** the ANCOLD Tailings Guidelines **[4],** and Victorian Government Tailings Guidelines **[5]** (which are in turn derived from ANCOLD). As outlined in these guidelines, two consequence categories are assessed for design purposes:

- The Dam Failure Consequence Category. This is an assessment of the potential failure modes of the TSF and the consequences to public safety, the environment and public infrastructure as the result of a dam failure.
- The Environmental Spill Consequence Category. This is determined by considering only the effects of a spill from the TSF during a flood or extended extreme wet weather period without the TSF itself failing.

The consequence category of a TSF is determined as a matrix of the severity of damage and loss occurring because of a dam failure/environmental spill, and the Population at Risk (PAR) downstream of the facility, or Potential Loss of Life (PLL).

The consequence category of the Brunswick West TSF was determined based on the results of the dam break assessment for the facility in context of the proposed flood protection measures, and is documented in ATCW report 10914.15-R02 **[6]**. The relevant components of the consequence category assessment are summarised in the following sections.

3.2 Severity Level Impact Assessment

An assessment of the severity of damage and loss for a dam failure scenario has been undertaken in accordance with the ANCOLD Guidelines on Consequence Categories **[3]** and is documented in both the design report **[1]** and dam break assessment **[6]**.

A severity of damage and loss for a dam failure scenario was assessed as “Major”, owing to the importance to business operations, impact to business credibility, community and political implications, and impact on financial viability. This is consistent with the severity of damage and loss for the nearby Brunswick TSF, which is in a similar regional setting as the Brunswick West TSF, resulting in similar damages and loss.

The severity of damage and loss for an environmental spill scenario has been assessed as “Medium”, owing to the release of potentially contaminated mine water and impact to the natural environment.

3.3 Population at Risk

The PAR from a dam failure was estimated based on the results of the dam break assessment of the Brunswick West TSF **[6]**. This assessment considered both Sunny Day Failure (SDF) and Flood Failure (FF) cases at critical positions around the facility and considered both no-mitigation and mitigation cases with respect to flooding impacts. The PAR was then estimated in the context of the proposed flood protection measures to be constructed as part of the Brunswick West TSF construction. Details of this assessment are documented in the dam break report **[6]**.

The critical downstream areas that would be impacted by a failure of the Brunswick West TSF will be the Brunswick Processing Plant, Heathcote-Nagambie Road, and potentially two residential houses further downstream.

The critical dam failure time was found to be a weekday daytime failure, owing to the number of mine personnel within the Brunswick Processing Plant area during this time period. The critical breach location was identified as occurring at the eastern embankment of the Brunswick West TSF.

The peak PAR values are summarised below:

Sunny Day Failure

- Brunswick Processing Plant 23
- Heathcote-Nagambie Road 1.7
- Downstream residential properties 0
- **Dam Failure PAR** **24.7 (rounded up to 25)**

Flood Failure

- Brunswick Processing Plant 0 (Plant to be shut down in prevailing conditions that would lead to failure)
- Heathcote-Nagambie Road 1.7
- Downstream residential properties 0
- **Dam Failure PAR** **1.7 (rounded up to 2)**

Environmental Spill

- Not likely to produce a PAR, as flow is controlled via the spillway. Assessed as <1

From the above assessment, the critical Dam Failure PAR was determined as 25, with the Environmental Spill PAR as <1

3.4 Estimation of Potential Loss of Life

An estimation of the PLL was undertaken for the facility in context of the critical failure time. The PLL assessment was undertaken using the methods described by Graham **[7]**, as recommended by ANCOLD **[3]**. These methods were developed based on case studies of historical dam failures, as the author notes there is no available procedure for predicting the exact number of fatalities that may arise from a dam failure.

The loss of life resulting from a dam failure is highly influences by three factors:

1. The number of people within the downstream inundation area,
2. The total amount of warning time available for evacuation, and
3. The severity of the flooding.

The PLL is then determined by estimating a fatality rate, and applying this to the PAR at individual areas within the downstream inundation area. Details of this assessment are documented in the dam break report **[6]**.

The assessment found the critical fatality rate of 0.04 at the Brunswick Processing Plant and 0.007 at Heathcote-Nagambie Road for both the sunny day and flood failure scenario. Applying these fatality rates to the assessed PAR (refer **Section 3.3**)

Sunny Day Failure

- Brunswick Processing Plant 0.92
- Heathcote-Nagambie Road 0.01
- **Dam Failure PLL** **0.93 (rounded up to 1)**

Flood Failure

- Brunswick Processing Plant 0
- Heathcote-Nagambie Road 0.01
- **Dam Failure PLL** **0.01**

3.5 Assessed Consequence Category

Based on the severity of loss and damage for each failure scenario (refer **Section 3.2**) and the PAR (refer **Section 3.3**), the assessed consequence categories for the Brunswick West TSF are as follows;

- Dam Failure **High B**
- Environmental Spill **Low**

ATCW notes that the consequence category assessed based on the PLL results in the same outcome as above, in accordance with ANCOLD **[8]**.

3.6 Design Criteria

Based on the assigned Consequence Category outlined in **Section 3.5**, the ANCOLD Guidelines **[4]** required minimum design criteria is summarised in **Table 2.** Notes on the ATCW adopted design criteria, and additional design criteria that is not required by ANCOLD are discussed in the TSF design report **[1]**.

TABLE 2: ANCOLD MINIMUM REQUIRED DESIGN CRITERIA

Criteria Description	Criteria Requirement		
Dam Failure Consequence Category	**HIGH B**		
Embankment stability minimum Factor of Safety (FoS)	**Loading Condition**	**Criteria**	**Material Strength Parameters**
	Static Short Term	FoS ≥ 1.3	Undrained Strength
	Static Long Term	FoS ≥ 1.5	Drained (effective) or Undrained strength (as appropriate for cohesive materials)
	Post Liquefaction (Safety Evaluation Earthquake)	FoS ≥ 1 – 1.2	Post-liquefied strength /residual strength for material that are potentially liquefiable. 20% reduction in peak shear strength for the materials that are not liquefiable.
Seismic Criteria	Operational Basis Earthquake (OBE) Safety Evaluation Earthquake (SEE)		1:475 to 1:1,000 AEP event 1:5,000 AEP event **(adopted 1:10,000 AEP)**
Minimum Spillway Critical Design Storm	1:100,000 AEP Critical Duration Storm + Wave run-up for 1:10 AEP wind, or Probable Maximum Flood (PMF)		
Environmental Spill Consequence Category	**LOW, but have adopted SIGNIFICANT**		
Maximum Operating Pond	Determined by semi-quantitative risk analysis methods, or 1:100 Notional AEP wet season runoff (fall back method)		
Minimum Extreme Storm Storage	1:100 AEP, 72 hr flood		
Contingency Wave Freeboard Allowance	1:10 AEP Wind		
Additional Freeboard	0.3 m		

4 SCOPE OF WORK

The ANCOLD Guidelines on Risk Assessment **[8]** recommend that for High Consequence Category dams, a quantitative risk assessment be conducted to quantify and evaluate the risk tolerability of the facility.

The CFMA involves a review and assessment of potential failure modes for the TSF embankments. For the purposes of this study, a critical failure was considered to be one that could result in loss of impoundment material and / or initiate a catastrophic failure of the embankment that would directly place mine personnel or persons downstream at risk.

The process for this assessment is summarised below:

1. Identify all potential failure modes for the facility that would result in a release of material, based on ATCW and MRCO experience and judgement, and drawing on databases of failure modes that have occurred at other facilities globally;
2. Document the failure progression (sequential steps required for a failure to occur), and the controls in place to mitigate a potential failure;
3. Undertake a qualitative screening of all the failure modes by categorising them as either "Potentially Credible" or "Non-Credible";
4. Conduct a quantitative analysis of the potentially credible failure modes to assign an overall probability of failure, or to discredit the failure mode entirely; and

5. Assess the overall level of risk for the facility, considering the probability of failure and the PLL.

5 POTENTIAL FAILURE MODES IDENTIFICATION AND SCREEING

5.1 Potential Failure Mode Identification

A list of Potential Failure Modes (PFMs) that could result in a breach of the embankments was compiled to initially assess the likelihood of a failure occurring. These failure cases can broadly be categorised as a failure due to:

- Geotechnical Piping,
- Overtopping of the embankment crest, or
- Embankment Instability

Additionally, as part of the dam break modelling **[6]**, a combined settlement and erosion failure was identified as potentially credible to provide verification of a potential sunny day embankment failure.

This list has been compiled on a collaborative basis between ATCW and MRCO based on engineering judgment, experience with the site and with similar TSFs.

The PFMs identified for the Brunswick West TSF are summarised in **Table 3.** A detailed breakdown of the failure modes and the failure progression for each potential failure mode is presented in **Appendix A**.

TABLE 3: POTENTIAL FAILURE MODES SUMMARY

Ref. No	Failure Mode
Geotechnical Piping	
P1	Geotechnical Piping through embankment - Cracking caused by differential settlement from steep underlying topography
P2	Geotechnical Piping through embankment - Cracking caused by differential settlement of foundations
P3	Geotechnical Piping through embankment - Cracking caused by loss of support from downstream shoulder
P4	Geotechnical Piping through embankment - Cracking caused by loose/poorly compacted layers in upstream clay zone
P5	Geotechnical Piping through embankment - Desiccation cracking through upstream clay zone
P6	Geotechnical Piping through embankment - Animal burrows and vegetation causing seepage path through embankment
P7	Geotechnical Piping caused by transverse seismic cracking
P8	Geotechnical Piping through foundations
P9	Geotechnical Piping into foundations
Embankment Overtopping	
O1	Embankment Overtopping due to loss of spillway capacity – Dam Break Flood Failure Case
O2	Embankment Overtopping due to crest scour from concentrated rainfall runoff
O3	Embankment Overtopping due to crest scour from pipeline burst
O4	Embankment Overtopping due to poor deposition management - Spillway Blockage
O5	Embankment Overtopping due to poor deposition management - Decant Blockage

Ref. No	Failure Mode
O6	Embankment Overtopping due to poor deposition management - Over deposition
O7	Embankment Overtopping due to build-up of excess tailings bleed water
O8	Embankment Overtopping due to higher than expected operating pond levels
O9	Embankment Overtopping due to single/multiple large storms that exhaust freeboard and exceeds spillway capacity
O10	Embankment Overtopping due to loss of spillway capacity from seismic induced crest settlement
O11	Embankment Overtopping due to reduced spillway capacity from seismic induced crest settlement
O12	Embankment Overtopping due to scour from failure of Spillway Erosion Protection Rip-Rap
Embankment Instability	
S1	Embankment Instability due to incorrect material characterisation - Embankment fill materials
S2	Embankment Instability due to incorrect material characterisation - Foundation materials
S3	Embankment Instability due to high phreatic surface
S4	Embankment Instability due to inadequately constructed embankments
S5	Embankment Instability due to inadequately prepared foundations
S6	Embankment Instability due to seismic instability
Combined Settlement and Erosion Failure	
C1	Embankment erosion failure due to cumulative static settlement and seismic deformation – <u>Dam Break Sunny Day Failure Case</u>

5.2 Identified Failure Controls

Initial qualitative screening of the PFMs was undertaken in consideration of all controls that can be implemented to reduce the likelihood of failure occurring. For each of the potential failure modes, the relevant controls that have been/can be implemented during different stages of the TSF lifecycle have been considered;

- <u>Design</u> – Elements incorporated at the design stage of the TSF that can reduce the likelihood or eliminate key stages during failure progression.
- <u>Construction</u> – Actions taken during construction to ensure the design intent is met, and that adverse conditions/scenarios that may increase the likelihood of a failure progression do not occur.
- <u>Operation</u> – Actions taken during operation of the facility to ensure the design intent is met, failure initiation conditions are removed, and that the facility remains fully functional.

A list of the relevant controls are summarised in **Table 4**.

TABLE 4: BRUNSWICK WEST TSF EMBANKMENT FAILURE CONTROLS

Control	Failure Aspect Mitigated
Design	
TSF designed with Bituminous Geomembrane (BGM) Liner on upstream face.	Prevent embankment seepage. Maintain phreatic surface low through the embankment. Prevent animal burrows into embankment upstream face.

Control	Failure Aspect Mitigated
Loose foundation material (topsoil and upper clays) removed, and remaining foundation clays compacted to 98% SMDD.	Prevention of embankment differential settlement.
Zone 1B Clayey Subgrade completely covered (upstream face with BGM liner, crest with road base material).	Prevention of desiccation cracking.
Spillway designed to safely pass the Probable Maximum Flood with pond starting at spillway invert level.	Prevention of flood waters overtopping embankments.
Additional freeboard provided from spillway maximum depth to embankment crest.	Prevention of flood waters overtopping embankments.
Tailings and return water pipelines aligned to not cross emergency spillway.	Maintaining spillway performance.
Tailings pipeline aligned on embankment upstream crest.	Prevention of downstream crest scour.
Additional road base material placed on embankment crest, and shaped to provide uniform 1-way crossfall into the TSF.	Prevention of desiccation cracking. Aid in prevention on crest scour.
TSF designed with maximum storage level at the spillway invert which shall not be exceeded.	Prevention of flood waters overtopping embankments.
TSF designed with limited number of deposition points at opposite end of decant structure.	Prevention of spillway blockage
Stochastic water balance of 1,000 scenarios of climate data to allow for detailed modelling of predicted maximum pond levels.	Facility designed for maximum water levels from storm water.
Water balance undertaken at conservative lower bound tailings properties that would produce the most amount of bleed water.	Facility designed for maximum water levels from tailings bleed.
Site-specific Seismic Hazard Assessment undertaken to understand maximum seismic levels at site.	Increased confidence in design seismic levels.
Embankment constructed by downstream methods, and primarily of compacted rockfill.	Minimisation of static settlement. Prevention of seismic deformation.
Erosion protection rip-rap designed to meet maximum expected velocities within spillway.	Prevention of downstream crest erosion when spillway flows.
Geotextile below erosion protection rip-rap.	Prevention of downstream crest erosion when spillway flows.
Lower bound strength parameters adopted for embankment and foundation materials.	Reduce likelihood of embankment stability failure.
Embankment geometry provides sufficient factor of safety against failure.	Reduce likelihood of embankment stability failure.
Embankment stability assessed at conservative maximum phreatic surface level.	Reduce likelihood of embankment stability failure.
Construction	
Dedicated liner installation sub-contractor, including testing and QA/QC program.	Reduce likelihood of unidentified construction flaws in BGM
Full-time construction QA/QC provided to ensure design specifications are met (foundation preparation, material specifications, material placement and compaction etc).	Ensure design intent is met for all elements of the TSF system.

Control	Failure Aspect Mitigated
Ensure construction debris is cleared from spillway following completion of construction.	Prevention of spillway blockage
Ensure construction debris is cleared from decant pipes, and that they are operational following completion of construction	Prevention of blockage of decant structure, and ensure water removal.
Operation	
Automated pump for removal of decant water.	Maintain water levels low in facility
Routine inspections to be carried out on the TSF critical elements, to be documented in the Operation, Maintenance and Surveillance Manual (OMS), including • Geosynthetic Liner • Signs of downstream seepage • Crest erosion/scour • Cracking & embankment settlement • Blockages across spillway • Pipeline deficiencies & degradation • Pump performance • Tailings beach development • Spillway and erosion rip-rap	Ensure potentially detrimental conditions on the embankments are identified and remedied
Vegetation to be regularly removed from the embankment crest and upstream face.	Prevention potential holes in the BGM liner. Dissuade animals from attempting to burrow into embankments
Dedicated inspections of the facility following significant seismic events.	Inspect for possible detrimental conditions from seismic activity, and identify what needs to be remedied.
Automated monitoring of pipeline flow pressures to provide for detection of burst pipelines.	Identification of whether a pipeline burst has occurred.
Provision of additional stand-by pump to allow for emergency removal of water from TSF.	Removal of surface water to prevent embankment overtopping.
Routine monitoring of tailings slurry composition to ensure design parameters are met.	Aid in preventing build-up of excess decant water.
Development and regular testing of a Dam Safety Emergency Plan (DSEP) for the facility, including evacuation of the Brunswick Processing Plant area if failure conditions are considered imminent.	Further reduce PLL in the event of a Dam Break

5.3 Qualitative Screening

With the relevant controls considered (refer **Section 5.2**), all of the potential failure modes have been qualitatively screened and assessed as one of the following

- Potentially Credible – A failure mode that is technically feasible or can be envisaged as possible even with all the controls in place, and may result in a dam failure of the TSF.
- Non-Credible – A failure mode that is considered as incredibly unlikely to occur with all the controls in place, or a scenario that is illogical in context of the facility. For example, this may be “failure through the upstream raises” when the embankment is constructed entirely by downstream methods.

The initial qualitative screening of the potential failure modes is summarised in **Table 5**.

TABLE 5: POTENTIAL FAILURE MODE INITIAL QUALITATIVE SCREENING SUMMARY

Ref. No	Failure Mode	Failure Mode Credibility	Justification
		Geotechnical Piping	
P1	Geotechnical Piping through embankment - Cracking caused by differential settlement from steep underlying topography	Not Credible	No abrupt changes in underlying topography within embankment footprint.
P2	Geotechnical Piping through embankment - Cracking caused by differential settlement of foundations	Not Credible	No deep, highly compressible soils to remain within embankment foundations
P3	Geotechnical Piping through embankment - Cracking caused by loss of support from downstream shoulder	Potentially Credible	-
P4	Geotechnical Piping through embankment - Cracking caused by loose/poorly compacted layers in upstream clay zone	Potentially Credible	-
P5	Geotechnical Piping through embankment - Desiccation cracking through upstream clay zone	Not Credible	Clayey Zone 1B subgrade designed to be completely covered (upstream face with BGM liner, crest with road base material) to prevent formation of desiccation cracking.
P6	Geotechnical Piping through embankment - Animal burrows and vegetation causing seepage path through embankment	Not Credible	Routine visual inspections (weekly) of the liner will identify any deficiencies and will be promptly repaired
P7	Geotechnical Piping caused by transverse seismic cracking	Potentially Credible	-
P8	Geotechnical Piping through foundations	Not Credible	Tailings will form a low permeability seal against the upstream face and foundations, limiting the depth of standing water against the foundations to prevent the initiation of piping. Only conceivable scenario for this to occur is partway through deposition with tailings just below foundation soil level and high pond levels. Failure in this case would only release a small amount of material (i.e. not a catastrophic failure).
P9	Geotechnical Piping into foundations	Potentially Credible	-

Ref. No	Failure Mode	Failure Mode Credibility	Justification
		Embankment Overtopping	
O1	Embankment Overtopping due to loss of spillway capacity.	Potentially Credible	-
O2	Embankment Overtopping due to crest scour from concentrated rainfall runoff	Not Credible	Significant concentrated rainfall scour is highly unlikely given uniform crest shape and drainage. Potential scour will be noticed early and repaired
O3	Embankment Overtopping due to crest scour from pipeline burst	Potentially Credible	-
O4	Embankment Overtopping due to poor deposition management - Spillway Blockage	Potentially Credible	-
O5	Embankment Overtopping due to poor deposition management - Decant Blockage	Potentially Credible	-
O6	Embankment Overtopping due to poor deposition management - Over deposition	Potentially Credible	-
O7	Embankment Overtopping due to build-up of excess tailings bleed water	Potentially Credible	-
O8	Embankment Overtopping due to higher than expected operating pond levels	Potentially Credible	-
O9	Embankment Overtopping due to single/multiple large storms that exhaust freeboard and exceeds spillway capacity	Potentially Credible	-
O10	Embankment Overtopping due to loss of spillway capacity from seismic induced crest settlement.	Not Credible	This failure assumed seismic deformation in excess of total spillway depth. Deformation for 1:10,000 AEP seismic event indicated maximum of 1.5% of total height to bedrock as deformation. At the maximum embankment height, this is less than 225 mm, which is less than the depth of the spillway.
O11	Embankment Overtopping due to reduced spillway capacity from seismic induced crest settlement	Potentially Credible	- *Note; this differs from Case O10 in that embankment deformation occurs, with flood waters partially flowing through spillway and crest low spot, causing failure.*

Ref. No	Failure Mode	Failure Mode Credibility	Justification
O12	Embankment Overtopping due to scour from failure of Spillway Erosion Protection Rip-Rap	Potentially Credible	-
Embankment Instability			
S1	Embankment Instability due to incorrect material characterisation - Embankment fill materials	Potentially Credible	-
S2	Embankment Instability due to incorrect material characterisation - Foundation materials	Potentially Credible	-
S3	Embankment Instability due to high phreatic surface	Potentially Credible	-
S4	Embankment Instability due to inadequately constructed embankments	Potentially Credible	-
S5	Embankment Instability due to inadequately prepared foundations	Potentially Credible	-
S6	Embankment Instability due to seismic instability	Not Credible	Embankment or foundation materials not identified as susceptible to liquefaction, and seismic instability improbable. Deformation for 1:10,000 AEP seismic event indicated maximum of 1.5% of total height to bedrock as deformation. At the maximum embankment height, this is less than 225 mm, which is less than the beach freeboard.
Combined Settlement and Erosion Failure			
C1	Embankment erosion failure due to cumulative static settlement and seismic deformation	Potentially Credible	-

6 QUANTITATIVE ANALYSIS

6.1 Overview

6.1.1 Assessment Conditions

The failure modes that were identified as "Potentially Credible" in **Section 5.3** have been numerically analysed to assess the likelihood of each failure mode occurring.

Failure has been analysed at the critical stage of the Brunswick West TSF, corresponding to the end of filing scenario **[1]**. Preceding events that may occur to result in embankment failure have been considered from these starting conditions.

