Appendices

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Appendix 1

Tailings Storage Facility Design Report (November 2011)

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CORTONA RESOURCES LIMITED

DARGUES REEF GOLD PROJECT

TAILINGS STORAGE FACILITY
FINAL DESIGN

Ref. PE801-00139/05
November 2011

Rev. No.	Date	Description	KP Approved	Client Approved
A	29 November 2011	Issued for Client Review	DJTM	
0	16 December 2011	Issued as Final	DJTM	

Knight Piésold Pty