These conditions are presented in **Figure 5**, and are summarised below;

- Embankment crest level of RL 200.0 m, with a 3% crest drainage into the TSF.
- Tailings at a maximum head of beach of RL 199.5 m.
- Surface water equal to the maximum operating pond of 14,000 m^3, or a depth of approximately 0.9 m at the decant structure, at RL 198.9 m.
- Emergency Spillway with an invert level of RL 199.5 m, and maximum flow capacity of 6.0 m^3/sec.
- Total catchment area defined by the embankment downstream crest edge, equal to 6.3 ha.

The above conditions represent the starting point for the assessment of overall failure, with additional factors required to result in failure of the embankment applied on a cumulative basis over these conditions.

6.1.2 Numerical Assessment of Failure

Estimation of the overall likelihood of failure generally requires one or more detrimental conditions present at the Brunswick West TSF, followed by an extreme loading scenario to cause failure. These factors are required to occur cumulatively, and in probabilistic terms, this represents the intersection of all contributing faults, as described in **Equation 1**.

EQUATION 1: ESTIMATION OF THE OVERALL PROBABILITY OF FAILURE

$$Pr_T = Pr_1 \cap Pr_2 \ldots \cap Pr_N = Pr_1 * Pr_2 * \ldots * Pr_N$$

Where Pr_1, Pr_2 etc. are contributory faults to P_T.

Estimation of probability of some events, particularly when considering two different loadings conditions for the same event, have at least one or more contributing faults. In probabilistic terms, this represents the union of the contributing faults, however ANCOLD **[8]** note there is no practical method of computing overall probability for multiple faults, and recommend an upper bound estimation of the probability of multiple faults (also known as De Morgan's Rule) as described in **Equation 2**.

EQUATION 2: UPPER BOUND ESTIMATION OF PROBABILITY OF MULTPLE FAULTS (DE MORGANS RULE)

$$Pr_T = Pr_1 \cup Pr_2 \ldots \cup Pr_N \approx 1 - (1 - Pr_1) * (1 - Pr_2) * \ldots * (1 - Pr_N)$$

Where Pr_1, Pr_2 etc. are contributory faults to P_T.

The nature of the design, construction and operation of a TSF can result in contributing factors caused by human error, which often subjective in nature have no inherent associated probability. For example, this may be "likelihood that pipelines are incorrectly placed across the spillway, and are not removed when a storm event is imminent".

In scenarios like this, ANCOLD **[8]** recommend the use of a mapping scheme where the estimated probabilities are related to verbal descriptors of likelihood, with specific reference to the mapping scheme described by Barneich et al. (1996) **[9]**. This mapping scheme provides an order of magnitude probability estimate of an event occurring. This mapping scheme has been used where required in the quantitative assessment and is reproduced in **Table 6**.

TABLE 6: SUBJECTIVE PROBABILITY ESTIMATION GUIDE AFTER BARNEICH ET AL. (1996) (REPRODUCED FROM ANCOLD)

Description of Condition or Event	Order of Magnitude of Probability	Likelihood
Occurrence is virtually certain.	1	Almost Certain
Occurrence of the condition or event are observed in the database.	10^{-1}	Likely
The occurrence of the condition or event is not observed, or is observed in one isolated instance, in the available database; several potential failure scenarios can be identified.	10^{-2}	Unlikely
The occurrence of the condition or event is not observed in the available database. It is difficult to think about any plausible failure scenario; however, a single scenario could be identified after considerable effort.	10^{-3}	Very Unlikely
The condition or event has not been observed, and no plausible scenario could be identified, even after considerable effort.	10^{-4}	Almost Impossible

6.1.3 Presentation of Failure Probabilities

The failure probabilities within the following sections have been estimated as conditional probabilities, assessed at the critical stage of the Brunswick TSF life cycle, corresponding to the end of filling of the TSF (refer **Section 6.1.1**), and assessed for the critical embankment section. The ANCOLD Guidelines on Risk Assessment **[8]** generally discuss the level of risk in terms of annual probability.

This assessment has been conducted for the long term operating conditions, up to the end of filling of the Brunswick West TSF. For this assessment, it has been assumed that annualised probability is approximately equal to the conditional probabilities estimated.

6.2 Geotechnical Piping

6.2.1 Credible Failure Cases

The failure cases related to geotechnical piping that were identified as potentially credible as part of the initial screening are outlined below:

Case P3 Geotechnical Piping through embankment - Cracking caused by loss of support from downstream shoulder.

Case P4 Geotechnical Piping through embankment - Cracking caused by loose/poorly compacted layers in core.

Case P7 Geotechnical Piping caused by transverse seismic cracking.

Case P9 Geotechnical Piping into foundations.

6.2.2 Method of Assessment

Failure of an embankment due to geotechnical piping requires a series of events to occur sequentially and remain in place until a failure mode can develop and cause a failure.

The geotechnical piping process can be summarised as follows:

1. Initiation – A concentrated leak in the embankment allows for water to begin to mobilise through the upstream zone (compacted Zone 1B) and initiate the internal erosion process.
2. Continuation – Relates to whether filters or downstream transition zones within the dam will prevent internal erosion from continuing further. In the context of the Brunswick West TSF, this is an assessment of the performance of the compacted Zone 3A and 3B materials.
3. Progression – An assessment as to whether;
    a. the soil within which the pipe is forming will support the roof of the pipe,
    b. if the pipe will collapse and seal the pipe,
    c. the upstream zones will limit flow to reach an equilibrium condition, or
    d. soil from the upstream zones will wash into the eroding soil and stop the process.
4. Detection & Intervention – Can signs of internal erosion be detected, and will intervention and repair be possible in the time available.
5. Breach – The initial piping failure cascading into a catastrophic failure that will lead to a release of the stored material.

Fell et al. (2015) **[10]** developed methods for estimating the probability of failure by internal erosion and piping, which they refer to as the "piping toolbox". This method was developed as a unified approach for carrying out risk analyses by the Unites States Bureau of Reclamation (USBR), United States Army Corps of Engineers (USACE), Consulting Engineers URS, and the University of New South Wales (UNSW). The piping toolbox was developed based on the analysis of historical embankment failures and expert judgement behind the importance and impact of the relevant factors. As such, estimation of the probability of piping failure has been undertaken following this procedure.

In the estimation of these probabilities, the relevant properties of the embankments have been based on the results of the geotechnical investigations and laboratory testing **[2]**, the adopted design parameters and the expected construction material properties **[1]**. Where there was uncertainty regarding a parameter in the probability calculation, a conservative estimate was made which presented the likely upper bound on the probability.

The methods for estimation of piping failure described by Fell et al. (2015) **[10]** were primarily developed for water storage dams, where a free water body is continually present at the upstream face of the embankment, and can contribute to a high seepage gradient if erosion begins. These methods remain applicable for tailings retaining embankments once the erosion process has begun, but consideration must be given to the effect of the tailings in the initiation process.

Fell et al. (2015) **[10]** indicates that initiation from a concentrated leak (such as through a crack or high permeability embankment material) is generally only expected above the level of the tailings, as the deposited tailings will generally form a low permeability seal against the embankment, resulting in a low seepage gradient and providing upstream flow limitation. The seepage gradient for a piping erosion failure is therefore driven by the operating pond levels, plus the most recently deposited tailings that are continuing to settle (upper half metre or so).

Initiation has been considered for two scenarios, as described by Fell et al. (2015) **[10]**;

- Below the "Pool of Record", described as the highest historical level plus up to 0.3 m. For this assessment, this is taken as the maximum operating pond level (198.9 m) plus 0.3 m, for a water level of RL 199.2 m.
- Above the "Pool of Record", for filling beyond the highest historical level plus 0.3 m. For this assessment, this is taken as the water level at spillway invert level (RL 199.5 m). Raising the water to spillway level requires a significant storm event, which has been estimated as the 1:1,000 AEP 72 hr event.

The probability within the piping toolbox is estimated for both water levels, and the overall probability of initiation at the either of these water levels is calculated using De Morgan's rule, as presented in **Equation 2**.

The potentially credible geotechnical piping failure cases, along with a summary of likely failure progressions, are presented in **Table 7.**

TABLE 7: POTENTIALLY CREDIBLE PIPING FAILURE CASES

Ref. No	Failure Mode	Failure Progression
P3	Geotechnical Piping through embankment - Cracking caused by loss of support from downstream shoulder.	• Significant flaw in BGM liner, which is un-noticed and not repaired. Leads to high seepage gradient against Zone 1B. • Increased seepage saturates Zone 3B, resulting in collapse and significant settlement of the downstream rockfill shoulder, leading to transverse cracking across Zone 1B from embankment crest to maximum operating pond. • Piping progresses through the embankment materials, emerging on the downstream face, resulting in an embankment failure.
P4	Geotechnical Piping through embankment - Cracking caused by loose/poorly compacted layers in upstream clay zone	• Significant flaw in BGM liner, which is un-noticed and not repaired. Leads to high seepage gradient against Zone 1B. • Continuous, poorly compacted layers across entire width of Zone 1B, which are not identified or remedied. • Piping progresses through the embankment materials, emerging on the downstream face, resulting in an embankment failure.
P7	Geotechnical Piping caused by transverse seismic cracking	• Significantly large seismic event occurs, damaging the embankments and causing a transverse cracking across Zone 1B from embankment crest to maximum operating pond. • Seismic event is also assumed to cause a tear in the BGM liner. • Piping progresses through the embankment materials, emerging on the downstream face, resulting in an embankment failure.
P9	Geotechnical Piping into foundations	• Significant flaw in BGM liner, which is un-noticed and not repaired. Leads to high seepage gradient against Zone 1B. • Collapse and significant settlement of downstream Zone 3B, leading to transverse cracking across Zone 1B from embankment crest to maximum operating pond. • Piping progresses into the clayey foundations, emerging at the downstream toe, resulting in an embankment failure.

6.2.3 Results

The results of the geotechnical piping assessment at each step are presented in **Table 8**, with a detailed assessment of each step of the internal erosion process documented in **Appendix B**.

The overall assessed probabilities of the failure cases are presented in **Table 9**.

TABLE 8: GEOTECHNICAL PIPING PROGRESSION STEP SUMMARY

Piping Stage	Failure Progression step	Assessed Probability
Preceding Events	Significant tear in BGM liner that goes un-noticed and un-repaired	1.0×10^{-2} (1)
Initiation	Initiation in Cracking by Differential Settlement as a result of cross section settlement from poorly compacted downstream shoulder	3.0×10^{-4}
	Initiation in Cracking from Seismic Events	4.0×10^{-7}
	Initiation from poorly compacted/highly permeable regions in the upstream zones	1.0×10^{-4}
Continuation	Continuation of erosion, given initiation by cracking	8.0×10^{-1}
	Continuation of erosion, given initiation from high permeability zone	5.0×10^{-2}

Piping Stage	Failure Progression step	Assessed Probability
Progression	Progression through the embankment, emerging at toe	5.0×10^{-2}
	Progression through the embankment and into the foundations	5.0×10^{-2}
Detection and Intervention	Leak is not detected in time, and repair efforts are unsuccessful	3.9×10^{-1}
Breach	Embankment breaches by instability, unravelling of the downstream toe, or sinkhole development	8.0×10^{-3}

Note (1) Adopted from subjective probability estimation guide, **Table 6.**

TABLE 9: GEOTECHNICAL PIPING FAILURE CASES ASSESSED PROBABILITY

Ref. No	Failure Mode	Failure Progression		Overall Assessed Probability
P3	Geotechnical Piping through embankment - Cracking caused by loss of support from downstream shoulder.	Tear in liner.	1.0×10^{-2}	3.8×10^{-10}
		Cracking from differential settlement.	3.0×10^{-4}	
		Continues through embankment (cracking)	8.0×10^{-1}	
		Progress through embankment and emerges at toe.	5.0×10^{-2}	
		Detection and Intervention fail	3.9×10^{-1}	
		Embankment Breaches	8.0×10^{-3}	
P4	Geotechnical Piping through embankment - Cracking caused by loose/poorly compacted layers in upstream clay zone	Tear in liner.	1.0×10^{-2}	7.9×10^{-12}
		Erosion into high permeability zones	1.0×10^{-4}	
		Continues through embankment (high perm)	5.0×10^{-2}	
		Progress through embankment and emerges at toe.	5.0×10^{-2}	
		Detection and Intervention fail	3.9×10^{-1}	
		Embankment Breaches	8.0×10^{-3}	
P7	Geotechnical Piping caused by transverse seismic cracking	Cracking from seismic event, and liner tear.	4.0×10^{-7}	5.0×10^{-11}
		Continues through embankment.	8.0×10^{-1}	
		Progress through embankment and emerges at toe.	5.0×10^{-2}	
		Detection and Intervention fail	3.9×10^{-1}	
		Embankment Breaches	8.0×10^{-3}	
P9	Geotechnical Piping into foundations	Tear in liner.	1.0×10^{-2}	3.8×10^{-10}
		Cracking from differential settlement.	3.0×10^{-4}	
		Continues through embankment (cracking)	8.0×10^{-1}	
		Progress into foundations	5.0×10^{-2}	
		Detection and Intervention fail	3.9×10^{-1}	
		Embankment Breaches	8.0×10^{-3}	

It can be seen that, when assessing the likelihood of a piping failure occurring through the TSF embankments, the sequential nature of a piping failure will inherently produce a very low probability of failure. The most likely failure cases were estimated with a probability of failure in the order of 10^{-10}.

The initiation of internal erosion from cracking is largely controlled by high level of compaction to be achieved within the downstream rockfill zones, limiting the potential for differential settlement and subsequent cracking. Similarly, initiation due to highly permeable layers across the width of the core is controlled by achieving a high level of compaction across the width of the core. Additionally, proper installation and incorporation of the geosynthetic liner provides another layer of protection against the initiation of internal erosion. It becomes evident that providing a high level of construction quality control and supervision at all stages of the embankment construction can provide a significant amount of control in the prevention of internal erosion initiation.

The continuation stage of internal erosion is an assessment of the downstream zones performance as filters, and the progression stage assesses whether the soils themselves will naturally stop the piping process. Both of these stages are dependent on whether the downstream zones are capable of supporting a geotechnical pipe. Zones 1B, 3A and 3B are highly likely to be able to support a geotechnical pipe due to the relatively high fines content, which is to be expected as the intention of these materials is to provide the bulk of the embankment, rather than acting as downstream filters. Providing a higher level of construction quality control to reduce the fines content of the rockfill, and remove the possibility of these zones holding a geotechnical pipe would be prohibitively costly, with little to no actual gains in embankment safety expected. Additionally, the majority of control of internal erosion is achieved by preventing it from initiating, and an acceptably low probability is estimated with the current arrangement.

An interpretation of the assessed probabilities in combination with the consequences of failure is further discussed in **Section 7.**

6.3 Overtopping

6.3.1 Credible Failure Cases

The failure cases related to overtopping that were identified as potentially credible as part of the initial screening are outlined below:

Case O1	Embankment Overtopping due to loss of spillway capacity
Case O3	Embankment Overtopping due to crest scour from pipeline burst
Case O4	Embankment Overtopping due to poor deposition management - Spillway Blockage
Case O5	Embankment Overtopping due to poor deposition management - Decant Blockage
Case O6	Embankment Overtopping due to poor deposition management - Over deposition
Case O7	Embankment Overtopping due to build-up of excess tailings bleed water
Case O8	Embankment Overtopping due to higher than expected operating pond levels
Case O9	Embankment Overtopping due to single/multiple large storms that exhaust freeboard and exceeds spillway capacity
Case O11	Embankment Overtopping due to reduced spillway capacity from seismic induced crest settlement
Case O12	Embankment Overtopping due to scour from failure of Spillway Erosion Protection Rip-Rap

6.3.2 Method of Assessment

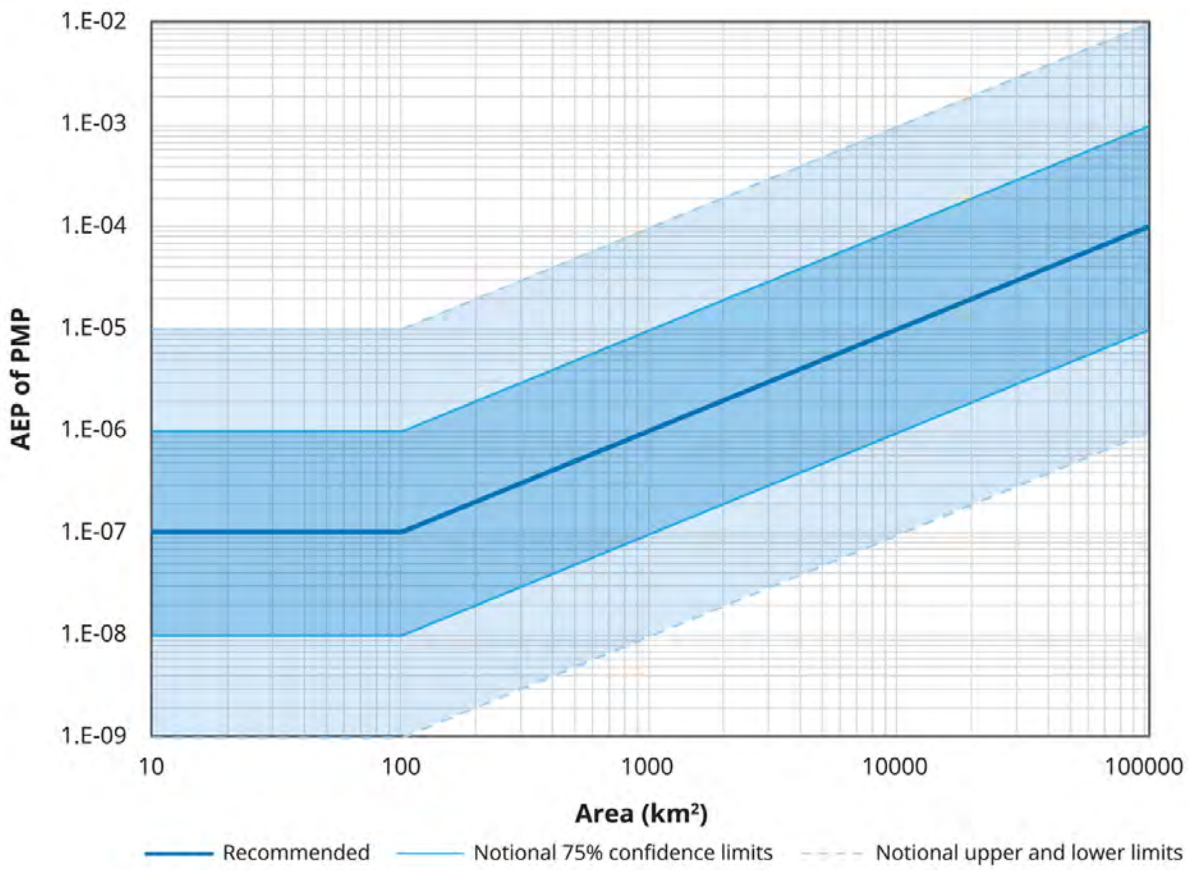
Failure of the embankment due to overtopping involves the loss of containment of surface water over the embankment crest from an extreme rainfall event. Water flows over the embankment crest and downstream slope in an uncontrolled manner, causing deep scouring and erosion, which accelerates and progresses an embankment failure. This can occur by either stormwater exceeding the spillway capacity, bypassing the spillway through a localised low point, or a combination of the two.

Estimation of probability from significant rainfall events involves correlating the required rainfall depth and duration to the Annual Exceedance Probability (AEP). The Bureau of Meteorology's (BoMs) Design Rainfall and Data System (2016) provides Intensity-Frequency-Duration (IFD) curves that define the design rainfall for sites in Australia. The data is provided as rainfall depth (or intensity) for various durations and AEP's, ranging from very common (1 in 1.6 years) to extreme events up to an AEP of 1 in 2,000 years.

Estimation of rainfall depths and AEP's beyond the data provided by the BoM required the estimation of the Probable Maximum Precipitation (PMP). The Australian Rainfall and Runoff (ARR) guidelines **[11]** describe the PMP storm event as lying beyond the credible limit of extrapolation, and border on the unknowable. As such, rainfall depths larger than the PMP can be considered as unrealistic and deemed non-credible.

ARR recommend that for catchment areas below 100 km^2, an AEP of between 1 x 10^{-6} and 1 x 10^{-8} for the PMP rainfall depths within the 75% confidence limits, as shown in **Chart 1**. ATCW have conservatively adopted 1 x 10^{-6}.

CHART 1: RECOMMENDED AEP ESTIMATES FOR PMP (FROM ARR [11])

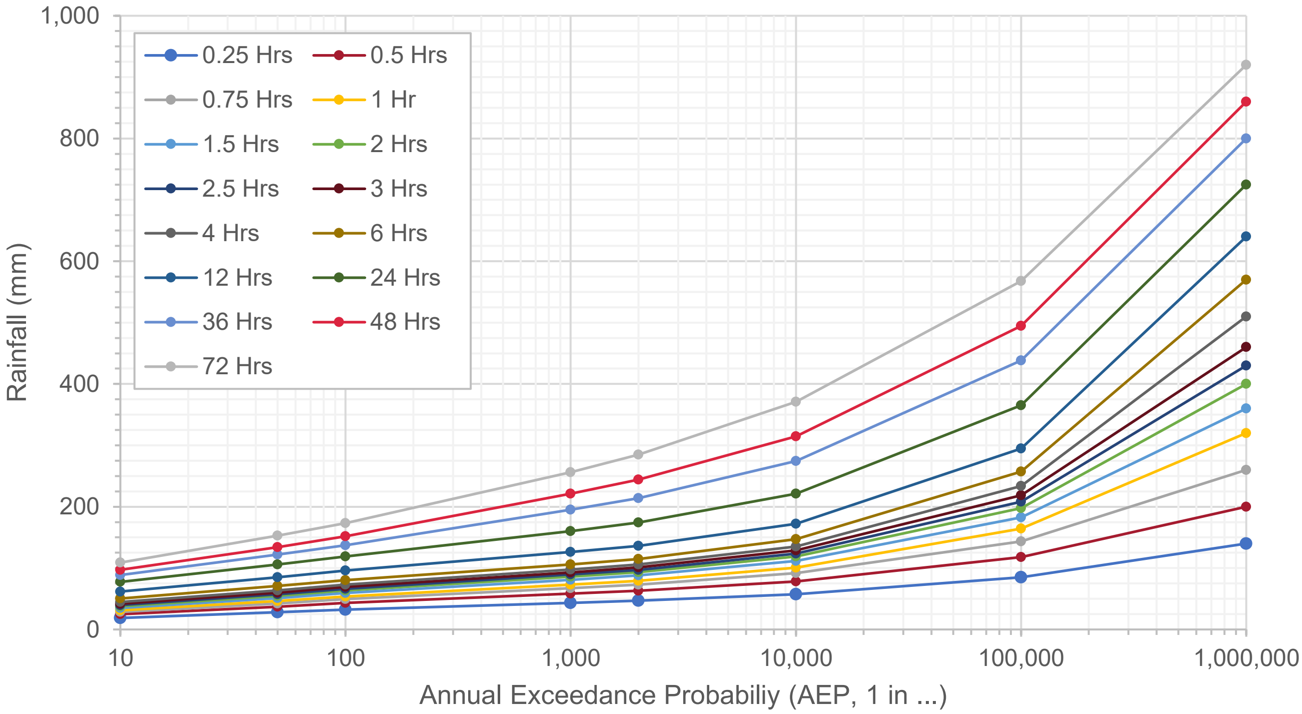


The design rainfall depths for the PMP are estimated using the Generalised Short Duration Method (GSDM) **[12]** for short durations (up to 6 hours) and Generalised South-East Australian Method (GSAM) **[13]** for long durations (up to 72 hours).

Once the PMP rainfall depths are established, estimation of other AEP events was undertaken using the methods described by Siriwardena & Weinmann **[14]** . This method recommends using the 1:1,000 and 1:2,000 AEP events obtained from the Bureau of Meteorology and the generated PMP rainfall depths as reference points for interpolation of storm events of varying AEP.

Estimations of the extreme depths used for the quantitative assessment are presented in **Chart 2**.

CHART 2: ESTIMATION OF EXTREME RAINFALL DEPTHS



The preceding events for an overtopping failure are either the raising of water within the TSF (through rainfall events, or inability to remove built up water), or lowering the level at which the water needs to reach to overtop the embankment (through reductions in the embankment crest or spillway capacity). The probability of these events have been estimated based on the required rainfall/AEP as presented in **Chart 2**, or anecdotally using the mapping scheme outlined in **Section 6.1**

Once the initial conditions for the failure case have been defined, the AEP of the storm event to cause an overtopping failure has been estimated as follows:

- In cases where the emergency spillway is active (either at partial or full capacity), the storage indication method (direct numerical procedure described in the ARR guidelines **[11]**) used for the design of the spillway **[1]** was adopted. This method determines the storm depth and critical duration that would exceed the spillway capacity and reach the embankment crest level. From this, the AEP of the storm event was determined based on the estimations of extreme rainfall depths (refer **Chart 2**).
- In cases where the emergency spillway is bypassed (i.e., completely blocked off), the storm depth of the 72 hr storm event to exceed the remaining freeboard was estimated, which then informed the AEP of the storm event.

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For the purpose of this assessment, overtopping failures have been defined as material (water and/or tailings) released in an uncontrolled manner over the embankment crest and down the embankment slope. It has been conservatively assumed that, once water is released from the embankments and is not flowing within a controlled structure (such as the emergency spillway), it will rapidly erode the embankment downstream slope and progress a cascading failure. In reality, the embankment rockfill zones will have some resistance to this erosion, and significant depths and velocities of water would be required to progress an embankment failure. However, assessment of the erodibility of the embankments is beyond the scope of this assessment, and this additional reduction factor has been conservatively omitted.

The potentially credible overtopping failure cases, along with a summary of likely failure progressions, are presented in **Table 10.**

TABLE 10: POTENTIALLY CREDIBLE OVERTOPPING FAILURE CASES

Ref. No	Failure Mode	Failure Progression
O1	Embankment Overtopping due to loss of spillway capacity	• Complete loss of spillway capacity (blockage due to pipelines, debris, storage of materials etc.) • 72 hr storm event to exceed freeboard from maximum operating pond (RL 198.9m) to embankment design crest (RL 200.0m). • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O3	Embankment Overtopping due to crest scour from pipeline burst	• Either tailings or return water pipeline bursts on embankment crest. • Pipeline burst and scour goes un-noticed and is not remedied • Flow of tailings slurry/water causes deep scouring down below spillway invert level (<199.5 m) • 72 hr storm event to exceed freeboard from maximum operating pond (198.9m) to scour level. • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O4	Embankment Overtopping due to poor deposition management - Spillway Blockage	• Unapproved change in deposition strategy results in tailings being deposited from southern embankment adjacent to the emergency spillway. • Tailings deposited embankment crest level (RL 200.0m), and are deposited across spillway mouth to a depth of 250 mm (new invert RL 199.75m), reducing storm storage capacity and spillway outflow capacity • Critical duration storm event to overwhelm reduced spillway from new maximum operating pond (199.75m) to embankment design crest (RL 200.0m). • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O5	Embankment Overtopping due to poor deposition management - Decant Issues	• Unapproved change in deposition strategy results tailings being deposited from entire perimeter and pushing water away from the decant structure, and water is unable to be removed from the TSF. • Decant water builds up to spillway invert level (RL 199.5m). • Critical duration storm event to overwhelm spillway from invert level to embankment design crest (RL 200.0m). • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.

Ref. No	Failure Mode	Failure Progression
O6	Embankment Overtopping due to poor deposition management - Over deposition	• Unapproved change in deposition strategy results in tailings being deposited above the design head of beach of RL 199.5m to RL 200.0m, reducing storm storage capacity. • Decant water builds up to spillway invert level. • Critical duration storm event to overwhelm spillway from invert level to embankment design crest RL 200.0m. • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O7	Embankment Overtopping due to build-up of excess tailings bleed water	• Significant change in tailings depositional properties (increased throughput, reduced solids concentration, and low settling density) from design parameters results in excessive tailings bleed. • Decant structure unable to remove excess water. Additional pumps not mobilised, and surface water builds up to spillway invert level. • Critical duration storm event to overwhelm spillway from invert level to embankment design crest RL 200.0m. • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O8	Embankment Overtopping due to higher than expected operating pond levels	• Design of facility for surface water (water balance, decant structure, pump capacity) is unsuitable. • Additional design measures are not implemented, and surface water builds up under normal operating conditions. • Critical duration storm event to overwhelm spillway from invert level to embankment design crest RL 200.0m. • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O9	Embankment Overtopping due to multiple large storms that exhaust freeboard and exceeds spillway capacity	• Initial large storm event occurs, raising decant pond to the spillway invert level. • Secondary large storm occurs relatively soon after. Critical duration storm event to overwhelm spillway from invert level to embankment design crest RL 200.0m. • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O11	Embankment Overtopping due to reduced spillway capacity from seismic induced crest settlement	• Seismic event causes significant embankment deformation, resulting in a localised low spot and effectively lowering the embankment crest level for overtopping to occur. • Critical duration storm event to overwhelm spillway from maximum operating pond to deformed embankment crest level. • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.
O12	Embankment Overtopping due to scour from failure of Spillway Erosion Protection Rip-Rap	• Large storm event causes rapid flow down spillway channel, starting at maximum operating pond. • Designed erosion protection measures (rip-rap and geotextiles) are unsuitable for use and quickly washes away. • Flow of water down unprotected embankment crest scours embankment, eroding the rockfill material and progressing an embankment failure. • Uncontrolled flow over embankment crest and downstream slope, eroding the rockfill material and resulting in a run-away failure of the embankment.

6.3.3 Results

The overall assessed probability of the overtopping failure cases is summarised in **Table 11**.

TABLE 11: OVERTOPPING FAILURE CASES ASSESSED PROBABILITY

Ref. No	Preceding Events		Overtopping Storm Event			Overall Assessed Probability
	Event	Estimated Value	Depth (mm)	Duration (Hrs)	AEP	
O1	Complete loss of spillway capacity	1×10^{-3} (1)	890	72	1.3×10^{-6}	1.3×10^{-9}
O3	Pipeline burst on embankment crest.	1×10^{-2} (1)	450	72	5.0×10^{-5}	5.0×10^{-10}
	Pipeline burst un-noticed for significant period of time to cause >0.5m scouring.	1×10^{-3} (1)				
	Total	*1×10^{-5}*				
O4	Unapproved change in deposition strategy (from southern embankment).	1×10^{-2} (1)	625	4	In excess of PMP ($>1 \times 10^{-6}$)	>> 1.0×10^{-10} (Note 2)
	Tailings deposited above maximum design level.	1×10^{-2} (1)				
	Total	*1×10^{-4}*				
O5	Unapproved change in deposition strategy (entire perimeter deposition).	1×10^{-2} (1)	825	2	In excess of PMP ($>1 \times 10^{-6}$)	>> 1.0×10^{-9} (Note 2)
	Deposition pushes water away from decant tower, cannot be removed.	1×10^{-1} (1)				
	Total	*1×10^{-3}*				
O6	Unapproved change in deposition strategy (deposition above maximum to RL 200.0m). *Note, this results in Maximum Operating Pond volume being at RL 199.4m*	1×10^{-2} (1)	600	2	In excess of PMP ($>1 \times 10^{-6}$)	>> 1.0×10^{-8} (Note 2)
O7	Significant, continual change in tailings depositional characteristics.	1×10^{-3} (1)	825	2	In excess of PMP ($>1 \times 10^{-6}$)	>> 1.0×10^{-10} (Note 2)
	Excess bleed water builds up, cannot be removed	1×10^{-1} (1)				
	Total	*1×10^{-4}*				
O8	Surface water management design insufficient/unsuitable.	1×10^{-4} (1)	825	2	In excess of PMP ($>1 \times 10^{-6}$)	>> 1.0×10^{-10} (Note 2)
	Additional design measures not implemented to prevent build-up of surface water.	1×10^{-1} (1)				
	Total	*1×10^{-4}*				
O9	72Hr Storm event raises water level to spillway invert.	5.0×10^{-5}	825	2	In excess of PMP ($>1 \times 10^{-6}$)	>> 5.0×10^{-11} (Note 2)
O11	Seismic event causes significant crest deformation.	1×10^{-4}	1,350	12	In excess of PMP ($>1 \times 10^{-6}$)	>> 1.0×10^{-10} (Note 2)
O12	Designed erosion protection measures insufficient/unsuitable.	1×10^{-4} (1)	500	72	2.0×10^{-5}	2.0×10^{-9}

Note (1) Adopted from subjective probability estimation guide, **Table 6.**

(2) Failure modes requiring storm events larger than the PMP have the overall assessed probability as “>>” of the combined probability, indicating the probability is well below this assessed value.

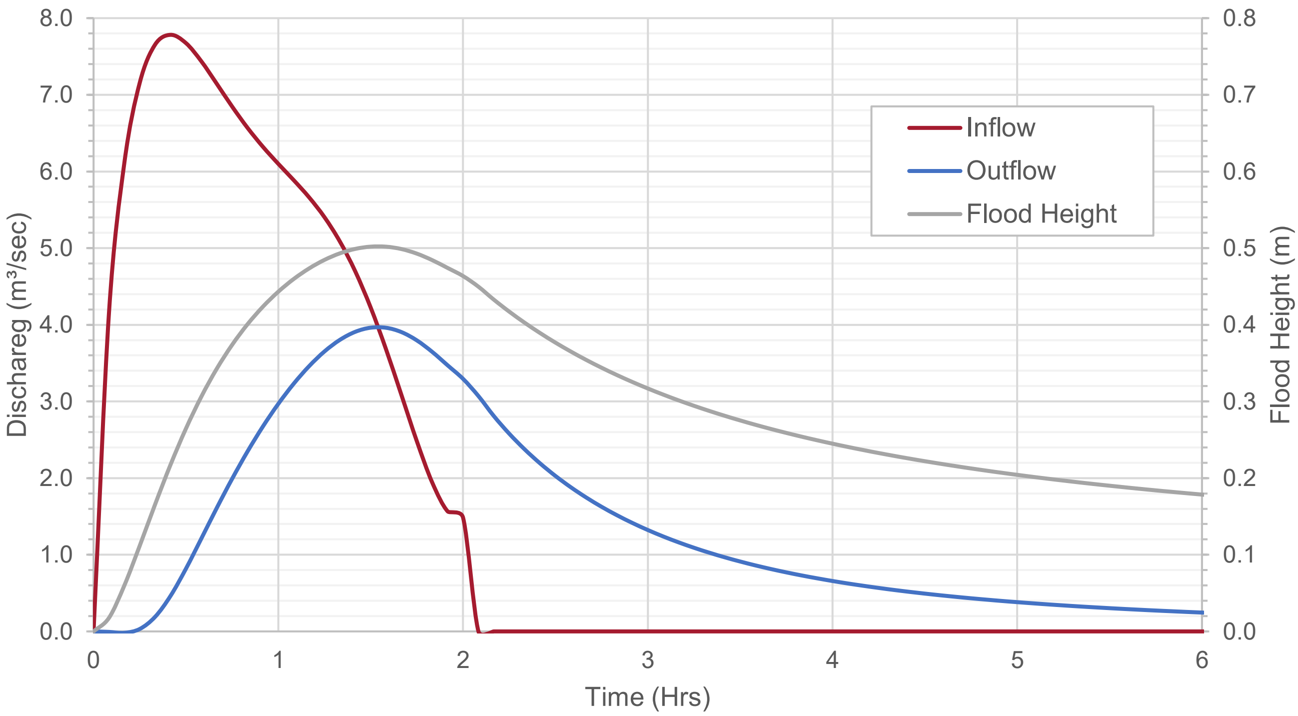
As is shown in **Table 11**, multiple of the embankment overtopping potential failure modes were found to require storm events in excess of the PMP to initiate failure. As such, the associated overall failure probabilities were estimated as incredibly low, and are considered to provide a negligible contribution to the overall risk of failure.

These failure scenarios were associated with storm events that were required to produce a flood that exceeded the capacity of the TSF emergency spillway, then overtopping the embankment crest. This can be attributed to two key factors;

1. The Brunswick West TSF is a paddock style TSF, where the catchment area is limited by the confines of the embankment. As such, no external catchments can feed into the facility and flood waves are limited by precipitation falling directly on the TSF itself.
2. The emergency spillway was designed to pass the flood wave from the PMP event with an additional 0.2 m of freeboard above the predicted peak flood height.

The flood inflow and outflow hydrograph for Case O6 (tailings over deposition) is presented in **Chart 3** as an example. It can be seen that the application of a 2 hour 600mm storm (50% larger than the PMP value) is required for the peak flood height to exceed the spillway depth of 0.5m. These failure scenarios require excessively large storm events significantly greater than the PMP to cause an overtopping failure, and provide a negligible contribution to the overall risk of failure.

CHART 3: CASE O6 – SPILLWAY INFLOW-OUTFLOW HYDROGRAPH



The remaining potential failure modes considered as plausible (i.e., the size of the overtopping storm event is less than the PMP) were generally associated with bypassing the spillway (either through blockage or deep scour erosion), these being Case O1 and O3, with estimated probabilities in the range of 10^{-9} to 10^{-10}. For failure to occur in these scenarios, high rainfall long duration storm events would be required to consume the remaining freeboard and result in embankment overtopping. These failure modes are predicated on a significant defect or flaw in the embankments, and the overall probability can be further reduced (or completely removed) through routine inspections of key aspects of the facility, prompt repair works where necessary, and the removal of excess surface water from the TSF when large storm events are anticipated.

Case O12 (where the spillway is activated, but the erosion protection measures completely fail) can be mitigated through proper engineering design of the erosion protection measures, and well supervised and documented construction undertaken to ensure the design intent is met.

Additionally, as discussed in **Section 6.3.2**, no additional considerations have been made for the erosion resistance of the embankment downstream shell, which would further reduce the likelihood of embankment failure. As such, the probabilities presented in **Table 11** are considerably conservative.

An interpretation of the assessed probabilities in combination with the consequences of failure is further discussed in **Section 7.**

6.4 Embankment Instability

6.4.1 Credible Failure Cases

The failure cases related to embankment instability that were identified as potentially credible as part of the initial screening are outlined below:

Case S1	Embankment Instability due to incorrect material characterisation – Embankment fill materials
Case S2	Embankment Instability due to incorrect material characterisation - Foundation materials
Case S3	Embankment Instability due to high phreatic surface
Case S4	Embankment Instability due to inadequately constructed embankments
Case S5	Embankment Instability due to inadequately prepared foundations

6.4.2 Method of Assessment

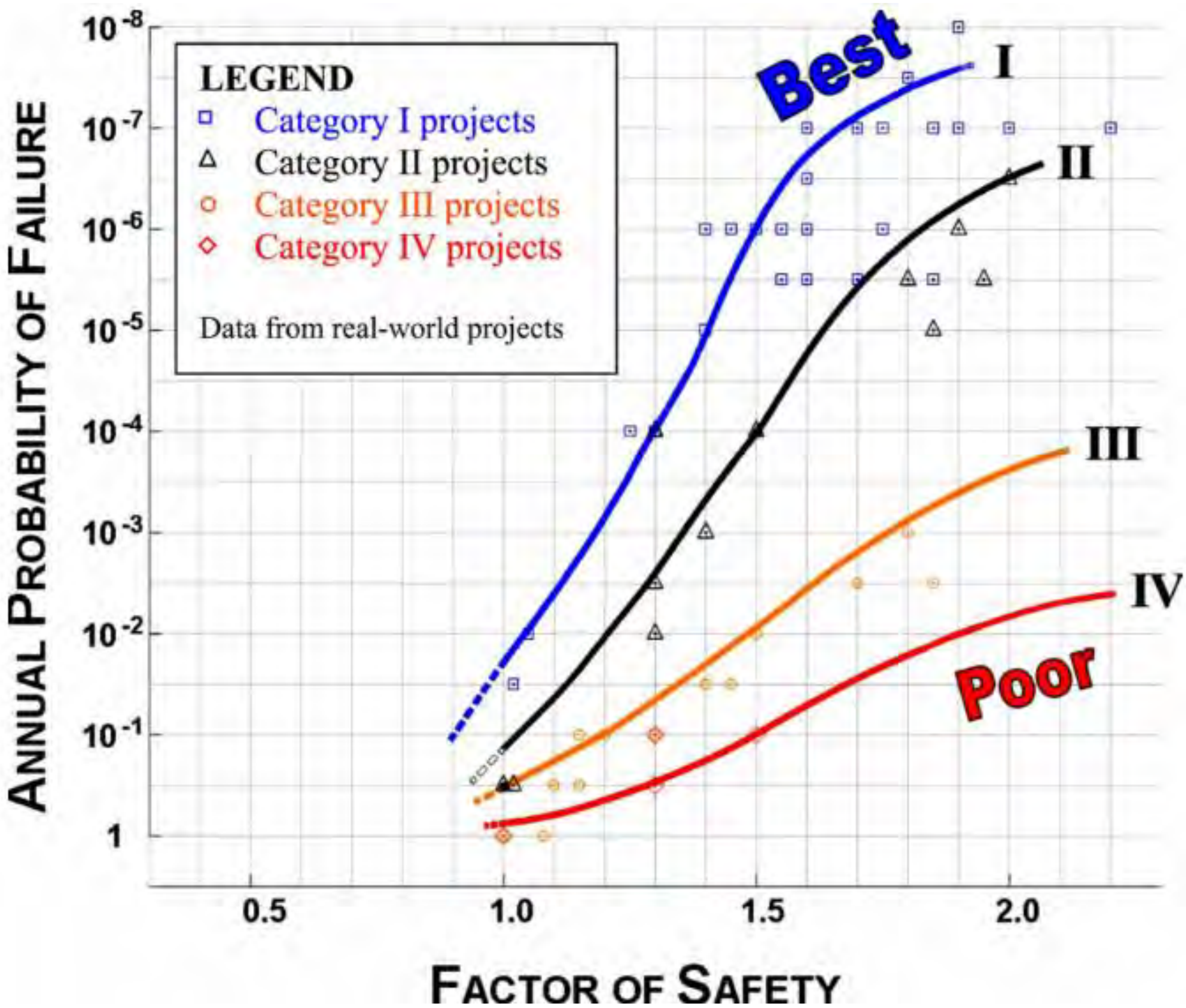
As part of the design of the Brunswick West TSF embankments **[1]**, ATCW undertook a conventional deterministic slope stability assessment. This process included the development of representative and conservative material strength parameters from in-situ and laboratory testing, and applying these material strength parameters to the embankment design and assessing the overall Factor of Safety (FoS) against embankment failure, and ensuring the minimum FoS found in the model is in excess of the ANCOLD **[4]** recommended minimum values. This stability assessment uses constant material strength parameters, and cannot produce a probability of embankment failure.

To accurately relate embankment design and stability to a likelihood of failure, a probabilistic stability assessment would be required. This process involves the generation of representative probability density functions for embankment material parameters, which are then input into the slope stability model and a Monte Carlo assessment is conducted. The probability of embankment instability failure is then estimated by the frequency at which the stability model produces a FoS below 1.0. To generate material probability density functions, a large amount of laboratory and in-situ testing is required to produce a sufficient representative sample size to generate the relevant statistical parameters (i.e., mean, minimum/maximum and standard deviation). This amount of investigation work was not conducted for the Brunswick West TSF, and a Monte Carlo assessment cannot be undertaken.

Alternatively, to allow for the relationship between embankment stability and likelihood of failure to be estimated, the methods described by Silva et al (2008) **[16]** have been adopted. This study presented a semi-empirical relationship between embankment factor of safety and annual probability of failure based on historical case studies of existing embankments of known design, construction and operation characteristics, and estimations of the probabilities of failure for each embankment with panels of dam experts. From this, 4 profiles were generated to relate embankment FoS with probability, from Category I (highest level of engineering design) to Category IV (little to no engineering design). For the Brunswick West TSF, Category II has conservatively been adopted.

The FoS vs Annual Probability of Failure profiles (extracted from Silva et al (2008) **[16])** are presented in **Chart 4**.

CHART 4: FACTOR OF SAFETY VS ANNUAL PROBABILITY OF FAILURE (EXTRACTED FROM SILVA 2008 [16])



The stability analyses were undertaken with the proprietary software SLOPE/W **[17]**, utilising the General Limit Equilibrium (GLE) method, satisfying both force and moment equilibrium criteria. Static and post seismic stability conditions have been considered for the Brunswick West TSF critical embankment section.

The relevant triggering events considered, and their implementation within the slope stability model, is summarised below;

Incorrect embankment fill characterisation	Modelled as a 20% reduction in peak strength.
Incorrect foundation characterisation	Modelled as a 20% reduction in peak strength.
High phreatic surface	Modelled as maximum phreatic surface from maximum operating pond to embankment downstream toe.
Inadequately constructed embankment	All rockfill material modelled as loose dumped rockfill at angle of repose.

Inadequately prepared foundations	Loose clay left within foundations susceptible to seismic liquefaction. Modelled as liquefied clay with an undrained strength ratio of 0.09, plus a 20% reduction in peak strength for the remaining materials to represent strain softening.

The potentially credible embankment instability failure cases, along with a summary of likely failure progressions, are presented in **Table 10.**

TABLE 12: POTENTIALLY CREDIBLE EMBANKMENT INSTABILITY FAILURE CASES

Ref. No	Failure Mode	Failure Progression
S1	Embankment Instability due to incorrect material characterisation – Embankment fill materials	• Laboratory testing and adopted embankment material strength parameters not suitable for the embankment, and over-representative embankment fill material strength parameters are adopted for use • At the end of tailings deposition, slip failure occurs through the embankment fill materials, results in a failure of the embankment.
S2	Embankment Instability due to incorrect material characterisation - Foundation materials	• Investigation, laboratory testing and adopted foundation material strength parameters not suitable, and over-representative foundation strength conditions are adopted for use. • At the end of tailings deposition, slip failure occurs through the foundations, results in a failure of the embankment.
S3	Embankment Instability due to high phreatic surface	• Flaw in BGM liner occurs, causing a phreatic surface to develop through the majority of the embankment. • Phreatic surface through embankment significantly higher than predicted by seepage analyses. • At the end of tailings deposition, slip failure occurs through the foundations, results in a failure of the embankment.
S4	Embankment Instability due to inadequately constructed embankments	• Inadequate compaction undertaken for all rockfill (Zone 3A and 3B) materials, resulting in material strengths closer to that of loose, dumped rockfill. • At the end of tailings deposition, slip failure occurs through the foundations, results in a failure of the embankment.
S5	Embankment Instability due to inadequately prepared foundations	• Inadequate stripping and re-compaction of foundation clays undertaken, resulting in loose, weak foundation materials beneath embankment. • Significantly large seismic event occurs, causing the foundation clays liquefy. • At the end of tailings deposition, slip failure occurs through the foundations, results in a failure of the embankment.

6.4.3 Results

The results of the embankment instability assessment is summarised in **Table 13** , and the graphical Slope/W outputs are presented in **Appendix C**. In addition to the identified failure cases in **Section 6.4.2**, an additional failure mode (Case 3.1) has also been assessed, considering the combined effects of a failure of the design elements.

TABLE 13: EMBANKMENT INSTABILITY ASSESSMENT

Ref. No	Failure Mode	Preceding Events		Slope Stability Assessment		Overall Assessed Probability	Fig.
		Event	Estimated Value	FoS	Estimated Probability (from Silva)		
Base	*Base Case (Long Term Static Stability)*	*None*	-	*2.97*	-	-	*C1*
S1	Incorrect material characterisation – Embankment fill materials	Laboratory testing insufficient/unsuitable, and adopted design embankment material strength parameters too high	1×10^{-4} (1)	2.66	1×10^{-7}	1.0×10^{-11}	C2
S2	Incorrect material characterisation - Foundation materials	Investigation and Laboratory testing insufficient/unsuitable, and adopted design foundation strength parameters too high	1×10^{-4} (1)	2.57	1×10^{-7}	1.0×10^{-11}	C3
S3	High phreatic surface	Significant tear in BGM liner that goes un-noticed and un-repaired	1×10^{-2} (1)	2.20	1×10^{-7}	1.0×10^{-13}	C4
		Increased seepage causes phreatic surface significantly higher than predicted in design	1×10^{-4} (1)				
S3.1	Combined failure of design elements	All design elements as above	1×10^{-8} (2)	1.71	3×10^{-5}	3.0×10^{-13}	C5
S4	Inadequately constructed embankments	Inadequate compactions undertake for all rockfill, with materials as loose rockfill	1×10^{-3} (1)	2.71	1×10^{-7}	1.0×10^{-10}	C6
Base	*Base Case (Post Seismic Stability)*	1:10,000 AEP Seismic Event	1×10^{-4}	*2.07*	-	-	*C7*
S5	Inadequately prepared foundations	Inadequate stripping and re-compaction of foundation clay, remaining in loose state	1×10^{-3} (1)	1.78	6×10^{-5}	6.0×10^{-12}	C8
		1:10,000 AEP Seismic Event	1×10^{-4}				

Note (1) Adopted from subjective probability estimation guide, **Table 6.**

(2) Combined from product of Insufficient Laboratory Testing and Increased Seepage

The stability assessments found that for all arrangements considered, a significant factor of safety against embankment instability is expected. All assessed factors of safety were significant, resulting in a minimum estimated probability of failure (from Silva (2008) **[16]**) in the order of 10^{-5}. The lowest factor of safety of the embankment conditions assessed was found for Case S3.1, considering a 20% reduction in all material strengths, as well as a very high phreatic surface through the embankment. This aimed to assess the worst possible scenario for design failure of the embankments (i.e. gross mischaracterisation of the material properties and unsuitable design undertaken). The factor of safety was 1.7, resulting in the estimated probability of 3×10^{-5}. It is noted that within the Slope/W model, these conditions would be the same as a post seismic failure with a high phreatic surface.

As demonstrated in **Table 13,** once consideration is made for the estimated probability of the preceding events occurring, the overall estimated probabilities of failure are incredibly low (10^{-10} or lower), demonstrating the overall robust design and extreme prevailing conditions that would be required to fail the embankments of the Brunswick West TSF.

The high factors of safety and resistance against instability (hence low probability of failure) can be attributed to two main factors;

- The geometric design of the embankment, incorporating a 4:1 (H:V) downstream batter slope to provide significant resistance against failure, and
- The geosynthetic liner to ensure the phreatic surface is maintained low through the embankment cross section, improving the overall strength of the embankment materials. This is a key design component, as it was identified that the highest single factor that produced the greatest decrease in FoS was due to the very high phreatic surface (almost completely saturating the entire downstream zones).

It is considered highly unlikely that any combination detrimental embankment conditions and seismic loading will result in a scenario where the assessed embankment factor of safety is below 1.0, resulting in failure of the embankment. As such, based on the overall assessed probabilities of failure presented in **Table 13**, the embankment instability cases are considered to provide a negligible contribution to the overall risk of failure.

6.5 Combined settlement and erosion failure

6.5.1 Overview and Method of Assessment

As mentioned in **Section 5.1**, a combined settlement and erosion failure was identified as potentially credible to provide verification of a potential sunny day embankment failure **[1]**. This failure involved the cumulative static settlement and seismic deformation of the embankment crest leading to the release of liquefied tailings and operational water, which erodes the embankment downstream face and leads to a cascading failure. As such, this failure case does not fit into the three broad categories identified for failure (geotechnical piping, overtopping from storm events, or embankment instability).

The failure progression for this case is summarised below:

- Static settlement of the embankment occurs post-construction due to poor quality controls and saturation (loose, uncompacted materials), which is un-noticed and not remedied by mine personnel.
- At the end of filling, a suitably large earthquake occurs, causing a significantly large amount of seismic deformation, as well as completely liquefying the tailings.
- The post-construction static settlement and seismic deformation of the embankment exceeds the remaining freeboard to the tailings head of beach RL 199.5 m, leading to operational water and liquefied tailings to mobilising over the lowered embankment crest.
- The liquefied tailings and water progress quickly over the embankment crest and down the slope, eroding the rockfill material, causing an unravelling failure of the embankment and releasing tailings and water.

This failure scenario involves multiple sequential steps to occur, and has been assessed by cumulatively considering the contributing factors to overall failure, as described in **Equation 1**.

As with the overtopping cases (refer **Section 6.3.2**), failure was considered to have occurred once fluid material (liquefied tailings and/or operational water) was released over the embankment crest in an uncontrolled manner. No additional considerations has been made for the erosion resistance of the embankment downstream shell, which would further reduce the likelihood of embankment failure, and the results estimated in the following section will be considerably conservative.

6.5.2 Results

Estimation of the probability of failure for this combined erosion-based failure event has been undertaken considering the individual probabilities of the contributing events. As outlined in **Section 6.1**, a minimum of 0.5m beach freeboard is specified from the embankment crest level to the tailings. As such, the combined static settlement and seismic deformation would need to exceed this design freeboard.

The probabilities of the contributing faults are discussed below;

- Static Settlement due to poor construction of the embankments – Based on the maximum amount of seismic deformation expected (225mm), at least 275mm of static settlement would be required post seismic event to exceed the minimum 0.5m beach freeboard. For a 15m high embankment, this is equal to static settlement of approximately 1.8%.

  Static settlement of rockfill can generally be expected as a result of saturation cycles. Based on studies of the behaviour of sedimentary rock masses **[18]**, up to 1% static settlement and can expected for well compacted rockfill, and 2% for loose rockfill. As such, the minimum 1.8% of settlement would require large portions of the embankment to remain uncompacted, and then saturate upon filling and rainfall cycles. This is not expected to be possible given that full time QA/QC will be provided for the embankment to ensure adequate rockfill compaction and liner installation for facility. However, a single scenario can be envisaged whereby this QA/QC is not undertaken, and rapid embankment construction is then undertaken to ensure completion of the facility in time, resulting in rockfill compaction being omitted, and poor quality liner installation resulting in large areas of defects. Based on the subjective probability estimation guide (refer **Table 6**), this event has been assigned a probability of 1 x 10^{-3}.

- Static Settlement is un-noticed and not remedied – The likely failure scenario is only expected to occur at the end of filling, when MRCO are expected to have ceased operations at the Costerfield Gold Mine. As such, the repair of settlement may be identified as a low priority, and the repair works are not undertaken. Based on the subjective probability estimation guide (refer **Table 6**), this event has been assigned a probability of 1 x 10^{-2}.

- Seismic Deformation – Deformation assessments of the embankments undertaken as part of the detailed design **[1]** identified that, for a 1 in 10,000 AEP seismic event, that up to 1.5% of settlement could occur. For a 15m high embankment, this is equal to 225 mm. This is equal to a probability of 1 x 10^{-4}.

- Tailings Liquefaction given the seismic event – As identified in the detailed design report **[1]**, the tailings within the Brunswick West TSF are expected to be liquefiable from a 1:10,000 AEP seismic event. This assessment was based on the liquefaction potential of the Bombay TSF tailings at the perimeter of the facility under the 1:2,000 AEP event, which found the saturated tailings as likely to liquefy. As such, the probability of this event occurring is 1.

The combination of the above contributing factors results in a probability of failure in the order of 10^{-9}. This is considered a reasonably conservative estimate, as this scenario was identified as the most probable sunny day failure case, and is more likely than the geotechnical piping and embankment instability scenarios considered.

7 EMBANKMENT FAILURE RISK ASSESSMENT AND TOLERABILITY

7.1 Level of Risk

Assessment of the overall risk tolerability for catastrophic failure of an embankment should consider both the likelihood of failure as well as the consequences should a failure occur. Acceptance and tolerance of the risks of failure is then made in consideration of efforts made to reduce the level of risk compared with their incremental effects.

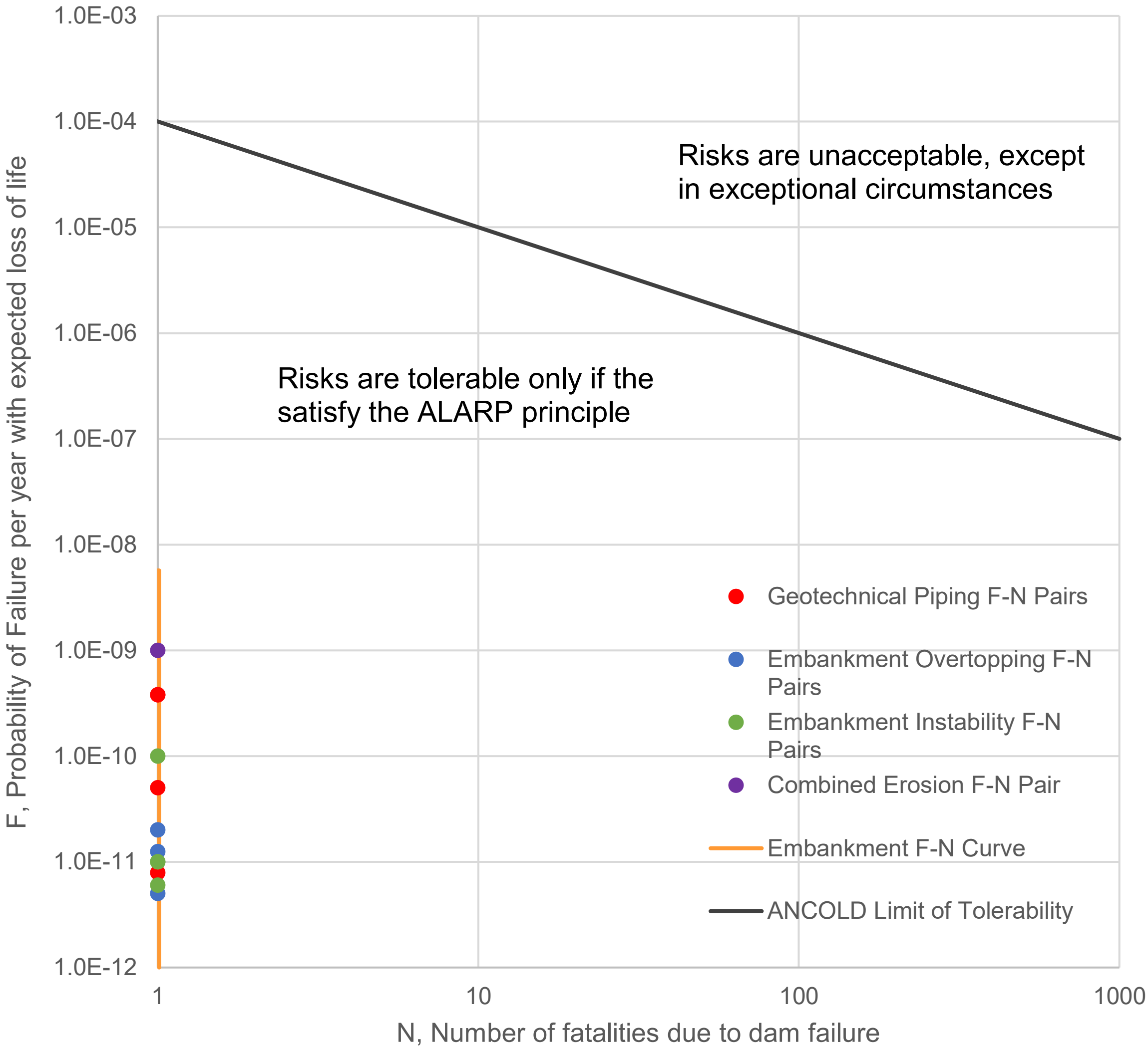
The ANCOLD Guidelines on Risk Assessment **[8]** provide guidance for the tolerability of public safety risks for the general community and workers associated with the facility, and considers the relationship between the annualised probability of failure and the potential number of fatalities due to dam failure. This is presented as an F-N chart, or the ANCOLD Societal Risk Guidelines, as presented in **Chart 5.** This chart presents the individual F-N pairs for each of the failure cases considered, as well as the combined F-N curve for the Brunswick West TSF.

This chart can be split into two regions;

- Risks are unacceptable, except in exceptional circumstances.
- Risks are tolerable only if the satisfy the “As Low As Reasonably Practicable (ALARP)” principle.
  - This is delineated from the above region by the “Limit of Tolerability” line.
  - The ALARP principle is further discussed in **Section 7.2**.

Based on the estimations of the PLL for a dam break scenario (refer **Section 3.4**) and the probabilities of failure (refer **Section 6)**, the risk tolerability for the Brunswick West TSF is presented in **Chart 5.**

CHART 5: ANCOLD SOCIETAL RISK GUIDELINES (NEW DAMS)



Note: ANCOLD provides the Limit of Tolerability line to a lower bound N value of 1. For fractional N values (i.e., N < 1), the F-N pairs have been presented at N = 1, with the probability values adjusted to compensate.

It can be seen from **Chart 5** that the F-N pairs for the assessed failure cases and the cumulative embankment F-N curve plots well below the ANCOLD Limit of Tolerability for new dams, and are within the region where the risk can be considered as tolerable if they satisfy the ALARP principle.

7.2 ALARP Principle

The ANCOLD Risk Guidelines **[8]** discuss the tolerability of risk in terms of the As Low as Reasonably Practicable (ALARP) principle. The ALARP principle considers all potential risk mitigation measures that can be implemented, and compared the potential incremental costs or level of effort required to implement them versus the incremental decreases to the overall level of risk. A non-practicable risk mitigation measure would be one where the costs (financial, time frames, level of effort etc.) are grossly disproportionate to the improvement that may be gained.

The risk mitigation and failure control measures proposed for design, construction and operation are summarised in **Section 5.2**.

The key ALARP considerations for the proposed Brunswick West TSF are summarised below:

- The residual societal risk for the Brunswick West TSF design is roughly 4 orders of magnitude below the ANCOLD Limit of Tolerability for new dams.
- The proposed design for the Brunswick West TSF satisfies best practice design for the facility, and has been designed to meet and exceed the minimum requirements of ANCOLD. Key risk control measures from the TSF design are summarised below;
  - Inclusion of the BGM liner to control initiation for geotechnical piping.
  - Formation of the embankment downstream slopes to 4:1 (H:V), resulting in a significant Factor of Safety against embankment instability.
  - Excavation of the emergency spillway beyond the depth of the PMP peak flood height to control water overtopping the embankment crest.
- The proposed design also incorporates two flood diversion bunds to aid in the prevention of material (tailings and/or water) inundating the Brunswick Underground Portal in the events of a dam failure to remove the PLL from the Dam Break scenario. While the additional costs to construct these bunds is not insignificant, these flood protection measures have been considered as necessary to ensure the safety of the mine workers within the underground network.
- Construction of the Brunswick West TSF is proposed to have full time construction QA/QC to ensure the design specifications are met, including foundation preparation, material specifications, and material placement and compaction.
- An Operation, Maintenance and Surveillance Manual (OMS) will be prepared and implemented prior to commissioning of the Brunswick West TSF, in accordance with the operating requirements of ANCOLD.
  - This document will describe the routine inspections required for key elements of the facility, and subsequent actions to be conducted to limit the potential progression of embankment failure.
  - Surveillance and instrumentation monitoring requirements will also be documented, to be conducted regularly, and reviewed annually as part of the dams engineer inspections.
- A Dam Safety Emergency Plan (DSEP) will also be prepared and implemented prior to commissioning of the Brunswick West TSF, in accordance with the operating requirements of ANCOLD.
  - This document will describe the procedures to be followed by personnel in the inundation zone based on trigger levels of key elements of the facility. These trigger levels are defined on a traffic-light system, from lowest (within normal operating levels) to highest (failure is imminent), and provides instruction on the required actions to take at different levels.
  - Timely evacuation of mine personnel from the Brunswick Processing Plant area when approaching the highest trigger level will further reduce the PLL in the event of a Dam Break .
  - As part of the operation of the facility, the DSEP will be regularly tested to ensure mine personnel within the inundation zone are sufficiently trained on the actions to take at different trigger levels.

The incremental costs and level of effort associated with further engineering risk reduction works for the proposed Brunswick West TSF would be significant compared to the relatively small to negligible further risk reduction that could be achieved.

As such, ATCW consider that the design of the Brunswick West TSF satisfies the ALARP principle, and that the residual risks are tolerable.

8 CONCLUSION

This report has presented the results of a credible failure mode assessment for the Brunswick West TSF based on the proposed design criteria as described in the design report **[1]**. The majority of the initial potential failure modes were qualitatively determined as non-credible. The remaining potential failure modes were then quantitatively assessed and were found to have a very low probability of failure. The risk mitigation measures implemented as part of the design are considered to satisfy the ALARP principle, with further risk reduction measures deemed as disproportionate in the cost and effort required compared to the risk reduction achieved.

It is therefore concluded that, provided the facility is constructed and operated in accordance with the design criteria **[1]**, that the residual risks imposed to the community and mine personnel due to the presence of the Brunswick West TSF is tolerable and the facility meets the requirements of ANCOLD **[8]** for the management and tolerability of risk.

9 CLOSURE

Your attention is drawn to the “Conditions of Report” which appear at the end of this report.

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FIGURES

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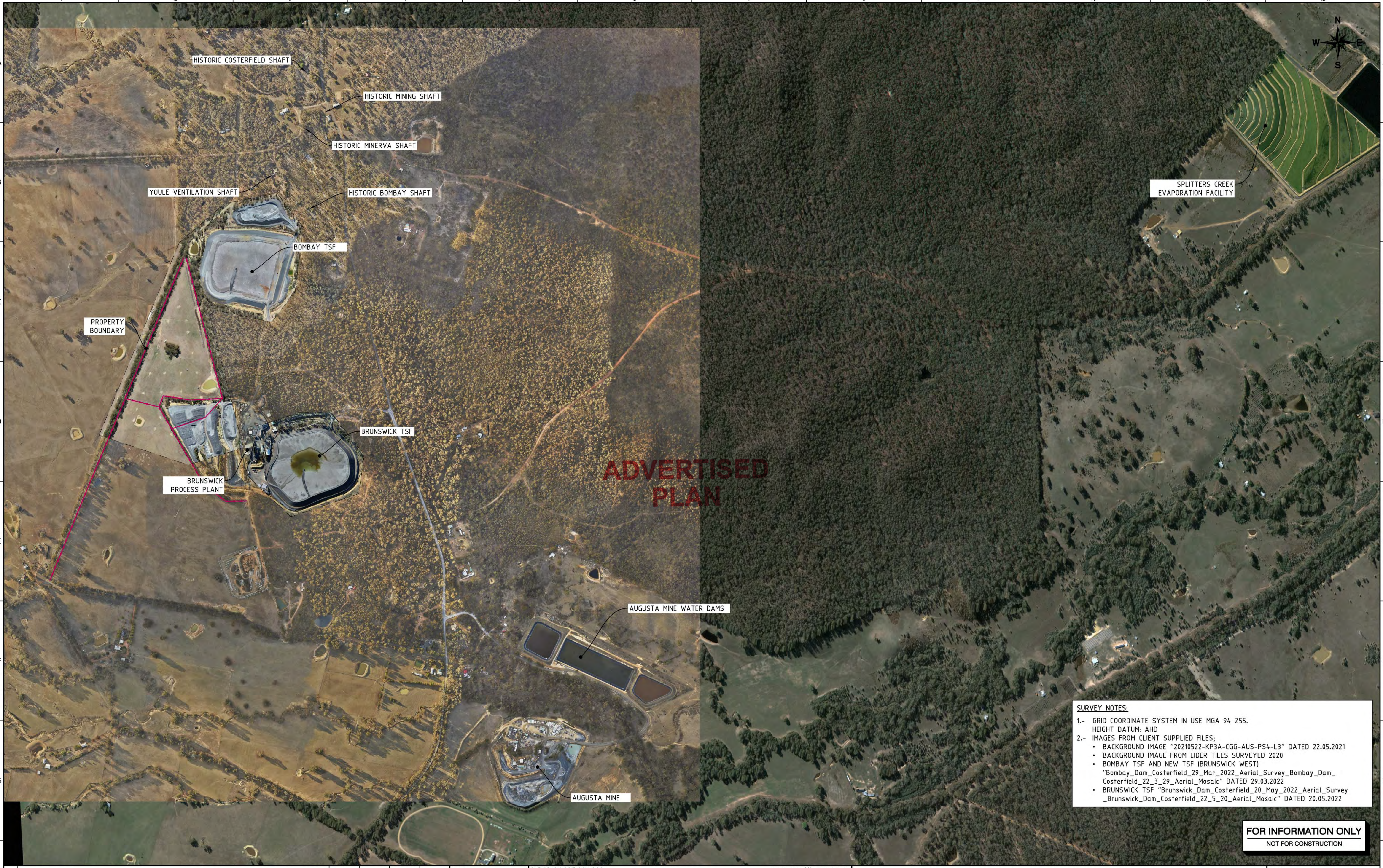

SURVEY NOTES:

1.- GRID COORDINATE SYSTEM IN USE MGA 94 Z55.
HEIGHT DATUM: AHD
2.- IMAGES FROM CLIENT SUPPLIED FILES;
- BACKGROUND IMAGE "20210522-KP3A-CGG-AUS-PS4-L3" DATED 22.05.2021
- BACKGROUND IMAGE FROM LIDER TILES SURVEYED 2020
- BOMBAY TSF AND NEW TSF (BRUNSWICK WEST) "Bombay_Dam_Costerfield_29_Mar_2022_Aerial_Survey_Bombay_Dam_Costerfield_22_3_29_Aerial_Mosaic" DATED 29.03.2022
- BRUNSWICK TSF "Brunswick_Dam_Costerfield_20_May_2022_Aerial_Survey_Brunswick_Dam_Costerfield_22_5_20_Aerial_Mosaic" DATED 20.05.2022

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BRUNSWICK WEST TAILINGS STORAGE FACILITY
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SHEET 1 OF 1

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- BRUNSWICK TSF "Brunswick_Dam_Costerfield_20_May_2022_Aerial_Survey_Brunswick_Dam_Costerfield_22_5_20_Aerial_Mosaic" DATED 20.05.2022

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BRUNSWICK WEST TAILINGS STORAGE FACILITY BRUNSWICK LOCALITY PLAN

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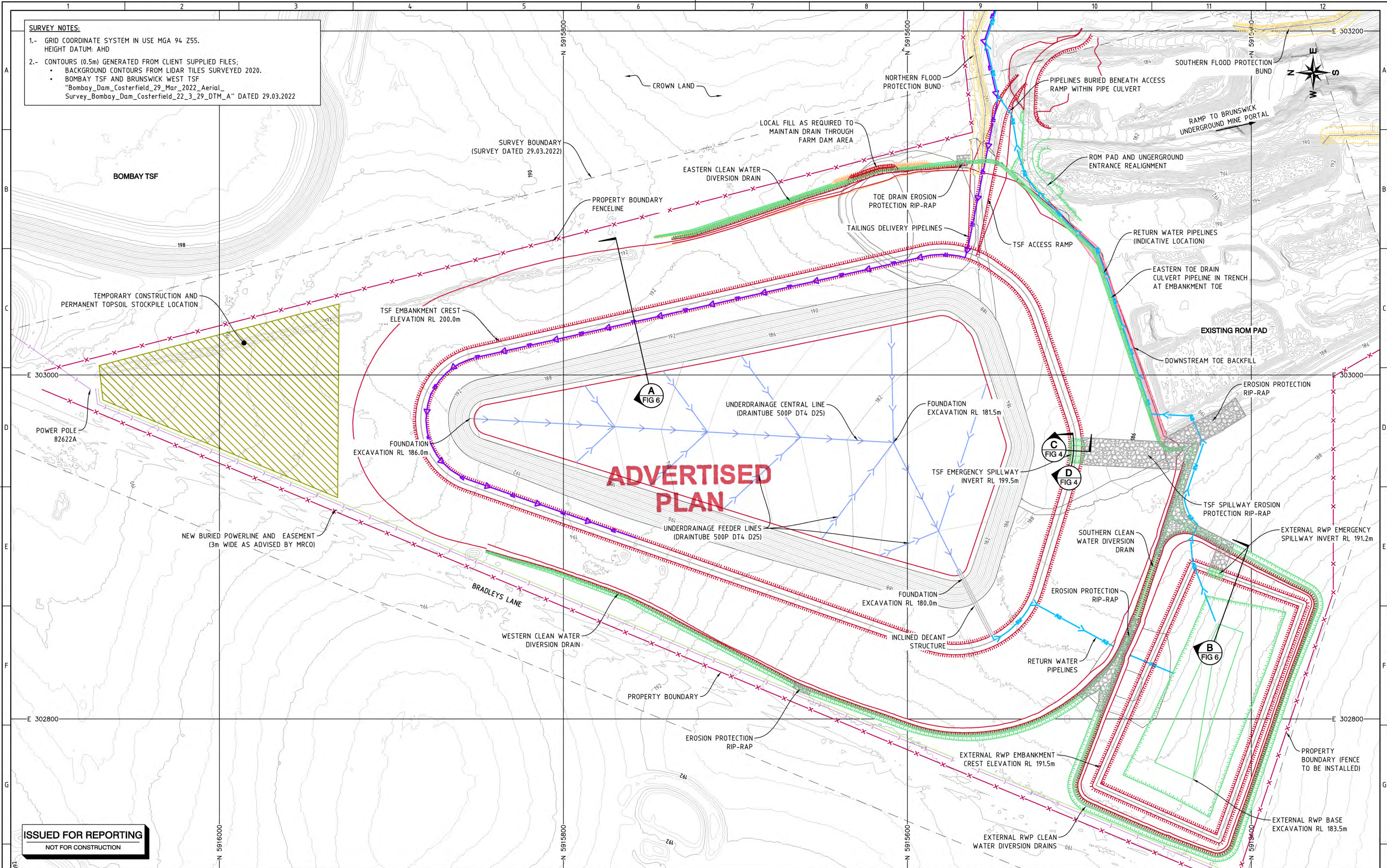

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FIGURE 3

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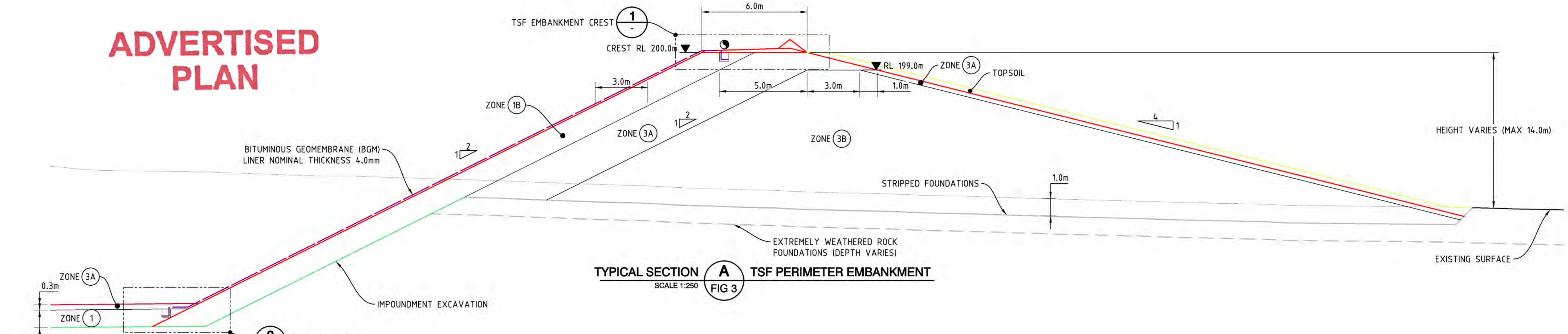
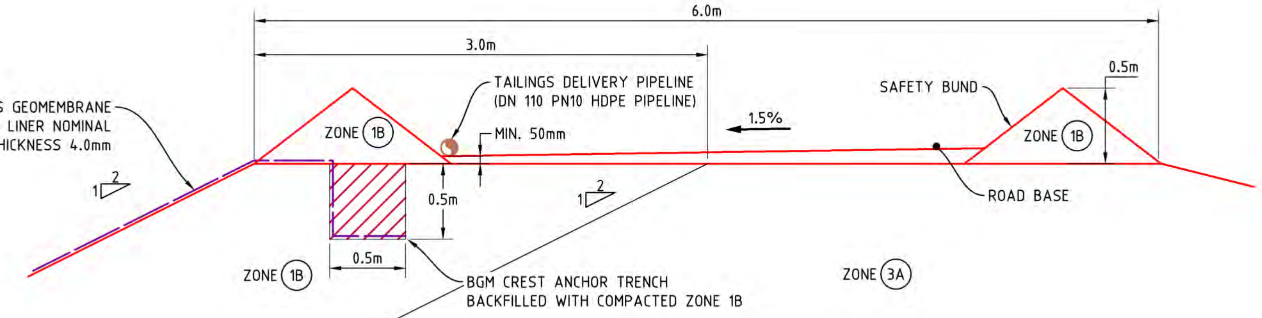
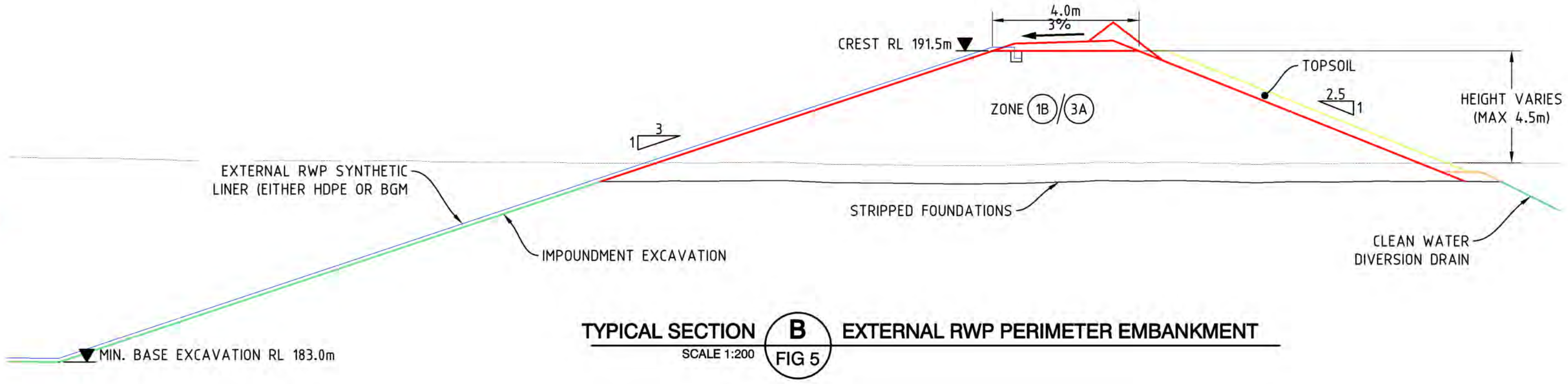


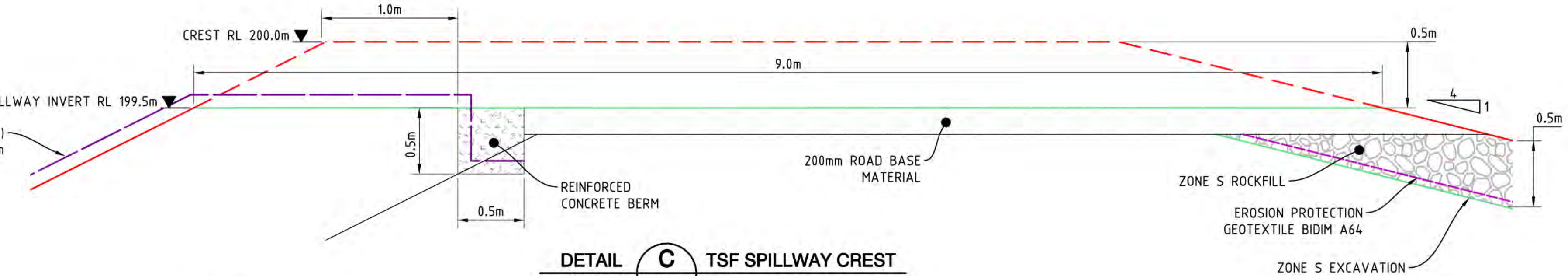
TYPICAL SECTION A TSF PERIMETER EMBANKMENT
SCALE 1:250 FIG 3



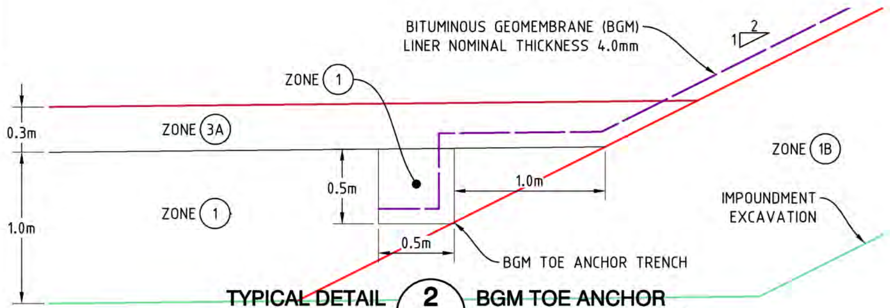
TYPICAL DETAIL 1 TSF EMBANKMENT CREST
SCALE 1:50



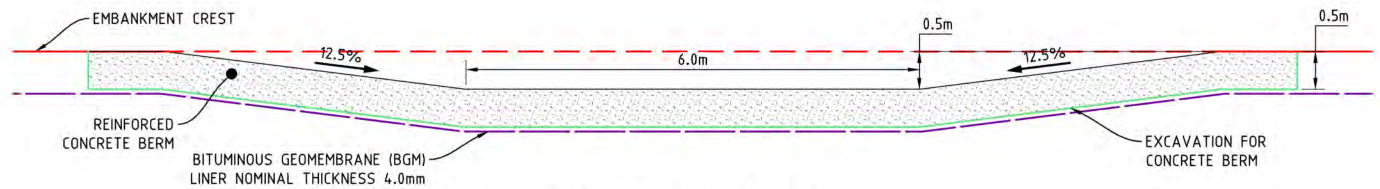
TYPICAL SECTION B EXTERNAL RWP PERIMETER EMBANKMENT
SCALE 1:200 FIG 5



DETAIL C TSF SPILLWAY CREST
SCALE 1:50 FIG 3



TYPICAL DETAIL 2 BGM TOE ANCHOR
SCALE 1:50



SECTION D EMERGENCY SPILLWAY
SCALE 1:100 FIG 3

MATERIAL ZONE DESCRIPTIONS

ZONE	DESCRIPTION
1	COMPACTED CLAY LINER. GRAVELLY CLAY. MATERIAL WON FROM IMPOUNDMENT EXCAVATION AND EMBANKMENT FOUNDATION STRIPPING. MATERIAL PLACED IN 250mm (LOOSE) LAYERS AND COMPACTED TO MIN. 98% STANDARD COMPACTION AND -2% TO +2 % OMC
1B	BGM EARTHFILL SUBGRADE. GRAVELLY CLAY TO CLAYEY GRAVEL. MATERIAL WON FROM IMPOUNDMENT EXCAVATION AND EMBANKMENT FOUNDATION STRIPPING. MATERIAL PLACED IN 300mm (LOOSE) LAYERS AND COMPACTED TO MIN. 98% STANDARD COMPACTION. BATTER SLOPE WITH MAXIMUM ASPERITIES OF LESS THAN 30mm
3A	TRANSITION FILL RESIDUAL, EXTREMELY & HIGHLY WEATHERED SILSTONE, WITH GRAVELS, SAND AND CLAYS. MATERIAL WON FROM UPPER IMPOUNDMNET EXCAVATION INTO ROCK. MATERIAL PLACED IN 300mm (LOOSE) LAYERS AND COMPACTED WITH VIBRATORY PADFOOT ROLLER (CONTRACTOR TO DETERMINE EQUIPMENT SIZING AND NUMBER OF PASSES WITH FIELD TRIALS)
3B	ROCKFILL. HIGHLY TO SLIGHTLY WEATHERED SILTSTONE, WITH GRAVELS AND SANDS. MATERIAL WON FROM LOWER IMPOUNDMENT EXCAVATION INTO ROCK, AND FROM STOCKPILE WHERE REQUIRED. MATERIAL PLACED IN 600mm (LOOSE) LAYERS AND COMPACTED WITH VIBRATORY PADFOOT ROLLER (CONTRACTOR TO DETERMINE EQUIPMENT SIZING AND NUMBER OF PASSES WITH FIELD TRIALS)
S	EROSION PROTECTION RIP-RAP MODERATELY WEATHERED TO FRESH ROCK. MATERIAL SCREENED FROM ZONE 3B MATERIAL. PLACED LOOSE IN A SINGLE LAYER WITHIN EXCAVATION

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FIGURE 4

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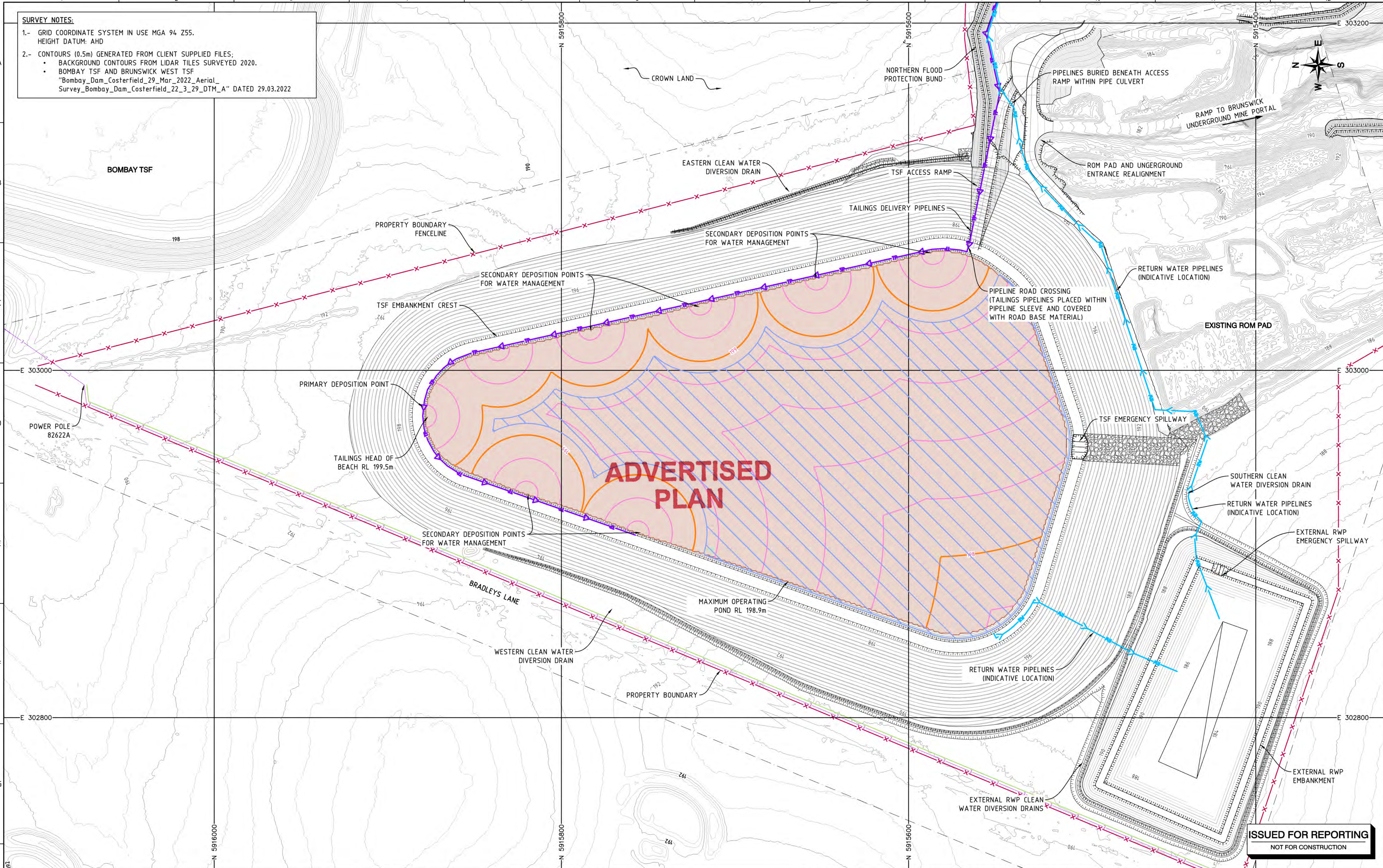

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CONDITIONS AT END OF FILLING

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FIGURE 5

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APPENDICES

ADVERTISED
PLAN

APPENDIX A – FAILURE MODE SCREENING

Client	**Mandalay Resources Costerfield Operations**
Site	**Costerfield Gold Mine**
Project	**Brunswick West Tailings Storage Facility**
Job No.	**109014.15**
Title	**Credible Failure Mode Assessment - Initial Qualitative Screening**
Prepared by	**Alex Campbell (ATCW)**
Date	**30-Nov-2022**

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Assumptions
Unless otherwise stated, the failure mode is assumed to occur at the critical stage, which is the end of filling of the TSF
These are failure modes that would result in a catastrophic failure of the embankment, resulting in released material

Reference	Failure Mode	Failure Progression	Controls			Failure Mode Credibility	Justification
			Design	Construction	Operation		
P1	Geotechnical Piping through embankment - Cracking caused by differential settlement from steep underlying topography	Significant tears/holes/flaws in Upstream Geosythetic Liner, which are not noticed or repaired. Water levels in the facility rise (either by storm event, or loss of decant removal capacity), begin to seep through flaws in the Geosynthetic Liner, and maximise head against the gravelly clay liner subgrade. Concentrated leak erosion against the gravelly clay subgrade allows excess seepage through the embankment interior. Excess seepage begins to erode the downstream rockfill, and is not filled in by material washed in from upstream. Intervention methods to stop the breach are unsuccessful. Embankment breaches, releasing tailings and water.	Design of facility with Geosynthetic Liner to prevent/manage seepage Removal and smoothing of potential steep topography changes (around diversion drains) to prevent localised differential settlement	Full time QA/QC provided to ensure design specifications for foundation preparation are met Dedicated Installation Crew, Testing regieme and QA/QC program to ensure Liner is installed and tested correctly	Regular removal of excess pooled water to minimise phreatic head differential against the clay Routine inspections to detect flaws in the Geosynthetic liner, and to monitor for signs of erosion progressing (cracking, seepage)	Not Credible	No abrupt changes in topography within majority of embankment which would allow for differential settlement. Localised areas (around diversion drains) will be removed and smoothed out
P2	Geotechnical Piping through embankment - Cracking caused by differential settlement of foundations		Design of facility with Geosynthetic Liner to prevent/manage seepage Loose material (topsoil and clay) removed from foundations, and remnant clay compacted (in excess of 98% Standard Compaction)	Full time QA/QC provided to ensure design specifications for foundation preparation are met Dedicated Installation Crew, Testing regime and QA/QC program to ensure Liner is installed and tested correctly	Regular removal of excess pooled water to minimise phreatic head differential against the clay Routine inspections to detect flaws in the Geosynthetic liner, and to monitor for signs of erosion progressing (cracking, seepage)	Not Credible	No deep or non-uniform changes in foundation conditions which may induce significant levels of differential settlement
P3	Geotechnical Piping through embankment - Cracking caused by loss of support from downstream shoulder		Design of facility with Geosynthetic Liner to prevent/manage seepage.	Full time QA/QC provided to ensure design specifications for rockfill placement and compaction are met Downstream rockfill to be compacted to a high density	Regular removal of excess pooled water to minimise phreatic head differential against the clay Routine inspections to detect flaws in the Geosynthetic liner, and to monitor for signs of erosion progressing (cracking, seepage)	Potentially Credible	
P4	Geotechnical Piping through embankment - Cracking caused by loose/poorly compacted layers in upstream clay zone		Design of facility with Geosynthetic Liner to prevent/manage seepage.	Full time QA/QC provided to ensure design specifications for placement and compaction of gravelly clay subgrade are met Gravelly clay subgrade to be compacted to a high density (in excess of 95% Standard Compaction)	Regular removal of excess pooled water to minimise phreatic head differential against the clay Routine inspections to detect flaws in the Geosynthetic liner, and to monitor for signs of erosion progressing (cracking, seepage)	Potentially Credible	
P5	Geotechnical Piping through embankment - Desiccation cracking through upstream clay zone		Design of facility with Geosynthetic Liner to prevent/manage seepage. Clay subgrade designed to be completely covered by Geosynthetic liner and rockfill to prevent desiccation cracks forming.	Full time QA/QC provided to ensure design geometry is met	Regular removal of excess pooled water to minimise phreatic head differential against the clay Routine inspections to detect flaws in the Geosynthetic liner, exposed subgrade, and to monitor for signs of erosion progressing (cracking, seepage)	Not Credible	Clay Subgrade designed to be completely covered (upstream face and crest) to prevent desiccation cracks forming
P6	Geotechnical Piping through embankment - Animal burrows and vegetation causing seepage path through embankment		Design of facility with Geosynthetic Liner to prevent/manage seepage, and prevent animals accessing the embankment upstream face directly.	Provision of animal-proof fencing around facility to prevent wildlife.	Regular removal of excess pooled water to minimise phreatic head differential against the clay Routine inspections to monitor for signs of animal burrows (holes in liner), or excessive vegetation Routine inspections to detect flaws in the Geosynthetic liner, and to monitor for signs of erosion progressing (cracking, seepage) Vegetation on embankment to be identified and promptly removed	Not Credible	Routine inspections of the liner will identify any deficiencies, and be promptly repaired
P7	Geotechnical Piping caused by transverse seismic cracking	Seismic Event occurs, causing differential settlement across embankment. Motion also creates tears in the Upstream Geosynthetic Liner. Differential settlement creates a deep crack across the clay subgrade down to water level within the facility (either down to operating pond level, or as deep as possible and rainfall causes the water level to rise). Seepage occurs through the gravelly clay subgrade, maximise head differential against the subgrade. Concentrated leak erosion against the gravelly clay subgrade allows excess seepage through the embankment interior. Excess seepage begins to erode the downstream rockfill, and is not filled in by material washed in from upstream. Intervention methods to stop the breach are unsuccessful. Embankment breaches, releasing tailings and water.	Design of facility with Geosynthetic Liner to prevent/manage seepage Embankment designed with spillway freeboard such that significant cracking is required to reach maximum pond levels	Full time QA/QC provided to ensure design specifications for rockfill placement and compaction are met to minimise potential for seismic cracking	Regular removal of excess pooled water to minimise phreatic head differential against the clay Inspection of facility following seismic events	Potentially Credible	
P8	Geotechnical Piping through foundations	Significant tears/holes/flaws in Upstream Geosynthetic Liner, which are not noticed or repaired. Water levels in the facility rise (either by storm event, or loss of decant removal capacity), begin to seep through flaws in the Geosynthetic Liner, and maximise head against the clay foundations. Uncompacted/loose foundations allow for high seepage rates beneath the embankment. Excess seepage begins to erode the downstream rockfill, and is not filled in up material washed in from upstream. Intervention methods to stop the breach are unsuccessful. Embankment breaches, releasing tailings and water.	Design of facility with Geosynthetic Liner extending to the base of foundation excavation. Loose material (topsoil and clay) removed from foundations, and remnant clay compacted (in excess of 98% Standard Compaction) Floor foundation is located below natural topography	Full time QA/QC provided to ensure design specifications for foundation preparation are met Dedicated Installation Crew, Testing regime and QA/QC program to ensure Liner is installed and tested correctly	Routine inspections to detect for signs of erosion progressing (cracking, seepage)	Not Credible	Tailings will form a low permeability seal against the upstream face and foundations, limiting the depth of standing water against the foundations to prevent the initiation of piping Only conceivable scenario for this to occur is partway through deposition with tailings just below foundation soil level and high pond levels. Failure in this case would only release a small amount of material (i.e. not a catastrophic failure)
P9	Geotechnical Piping into foundations	Significant tears/holes/flaws in Upstream Geosynthetic Liner, which are not noticed or repaired. Water levels in the facility rise (either by storm event, or loss of decant removal capacity), begin to seep through flaws in the Geosynthetic Liner, and maximise head against the gravelly clay subgrade Seepage works downwards through the embankment, directly into the foundations. Uncompacted/loose foundations allow for high seepage rates beneath the embankment. Excess seepage begins to erode the downstream rockfill, and is not filled in up material washed in from upstream. Intervention methods to stop the breach are unsuccessful. Embankment breaches, releasing tailings and water.	Design of facility with Geosynthetic Liner extending to the base of foundation excavation. Loose material (topsoil and clay) removed from foundations, and remnant clay compacted (in excess of 98% Standard Compaction).	Full time QA/QC provided to ensure design specifications for foundation preparation, and placement/compaction of the clay subgrade are met Dedicated Installation Crew, Testing regime and QA/QC program to ensure Liner is installed and tested correctly	Routine inspections to detect for signs of erosion progressing (cracking, seepage)	Potentially Credible	

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Prepared by	Alex Campbell (ATCW)
Date	30-Nov-2022

Assumptions
Unless otherwise stated, the failure mode is assumed to occur at the critical stage, which is the end of filling of the TSF
These are failure modes that would result in a catastrophic failure of the embankment, resulting in released material

Reference	Failure Mode	Failure Progression	Controls			Failure Mode Credibility	Justification
			Design	Construction	Operation		
O1	Embankment Overtopping due to loss of spillway capacity	Spillway capacity reduced or completely removed by blockage (such as debris, pipelines or storage of materials) Storm event exceeds reduced spillway capacity/remaining freeboard Flow of stormwater and mobilised tailings over the embankment crest causes deep scouring, which progressing an embankment failure.	Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway Tailings pipeline and decant pipeline aligned to not cross spillway	Ensure construction debris is cleared from spillway once embankment is complete.	Routine inspections of the TSF to check for and remove potential blockages	Potentially Credible	
O2	Embankment Overtopping due to crest scour from concentrated rainfall runoff	Extended rainfall causes concentrated rainfall runoff and scours the embankment crest to below spillway level, reducing available freeboard. Storm event occurs in excess of this reduced freeboard Flow of stormwater and mobilised tailings through the initial scour exacerbate scouring of the embankment crest, which progressing an embankment failure	Embankments designed with a uniform cross fall to allow rainfall to freely drain into the TSF without concentration Embankments designed with additional road base material to prolong potential erosio	Full time QA/QC provided to ensure design specifications are met, particularly around compaction of the road base.	Routine inspections of TSF required to identify any potential scouring/concentrated runoff, and to be readily repaired	Not Credible	Significant concentrated rainfall scour is highly unlikely given uniform crest shape and drainage. Potential scour would be noticed early and repaired
O3	Embankment Overtopping due to crest scour from pipeline burst	Pipeline (either tailings or return water) bursts, causing material to flow uncontrollably on embankment crest at high velocity. Pipeline burst is not noticed by operations. Flow of material scours embankment crest to below spillway level, reducing available freeboard. Storm event occurs in excess of this reduced freeboard Flow of stormwater and mobilised tailings through the initial scour exacerbate scouring of the embankment crest, which progressing an embankment failure	Embankments designed with additional road base material to prolong potential erosion. Pipelines placed on upstream side of embankment crest to direct potential burst flows towards the interior of the facility.	Full time QA/QC provided to ensure design specifications are met, particularly around compaction of the road base.	Continual (i.e. automated) monitoring of pipeline flow pressures to monitor for deficiencies Routine inspections of TSF and Pipelines for signs of degradation	Potentially Credible	
O4	Embankment Overtopping due to poor deposition management - Spillway Blockage	Poor deposition management causes tailings to build-up in spillway, reducing or completely removing spillway capacity Storm event exceed reduced spillway capacity/remaining freeboard Flow of stormwater and mobilised tailings over the embankment crest causes deep scouring, which progressing an embankment failure.	TSF designed with a maximum storage level which will be below the Spillway Invert Level which shall not be exceeded Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway		Routine inspections of the TSF to check for tailings near the spillway invert, and move deposition away from the spillway if needed Tailings to be primarily spigotted at opposite end of facility from spillway	Potentially Credible	
O5	Embankment Overtopping due to poor deposition management - Decant Blockage	Poor deposition management causes tailings to push water away from decant shaft AND/OR tailings inundate inclined decant shaft and remove capacity for removal of water Water cannot be removed from the facility, and builds up to spillway level under normal conditions Storm event exceeds spillway capacity/remaining freeboard Flow of stormwater and mobilised tailings over the embankment crest causes deep scouring, which progressing an embankment failure.	Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway TSF Designed with limited number of deposition points at opposite end of the decant structure to enable pond to be maintained around decant structure Decant pipeline wrapped in geofab to reduce risk of decant blockage due to tailings ingress		Routine inspections of the TSF to monitor deposition, pond extents, decant blockages, and implement remediate actions if needed	Potentially Credible	
O6	Embankment Overtopping due to poor deposition management - Over deposition	Poor deposition management results in tailings being deposited above the maximum storage level, reducing available freeboard Storm event consumes reduced freeboard and exceeds spillway capacity Flow of stormwater and mobilised tailings over the embankment crest causes deep scouring, which progressing an embankment failure.	TSF designed with a maximum storage level which will be at or below the Spillway Invert Level which shall not be exceeded Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway		Routine surveillance to ensure maximum storage level is not exceeded Tailings to be spigoted in select locations making over deposition difficult to achieve	Potentially Credible	
O7	Embankment Overtopping due to build-up of excess tailings bleed water	Increased significant amounts of bleed water occur due to a combination of change in slurry composition and loss of decant removal capability Increased pooling it not noticed, and stand-by pumps are not mobilised Storm event consumes reduced freeboard and exceeds spillway capacity Flow of stormwater and mobilised tailings over the embankment crest causes deep scouring, which progressing an embankment failure.	Stochastic water balance to include variations on tailings composition, and potential impacts on pond levels Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway		Procurement of an additional stand-by pump for emergency removal of water from the TSF Upgrade to tailings delivery system to allow for full utilisation of High Rate Thickener Routine Inspections and monitoring of pond level/extents Routine monitoring of tailings slurry composition to ensure bleed rate is within design limits	Potentially Credible	
O8	Embankment Overtopping due to higher than expected operating pond levels	Water balance assessment not suitable for facility, resulting in insufficient surface water removal (i.e., insufficient pumping capacity) and rising operating pond Increased pooling it not noticed, and stand-by pumps are not mobilised Storm event consumes reduced freeboard and exceeds spillway capacity Flow of stormwater and mobilised tailings over the embankment crest causes deep scouring, which progressing an embankment failure.	Stochastic water balance of 1000 scenarios of climate data to allow for detailed modelling of predicted maximum pond levels Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway		Pumping capacity via decant system (and stand-by pumps) provided to aid in removal of stormwater Routine Inspections and monitoring of pond level/extents	Potentially Credible	
O9	Embankment Overtopping due to single/multiple large storms that exhaust freeboard and exceeds spillway capacity	Single or multiple storm events larger than design events consume freeboard and exceed spillway capacity Flow of stormwater and mobilised tailings over the embankment crest causes deep scouring, which progressing an embankment failure.	Freeboard above Maximum Operating Pond to Spillway Invert Level provided Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway		Pumping capacity via decant system (and stand-by pumps) provided to aid in removal of stormwater	Potentially Credible	
O10	Embankment Overtopping due to loss of spillway capacity from seismic induced crest settlement	Seismic event causes crest deformation in excess of spillway depth, creating a low spot below spillway level, effectively removing spillway capacity Storm event occurs consuming remaining freeboard, and flows through deformation low spot Flow of stormwater and mobilised tailings causes scouring of the embankment crest, which progressing an embankment failure.	Site-specific Seismic Hazard Assessment undertaken to understand maximum seismic levels Embankment constructed by downstream method with primarily compacted rockfill, and are expected to experience only minimum deformation under SEE loading	Full time QA/QC provided to ensure design specifications are met to minimise potential for crest settlement	Inspections following seismic event to be undertaken to assess for potential crest settlement.	Not Credible	Deformation for 1:10,000 AEP Seismic Event indicate maximum 1.5% of total height to bedrock as deformation. At 15m, this is 225mm, which is less that the depth of the spillway and beach freeboard.
O11	Embankment Overtopping due to reduced spillway capacity from seismic induced crest settlement	Seismic event causes crest deformation, creating a low spot above spillway invert level, reducing spillway capacity Storm event occurs consuming remaining freeboard, and flows through the reduced spillway. Water eventually flows through the deformation low spot Flow of stormwater and mobilised tailings causes scouring of the embankment crest, which progressing an embankment failure.	As above, plus Spillway designed to safely pass the Probable Maximum Flood (PMF) with the pond starting at Spillway Invert Level, with additional freeboard provided in the spillway	Full time QA/QC provided to ensure design specifications are met to minimise potential for crest settlement	Inspections following seismic event to be undertaken to assess for potential crest settlement.	Potentially Credible	
O12	Embankment Overtopping due to scour from failure of Spillway Erosion Protection Rip-Rap	Large storm event occurs, causing rapid flow down the spillway. Erosion protection rip-rap is not suitable, and is quickly washed away/displaced. Flow of water over the now exposed embankment causes scouring down to tailings level. Stormwater and mobilised tailings then accelerate the scouring, which progressing an embankment failure.	Erosion Protection Rip-Rap designed to meet the maximum expected velocities down slope, and installed into the embankment Geotextile underlay provided for additional erosion protection	Full time QA/QC provided to ensure design specifications for rip-rap rockfill material selection and placement.	Routine inspections of Spillway area for signs of degradation of Rip-Rap	Potentially Credible	

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Assumptions

Unless otherwise stated, the failure mode is assumed to occur at the critical stage, which is the end of filling of the TSF

These are failure modes that would result in a catastrophic failure of the embankment, resulting in released material

Reference	Failure Mode	Failure Progression	Controls			Failure Mode Credibility	Justification
			Design	Construction	Operation		
S1	Embankment Instability due to incorrect material characterisation - Embankment fill materials	Adopted embankment material strength parameters too high, and unsuitable embankment geometry is designed. Under static loading embankment, experiences a slip failure and deformation equal to the remaining freeboard, resulting in tailings and water flow over the embankment crest, progressing an embankment failure.	Conservative embankment material strength parameters based on laboratory testing adopted. Embankments designed with suitable batter slope to provide sufficient FoS against static failure of embankment materials only.	Full time QA/QC provided to ensure embankment design specifications are met	Routine inspections to monitor for signs of movement	Potentially Credible	
S2	Embankment Instability due to incorrect material characterisation - Foundation materials	Adopted foundation material strength parameters too high, and unsuitable foundation conditions are adopted Under static loading embankment, experiences a slip failure and deformation equal to the remaining freeboard, resulting in tailings and water flow over the embankment crest, progressing an embankment failure.	Conservative foundation material strength parameters based on laboratory testing adopted Embankments designed to provide sufficient FoS against static failure through foundations	Full time QA/QC provided to ensure foundation preparation design specifications are met	Routine inspections to monitor for signs of movement	Potentially Credible	
S3	Embankment Instability due to high phreatic surface	Storm event raises water level to spillway invert level. Significantly large hole in Geosynthetic Liner present Water is not removed, and creates a high phreatic surface through the embankment section Under static loading embankment with a high phreatic surface, experiences a slip failure and deformation equal to the remaining freeboard, resulting in tailings and water flow over the embankment crest, progressing an embankm failure.	Embankment stability assessed at maximum phreatic surface level Design of facility with Geosynthetic Liner to stop phreatic surface development Majority of embankment designed as rockfill to freely drain water and prevent build-up of excess pore pressures	Full time QA/QC provided to ensure embankment design specifications are met to prevent un-expected low permeability zones in downstream rockfill	Pumping capacity via decant system (and stand-by pumps) provided to aid in removal of stormwater	Potentially Credible	
S4	Embankment Instability due to inadequately constructed embankments	Unsuitable material used in construction of the embankments over a continuous region of the embankment section. This unsuitable material is considerably weaker than the design materials Under static loading embankment, experiences a slip failure and deformation equal to the remaining freeboard, resulting in tailings and water flow over the embankment crest, progressing an embankment failure.		Full time QA/QC provided to ensure suitable embankment materials are used in construction, and placed/compacted in accordance with the specifications to meet the design intent.	Routine inspections to monitor for signs of movement	Potentially Credible	
S5	Embankment Instability due to inadequately prepared foundations	Inadequate foundation preparation undertaken during construction, resulting in a continuous weak region across the embankment section. Under static loading embankment, experiences a slip failure and deformation equal to the remaining freeboard, resulting in tailings and water flow over the embankment crest, progressing an embankment failure.		Full time QA/QC provided to ensure suitable foundation preparation is undertaken (stripping of unsuitable material, compaction of foundations) in accordance with the specifications to meet the design intent	Routine inspections to monitor for signs of movement	Potentially Credible	
S6	Embankment Instability due to seismic deformation	Significantly large seismic event occurs, causing deformation in excess of the freeboard to operational pond level Tailings and operational water flow over the embankment crest, progressing an embankment failure.	Site-specific Seismic Hazard Assessment undertaken to understand maximum seismic levels Embankment constructed by downstream method with primarily compacted rockfill, and are expected to experience only minimum deformation under SEE loading.		Inspections following seismic event to be undertaken to assess for signs of movement	Not Credible	Deformation for this scenario is greater than the deformation required for Case O10, so is determined non-credible
C1	Embankment erosion failure due to cumulative static settlement and seismic deformation	Poor construction controls implemented, resulting in loose uncompacted downstream rockfill. Static settlement of the embankment crest occurs over the life of the facility, which is un-noticed and not remedied mine personnel. Seiemic event causes significant crest deformation and liquefies tailings deposit. Cumualtive static settlement and crest deformation is in excess of the beach freeboard. Tailings and operational water flow though low spot in the embankment, causing furhter scouring of the embankm crest, which progresses an embankment failure.	TSF designed with significant beach freeboard Downstream rockfill specified to be compacted to minimise settlement Site-specific Seismic Hazard Assessment undertaken to understand maximum seismic levels Embankment constructed by downstream method with primarily compacted rockfill, and are expected to experience only minimum deformation under SEE loading.	Full time QA/QC provided to ensure embankment design specifications are met	Routine inspections and aerial survey to identify crest movement, with works to be remedied.	Potentially Credible	

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APPENDIX B – GEOTECHNICAL PIPING ASSESSMENT

Crack initiation by Differential Settlement as a result of cross section settlement from poorly compacted shoulder (IM4)				Notes
Assess likelihood of Transverse Cracking				
Based on IM4, Table A2.7				
Relative Importance Factor	**RF**	**LF**	**Likelihood Factor**	
Embankment Zoning & Compaction	3	2	Neutral	"Core" slopes flatter than 45 degrees, and core well compacted
Core Width Geometry	2	4	Much More Likely	W/H = 3m / 15 m
Height of Embankment	1	1	Less Likely	Height up to 15m
RF*LF	15			
Based on Table A2.8				
RF*LF	15			
	Below PoR		Above PoR	
Probability (Transverse Cracking, Pc)	3.00E-04		3.00E-03	
Assess likelihood erosion will intiate within the crack				
Based on Table A2.24				
RF*LF	15			
Maximum likely crack width at dam crest (Cmax)	75	mm		
Depth (d1)	0.5	m		
Depth (d2)	1.3	m		Increased by 0.5m to account for recently deposited low density tailings
Based on Table A2.25				
Likely Crack Width at Upper Reservoir level (Cd1)	40	mm		Crack widths taken as 40mm, as both values fall below the data presented in Table A2.25
Likely Crack Width at Lower Reservoir level (Cd2)	40	mm		
Estimating the average gradient of flow through the crack				
Width of "Core" that cracking must initiate across	3	m		Taken as entirety of Zone 1B (3.0m)
Average Flow Gradient	0.27	m/m		
Zone 1B material is **dispersive**. Material as a whole generally classifed as CL-CH material				
Based on Table A2.35				
Likely crack width within core being considered	40	mm		
Average Flow Gradient	0.27	m/m		

Likely Crack Width (mm)	Probability
25	1
50	1

Probability(Erosion initiation in a crack, Pic)	1.00E+00		
Assess overall probability	Below PoR	Above PoR	
Combined Probability	3.00E-04	3.00E-03	

Initiation through desiccation cracks in the crest of dam (IM7)				Notes
Assess likelihood of Transverse Cracking				
Based on Table A2.11				
Relative Importance Factor	**RF**	**LF**	**Likelihood Factor**	
Crest Zoning and Surface Layer over Core	3	2	Neutral	minimum 100mm over crest, plus BGM liner at front of Zone 1B.
Climate	2	1	Less Likely	Temperate climate
Plasticity of Core Material	1	3	More Likely	Medium to high plasticity material
RF*LF	11			
Based on Table A2.12				
RF*LF	11			
	Below PoR		Above PoR	
Probability (Transverse Cracking, Pc)	1.00E-02		1.00E-02	
Screening of maximum dessication crack depth				
Although the road base material is no sufficient to fully stop dessication cracking, the arrangement of the BGM liner prevents exposure of the dessication crack to the water/tailings, and is consided not possible				
Access overall probability				
	Below PoR		Above PoR	
Combined Probability	0.00E+00		0.00E+00	Deemed non-credible

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Crack initiation by Siesmic Events (IM13)				Notes
Estimate Earthquake Annual Exceedance Probability				
Based on IM13 Figure A2.9 (Earth and Rockfill Dams)				
Highest probability will be smaller earthquake causing minimum damage to initiate a crack.				
Taken as Boundary between Class 0 and Class 1 Damage				
M = 6.2, a = 023g				
Annual Exceedance Probability	1: 5,000			
Probability (Seismic Event)	2.00E-04			
Assess likelihood of Transverse Cracking				
Based on embankment section and profile, transverse cracking is only likely due to settlement of the foundation materials beneath the embankment				
Based on IM5, Table A2.9				
Relative Importance Factor	**RF**	**LF**	**Likelihood Factor**	
Foundation geology and geometry	3	2	Neutral	Shallow soils
Side slopes of compressible zones	2	1	Less Likely	
Height of Embankment	1	1	Less Likely	Less than 15m
RF*LF	9			
Based on Table A2.39				
Probability (Seismic Transverse Cracking, Pc)	1.00E-02			
Maximum Likely Crack Width (mm)	20			
Assess likelihood erosion will intiate within the crack				
Based on Table A2.24				
RF*LF	9			
Maximum likely crack width at dam crest (Cmax)	20	mm		
Depth (d1)	0.5	m		
Depth (d2)	1.3	m		Increased by 0.5m to account for recently deposited low density tailings
Based on Table A2.25				
Likely Crack Width at Upper Reservoir level (Cd1)	2	mm		
Likely Crack Width at Lower Reservoir level (Cd2)	1	mm		
Estimating the average gradient of flow through the crack				
Width of "Core" that cracking must initiate across	3	m		Taken as entirety of Zone 1B (3.0m)
Average Flow Gradient	0.27	m/m		
Zone 1B material is **dispersive**. Material as a whole generally classifed as CL-CH material				
Based on Table A2.35				
Likely crack width within core being considered	1.5	mm		
Average Flow Gradient	0.27	m/m		

Likely Crack Width (mm)	Probability
1	0.1
2	0.3

Probability(Erosion initiation in a crack, Pic)	2.00E-01	
Assess overall probability		
Combined Probability	4.00E-07	

Initiation of Erosion by poorly compacted/High permeability zone within the core (IM14)				Notes
Assess likelihood of poorly compacted/high permeability zone				
Based on IM14, Table A3.1 (Cohesive Soils)				
Relative Importance Factor	**RF**	**LF**	**Likelihood Factor**	
Compactive Effort	3	1	Less Likely	Will be compacted to minimum 98% SMDD, with field trials to confirm compaction
Borrow Area/Site Supervision	2	2	Neutral	Uniform soils from cut-to-fill operations with good site supervision.
Core Geometry (W/H)	1	4	Much More Likely	W/H = 3m / 15 m
RF*LF	11			
Based on Table A3.3				
Estimate for above Pool of Record (PoR)				
RF*LF	11			
	Below PoR		Above PoR	
Probability (Poorly Compacted/High Permeability Zone Pp)	1.00E-04		1.00E-03	
Based on Table A3.24				
Observed Seepage	None	-		
Adjustment Factor	1			
Based on Section A3.6.1				
Check if soil is cohesive or not				
Zone 1B is cohesive with a Plasticity index > 7, therefore, probability of backwards erosion and suffision are negligible.				
Assess overall probability	Below PoR		Above PoR	
Combined Probability	1.00E-04		1.00E-03	

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Scenario 2 or Scenario 3 Determination

Check if downstream zones are capable of holding a crack or pipe

Zone 3A

No laboratory testing undertaken. Rockfill: d_{max} = 300mm, Some fines (10-20%)
Material is well compacted.
Plastic Fines

Zone 3B

No laboratory testing undertaken. Rockfill: d_{max} = 400mm, Minimal fines (< 10%)
Material is well compacted.
Plastic Fines

Based on Section A8.1.1, Zone 3A and 3B are capable of holding a crack or pipe

Scenario 2 - Downstream, cohesive material

Based on Table A8.1

Cracking due to differential settlement

Most likely method of cracking is differential settlement of downstream shoulder. Mechanism causing cracking would cause a crack in the DS shell

Zone 3A/3B well compacted, but not likely to hold crack and may collapse	0.5 - 0.9
Probability (Continuation, initiation through cracking)	8.00E-01

Crasking due to high permeability zone

Leak unlikely to find an exit through the shoulder, given wide downstream shoulder, well compacted, and different lift thicknesses used	0.01 - 0.1
Probability (Continuation, initiation from high perm. zone)	5.00E-02

ADVERTISED PLAN

Progression Through Embankment				
Assess likelihood of compacted soil holding a roof				
Based on Table 11-1				
Material Zone	**Soil Classification**	**% Fines**	**Plasticity**	**Likelihood to support a roof**
Zone 1B	CH - CL	>50%	32 - 15	1.00E+00
Zone 3A	SC, GC	10% to 20%	Plastic Fines	1.00E+00
Zone 3B	SC-SW with Gravel	< 10%	Plastic Fines	5.00E-01
		Probability (All soils will hold roof)	5.00E-01	
Assess likelihood of Crack Filling action				
Based on Table 11-2				
Embankment is a zoned earth and rockfill dam, with a sloping "core" of Zone 1A on the upstream face. Zone 1B material is a dispersive high plasticity clay Zone 3A and 3B are well graded rockfill Downstream transition zone is present Crack filling action is negligable				
		Probability (No crack filling action occuring)	1.00E-01	
Assess likelihood of flow limitation				
Based on Table 11.3				
This scenario is tear in BGM, therefore "upstream" material is water with tailings fines				
		Probability (No upstream flow limitation)	1.00E+00	
		Probability (Overall continuation through the embankment)	5.00E-02	

ADVERTISED PLAN

Through embankment into foundations - Into soils

Assess likelihood of compacted soil holding a roof

Based on Table 11-1

Material	Soil Classification	% Fines	Plasticity	Likelihood to support a roof
Zone 1B	CH - CL	>50%	32 - 15	1.00E+00
Zone 3A	SC, GC	10% to 20%	Plastic Fines	1.00E+00
Zone 3B	SC-SW with Gravel	<5%	Plastic Fines	5.00E-01
Foundation	CL - CH	>50%		1.00E+00

Probability (All soils will hold roof) 5.00E-01

Assess likelihood of Crack Filling action

Based on Table 11.2

Embankment is a zoned earth and rockfill dam, with a sloping "core" of Zone 1A on the upstream face. Zone 1A material is a dispersive high plasticity clay
Zone 3A and 3B are well graded rockfill
Downstream transition zone is present
Crack filling action is negligable

Probability (No crack filling action occuring) 1.00E-01

Assess likelihood of flow limitation

Based on Table 11.3

This scenario is tear in BGM, therefore "upstream" materail is water with tailings fines

Probability (No upstream flow limitation) 1.00E+00

Probability (Overall continuation through the embankment) 5.00E-02

Detection, Intervention and Repair

Likely progression time	Notes

Assess likelihood of soil to support a pipe roof

Based on all previous assessments, all embankment soils are highly likely to be able to support a piping roof

Assess rate of erosion of the core

Based on Table 12-2

Material Zone	Soil Classification	Erosion Rate Index (IHET)	Time for Erosion
Zone 1B	CH (LL<65)	4	Rapid to Medium (R-M)

Assess likelihood of flow limitation

Based on previous assessments, upstream zone is >15% fines, highly likely to support a roof and will not limit flow

Assess possible breach time

Based on Table 12-3

Zone 3A and 3B are coarse grained rockfill of significant width. Breach time likely to be slow.

Assess total progression time

Based on Table 12-1

Ability to support a pipe roof	Yes
Rate of Upstream core erosion	R-M
Upstream Flow limiter	No
Breach Time	S

Approximate Likely Time for Breach Progression Medium

2 - 7 Days

Detection

Leak detection				Notes
Assess likelihood of not observing a concentrated leak				
Based on Table 12.5				
Relative Importance Factor	**RF**	**LF**	**Likelihood Factor**	
Can the leak be observed at the downstream toe	3	1	Less Likely	Foundation compacted clay, so leaks will emerge at toe, and mown grasses
Dam zoning which affects where a leak will emerge	2	3	More likely	Zoned earthfill and rockfill dam
Seepage instrumentation	1	3	More likely	Seepage partly collected by toe drains
RF*LF	12			
Based on Table 12.6				
RF*LF	12			
Probability (Leak not observable, nol)	2.00E-01			
Assess likelihood that, given leak is observable, it is not detected				
Probability (Leak is observable, nol')	8.00E-01			
Determine probability leak is not detected				
Based on Table 12.7				
Inspection frequency	Daily visual inspection			Some form of daily inspection done
	Near Public			Adjacent to Bradleys Lane
Approximate Likely Time for Breach Progression	2 - 7 days			
Probability (not detected, nd)	5.00E-02			

Intervention and Repair		Notes
Assess likelihood that, given leak is detected, intervention and repair is not sucessful		
Approximate Likely Time for Breach Progression is 2 - 7 days. In some cases, it will be practical to intervene sucessfully in this time. Given it is a tailings dam, and requires saturated tailings/standing water to be present against the embankment upstream face for a significant period of time, there are very straight forward methods to intervene (implement emergency pumping, cease deposition)		
Probability (Leak detected, interventon not sucessful, Pui)	2.00E-01	

Overall Detection and Intervention		Notes
Probability Intervention fails = Probability not observing leak because it is not observable, **P(nol)** + Probability leak is observable but not detected, **P(nol')*P(nd)** + Probability leak is observable and detected, but invervention fails, **P(nol')*P(nd')*P(ui)**		
Probability (Intervention fails)	3.92E-01	

Overall Breach		
Initial Screening		**Notes**
Breach Mechanisms	Gross enlargement of pipe, followed by collapse Downstream slope instability Sloughing or Unravelling of downstream toe Sinkhole development	
Initial Screening of Breach Mechanisms		
Based on Table 13.1		
Dam Zoning type	Zoned earthfill & Rockfill	
Breach Mechanisms	**Possible Mechanism?**	
Gross Enlargement	N	
Slope Instability	Y	
Sloughing or Unravelling	Y	
Sinkhold Development	Y	

Instability of the downstream slope/foundations				Notes
Internal erosion must increase the pore pressures so that the embankment factor of safety falls below 1.0, resuling in sliding deformations and loss of freeboard to the water level				
Assess likelihood of slope instability due to increased seepage flows				
Based on Table 13.4				
Relative Importance Factor	**RF**	**LF**	**Likelihood Factor**	
Internal drainage measures in dam	3	1	Less Likely	Free draining downstream rockfill
Downstream Slope	2	1	Less Likely	4:1 (H:V) batter slope
Downstream shell materials	1	1	Less Likely	Coarse grained rockfill
RF*LF	6			
Based on Table 13.5				
RF*LF	6			
For Excessive erosion				
Probability (Slope instability due to increased seepage, si-i)	1.00E-04			
Assess likelihood of loss of freeboard given instability				
Based on Table 13.10				
Freeboard compared to dam height	3	1	Less Likely	1.1m from crest to maximum operating pond. 1.1m / 15m > 7%
Presence of strain weakening soils	2	1	Less Likely	Sandy Clays, and rockfill
Crest Width	1	2	Neutral	6m wide crest
RF*LF	7			
Based on Table 13.11				
RF*LF	7			
Probability (Breach by loss of freeboard, si-if)	3.00E-03			
Assess likelihood of breach due to slope instability and loss of freeboard				
Probability (Breach by slope instability and loss of freeboard)	3.00E-07			

Unravelling of the downstream slope				Notes
For sloughing to occur, downstream face would have to be relatively steep, and the shoulder material a cohesionless soil. This process would have to be allowed to continue until it eroded the entirity of the crest				
Assess likelihood of breach by unravellimg				
Rockfill downstream zone				
Based on Table 13.15				
Material in downstream zone	3	1	Less Likely	Free draining downstream rockfill
Downstream Slope of Embankment	2	1	Less Likely	4:1 (H:V) batter slope
Freeboard at time of incident	1	3	More Likely	1.1m from crest to maximum operating pond
RF*LF	8			
Based on Table 13.16				
RF*LF	8			
For Excessive erosion				
Probability (Breach by unravelling)	4.00E-03			

Sinkhole/Crest Settlement Development				Notes
Assess likelihood of sinkhole/crest settlement development from internal erosion				
Based on Table 13.18				
Internal erosion into the embankments and/or into the foundations				
Probability (Sinkhole/crest settlement development, s-f)	6.00E-01			
Assess likelihood of loss of freeboard due to sinkhole/crest settlement				
Based on Table 13.19				
Freeboard at time of incident	3	3	More Likely	1.1m from crest to maximum operating pond
Crest width	2	2	Neutral	6m wide crest
Material in the core of the embankment	1	1	Less Likely	High plasticity clays, well compacted
RF*LF	14			
Based on Table 13.20				
RF*LF	14			
For Excessive erosion				
Probability (Breach by unravelling)	4.00E-03			

Overall Probability of a Breach		Notes
Probability (Any breach occuring	7.98E-03	

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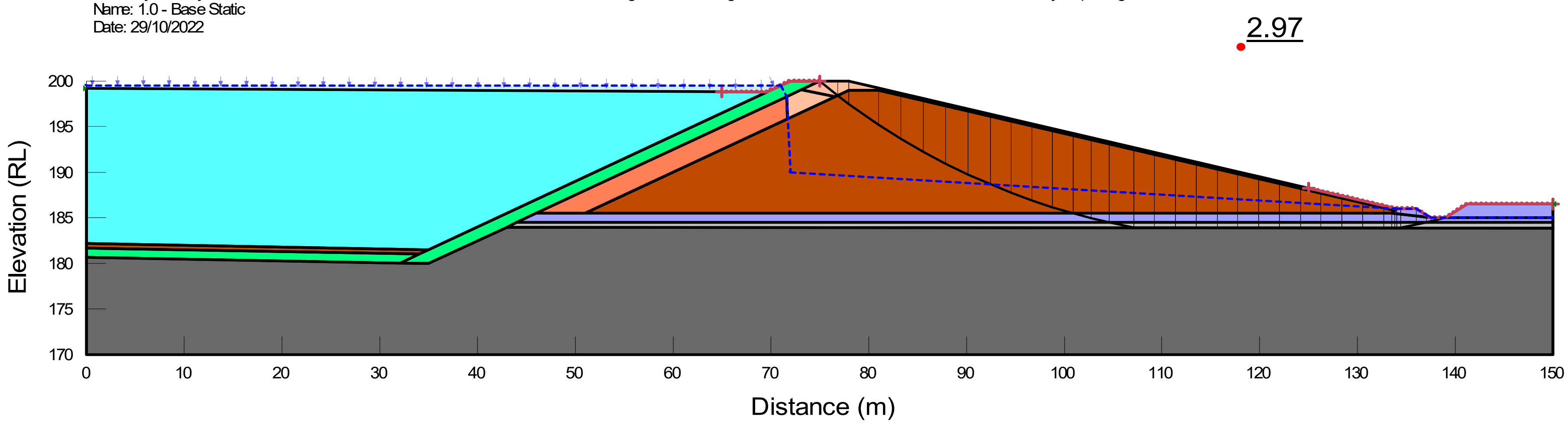
APPENDIX C – EMBANKMENT INSTABILITY ASSESSMENT

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 1.0 - Base Static
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Strength Function	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (Undrained)	SHANSEP	19					20	0.28	1
	EW Siltstone Foundation	Mohr-Coulomb	22		0	36	0			1
	Foundation Clay (Drained)	Mohr-Coulomb	20.5		38	20	0			1
	Fresh Tailings	Spatial Mohr-Coulomb	16		0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)								1
	Rockfill (Zone 3A) - Saturated (Dr/Undr)	Mohr-Coulomb	22		15	23	0			1
	Rockfill (Zone 3A) - Unsaturated (Dr/Undr)	Mohr-Coulomb	20		30	28	0			1
	Rockfill (Zone 3B) (Dr/Undr)	Shear/Normal Fn.	22	Leps Lower Bound			0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

Costerfiled Gold Mine

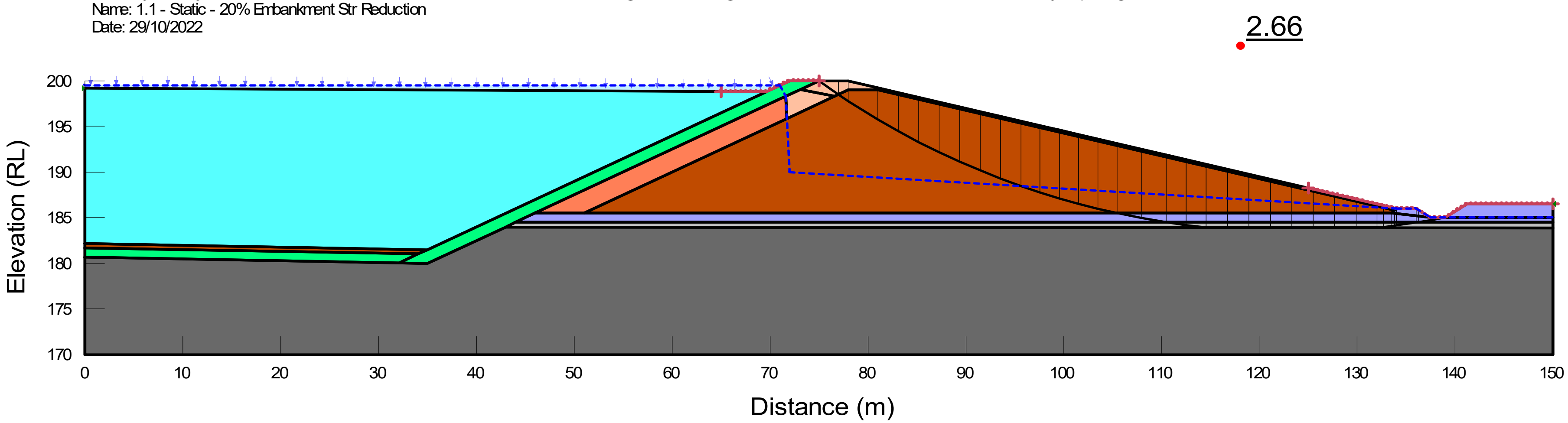
Brunswick West Tailings Storage Facility

Embankment Instability Assessment - Long Term Static Stability
Base Case

Date: 30/11/2022 **Job No:** 109014.15

FIGURE C1

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 1.1 - Static - 20% Embankment Str Reduction
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Strength Function	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (80%)	SHANSEP	19					16	0.224	1
	EW Siltstone Foundation	Mohr-Coulomb	22		0	36	0			1
	Foundation Clay (Drained)	Mohr-Coulomb	20.5		38	20	0			1
	Fresh Tailings	Spatial Mohr-Coulomb	16		0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)								1
	Rockfill (Zone 3A) - Saturated (Dr/Undr) (80%)	Mohr-Coulomb	22		12	18.4	0			1
	Rockfill (Zone 3A) - Unsaturated (Dr/Undr) (80%)	Mohr-Coulomb	20		24	22.4	0			1
	Rockfill (Zone 3B) (Dr/Undr) (80%)	Shear/Normal Fn.	22	Leps Lower Bound (80%)			0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

Costerfiled Gold Mine

Brunswick West Tailings Storage Facility

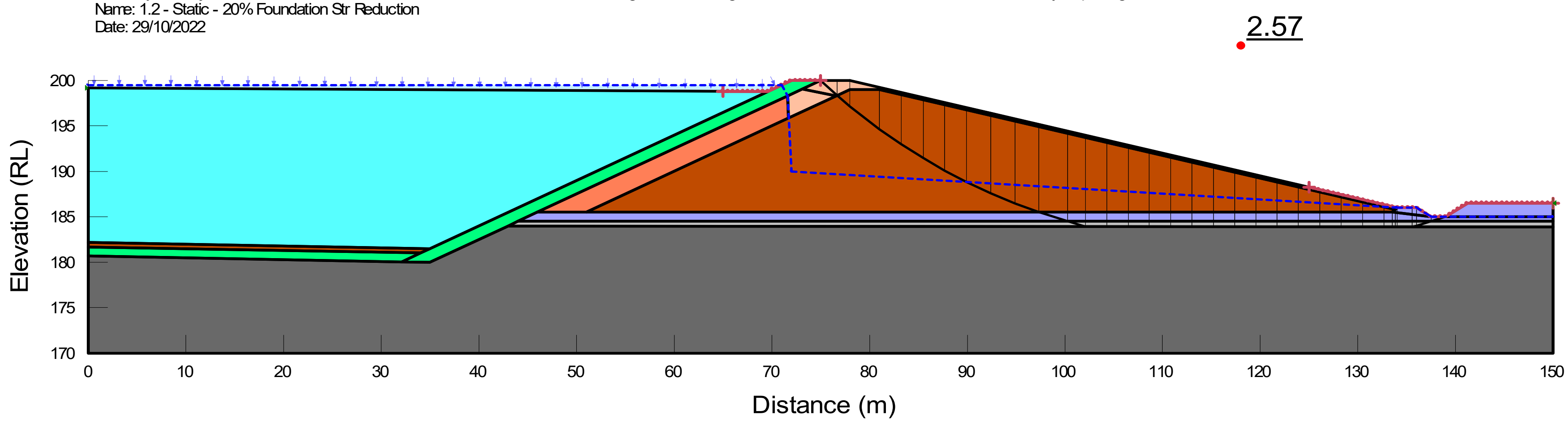
Embankment Instability Assessment - Long Term Static Stability
Incorrect Material Characterisation - Embankment Fill Materials

Date: 30/11/2022 **Job No:** 109014.15

FIGURE C2

K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\Appendix C - Embankment Instability Assessment C2

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 1.2 - Static - 20% Foundation Str Reduction
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Strength Function	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (Undrained)	SHANSEP	19					20	0.28	1
	EW Siltstone Foundation (80%)	Mohr-Coulomb	22		0	28.8	0			1
	Foundation Clay (Drained) (80%)	Mohr-Coulomb	20.5		30.4	16	0			1
	Fresh Tailings	Spatial Mohr-Coulomb	16		0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)								1
	Rockfill (Zone 3A) - Saturated (Dr/Undr)	Mohr-Coulomb	22		15	23	0			1
	Rockfill (Zone 3A) - Unsaturated (Dr/Undr)	Mohr-Coulomb	20		30	28	0			1
	Rockfill (Zone 3B) (Dr/Undr)	Shear/Normal Fn.	22	Leps Lower Bound			0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

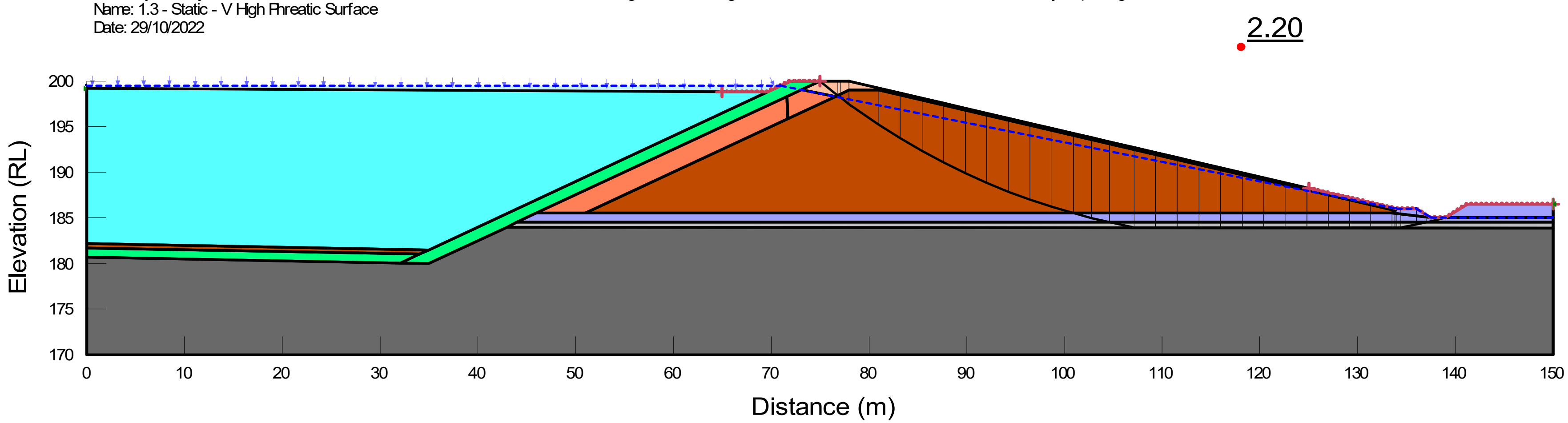
Costerfiled Gold Mine

Brunswick West Tailings Storage Facility

Embankment Instability Assessment - Long Term Static Stability
Incorrect Material Characterisation - Foundation Materials

Date: 30/11/2022 **Job No:** 109014.15 **FIGURE C3**

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 1.3 - Static - V High Phreatic Surface
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Strength Function	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (Undrained)	SHANSEP	19					20	0.28	1
	EW Siltstone Foundation	Mohr-Coulomb	22		0	36	0			1
	Foundation Clay (Drained)	Mohr-Coulomb	20.5		38	20	0			1
	Fresh Tailings	Spatial Mohr-Coulomb	16		0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)								1
	Rockfill (Zone 3A) - Saturated (Dr/Undr)	Mohr-Coulomb	22		15	23	0			1
	Rockfill (Zone 3A) - Unsaturated (Dr/Undr)	Mohr-Coulomb	20		30	28	0			1
	Rockfill (Zone 3B) (Dr/Undr)	Shear/Normal Fn.	22	Leps Lower Bound			0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

Costerfiled Gold Mine

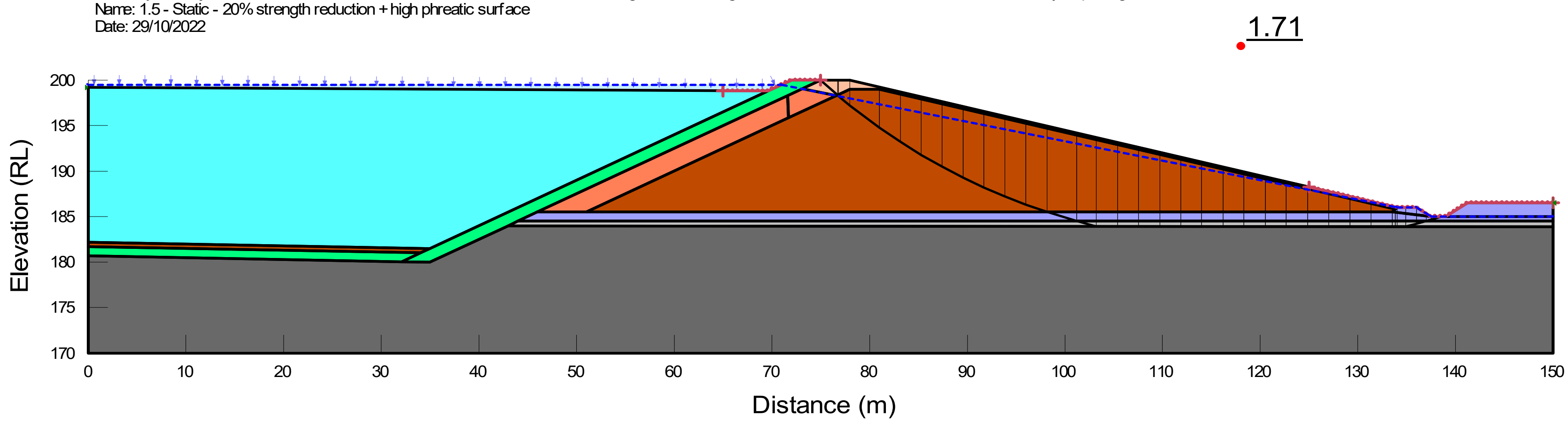
Brunswick West Tailings Storage Facility

Embankment Instability Assessment - Long Term Static Stability
High Phreatic Surface

Date: 30/11/2022 **Job No:** 109014.15

FIGURE C4

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 1.5 - Static - 20% strength reduction + high phreatic surface
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Strength Function	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (80%)	SHANSEP	19					16	0.224	1
	EW Siltstone Foundation (80%)	Mohr-Coulomb	22		0	28.8	0			1
	Foundation Clay (Drained) (80%)	Mohr-Coulomb	20.5		30.4	16	0			1
	Fresh Tailings	Spatial Mohr-Coulomb	16		0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)								1
	Rockfill (Zone 3A) - Saturated (Dr/Undr) (80%)	Mohr-Coulomb	22		12	18.4	0			1
	Rockfill (Zone 3A) - Unsaturated (Dr/Undr) (80%)	Mohr-Coulomb	20		24	22.4	0			1
	Rockfill (Zone 3B) (Dr/Undr) (80%)	Shear/Normal Fn.	22	Leps Lower Bound (80%)			0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

Costerfiled Gold Mine

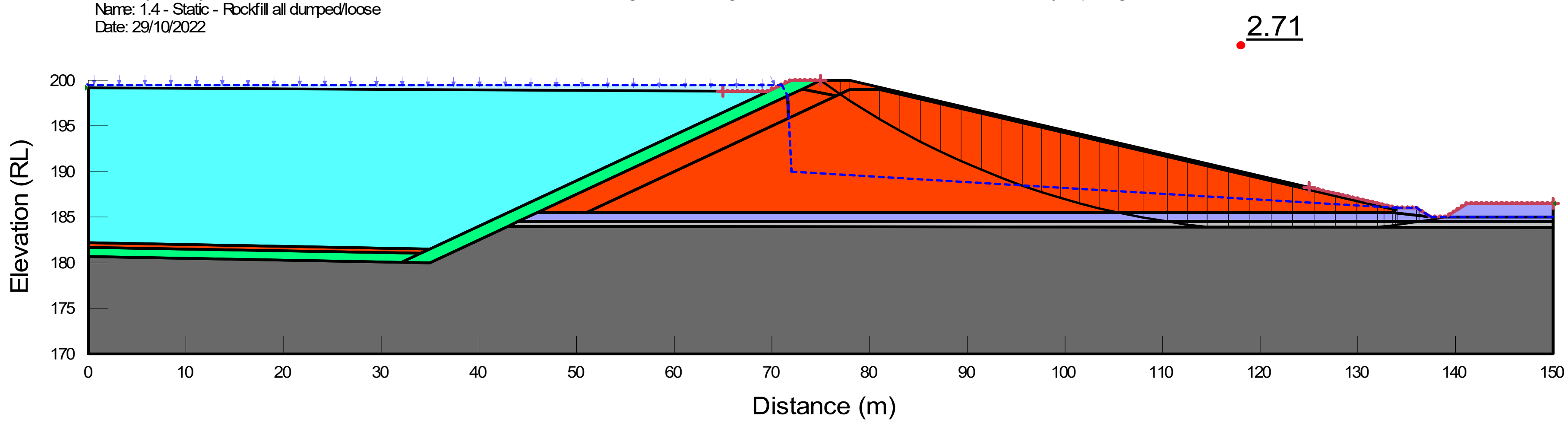
Brunswick West Tailings Storage Facility

Embankment Instability Assessment - Long Term Static Stability
Combined Failure of Design Elements

Date: 30/11/2022 **Job No:** 109014.15 **FIGURE C5**

K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\Appendix C - Embankment Instability Assessment C5

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 1.4 - Static - Rockfill all dumped/loose
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (Undrained)	SHANSEP	19				20	0.28	1
	EW Siltstone Foundation	Mohr-Coulomb	22	0	36	0			1
	Foundation Clay (Drained)	Mohr-Coulomb	20.5	38	20	0			1
	Fresh Tailings	Spatial Mohr-Coulomb	16	0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)							1
	Rockfill (Zone 3A/3B) Loose (Dr/Undr)	Mohr-Coulomb	22	0	35	0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

Costerfiled Gold Mine

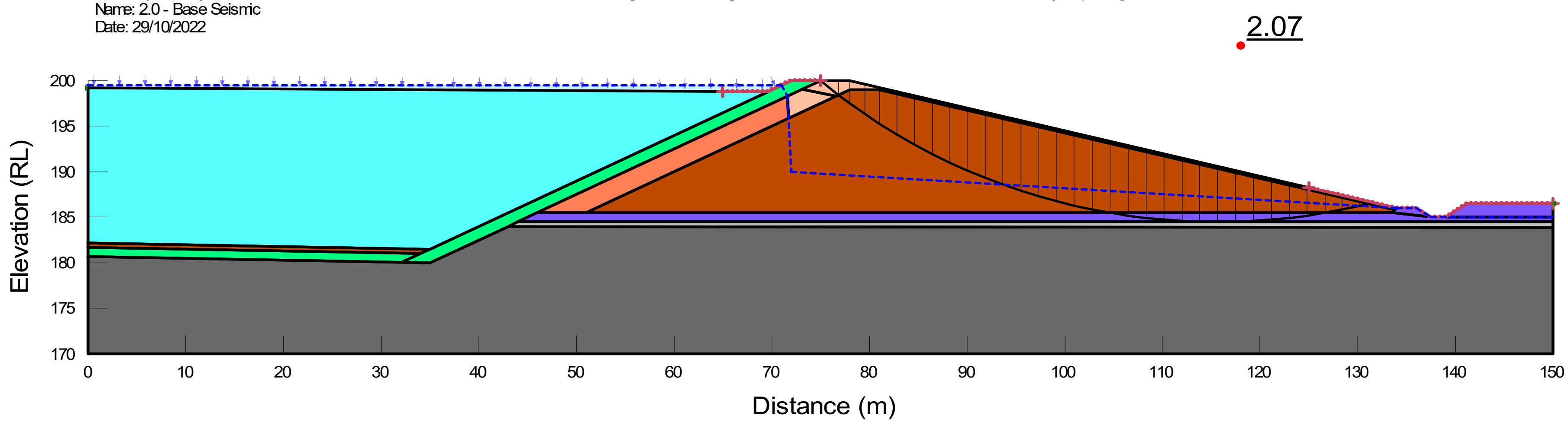
Brunswick West Tailings Storage Facility

Embankment Instability Assessment - Long Term Static Stability
Inadequately Constructed Embankments

Date: 30/11/2022 **Job No:** 109014.15

FIGURE C6

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 2.0 - Base Seismic
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Strength Function	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (80%)	SHANSEP	19					16	0.224	1
	EW Siltstone Foundation (80%)	Mohr-Coulomb	22		0	28.8	0			1
	Foundation Clay (Post Seis)	SHANSEP	20.5					16	0.28	1
	Fresh Tailings	Spatial Mohr-Coulomb	16		0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)								1
	Rockfill (Zone 3A) - Saturated (Dr/Undr) (80%)	Mohr-Coulomb	22		12	18.4	0			1
	Rockfill (Zone 3A) - Unsaturated (Dr/Undr) (80%)	Mohr-Coulomb	20		24	22.4	0			1
	Rockfill (Zone 3B) (Dr/Undr) (80%)	Shear/Normal Fn.	22	Leps Lower Bound (80%)			0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

Costerfiled Gold Mine

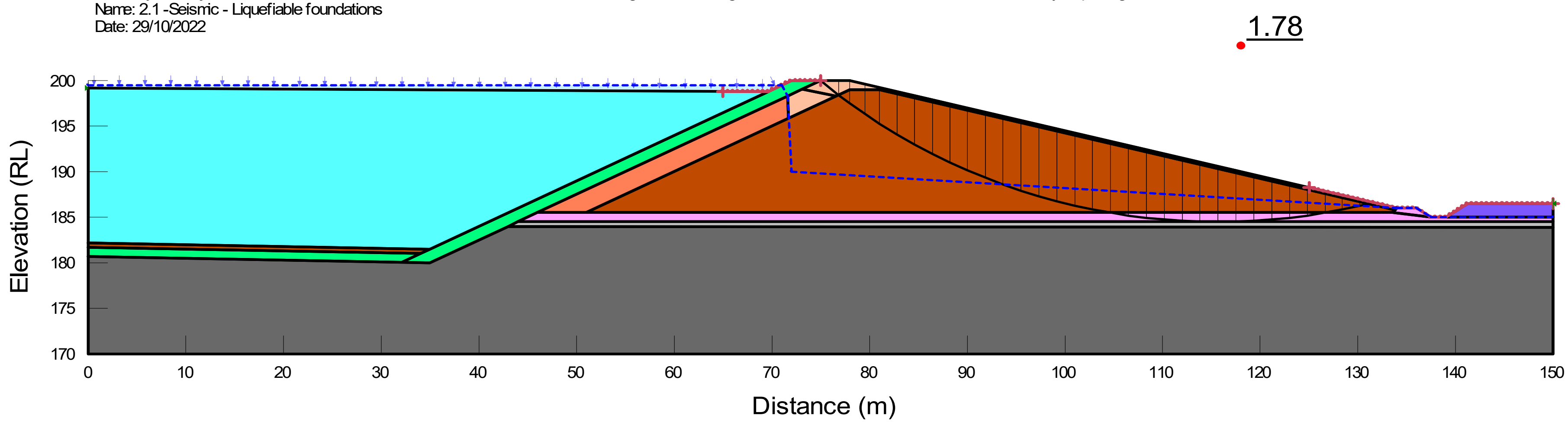
Brunswick West Tailings Storage Facility

Embankment Instability Assessment - Post Seismic Stability
Base Case

Date: 30/11/2022 **Job No:** 109014.15 **FIGURE C7**

K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\Appendix C - Embankment Instability Assessment C7

Costerfield Brunswick West TSF Design
File Name: New TSF - BGM Design (Stability - CFMA).gsz
Directory: K:\Projects\109\109014 Costerfield Mine, Costerfield\15 New TSF Investigation and Design\Data and Calcs\Risk Assessment\CFMA\Stability\Reporting\
Name: 2.1 -Seismic - Liquefiable foundations
Date: 29/10/2022



Color	Name	Model	Unit Weight (kN/m³)	Strength Function	Cohesion' (kPa)	Phi' (°)	Phi-B (°)	Minimum Strength (kPa)	Tau/Sigma Ratio	Piezometric Line
	Compacted Clay (Zone 1/1B) (80%)	SHANSEP	19					16	0.224	1
	EW Siltstone Foundation (80%)	Mohr-Coulomb	22		0	28.8	0			1
	Foundation Clay (Liquefied)	SHANSEP	20.5					0	0.1	1
	Foundation Clay (Post Seis)	SHANSEP	20.5					16	0.28	1
	Fresh Tailings	Spatial Mohr-Coulomb	16		0	0	0			1
	HW Siltstone Foundation	Bedrock (Impenetrable)								1
	Rockfill (Zone 3A) - Saturated (Dr/Undr) (80%)	Mohr-Coulomb	22		12	18.4	0			1
	Rockfill (Zone 3A) - Unsaturated (Dr/Undr) (80%)	Mohr-Coulomb	20		24	22.4	0			1
	Rockfill (Zone 3B) (Dr/Undr) (80%)	Shear/Normal Fn.	22	Leps Lower Bound (80%)			0			1

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MANDALAY RESOURCES COSTERFIELD OPERATIONS

Costerfiled Gold Mine

Brunswick West Tailings Storage Facility

Embankment Instability Assessment - Post Seismic Stability
Inadequately Prepared Foundations

Date: 30/11/2022 **Job No:** 109014.15

FIGURE C8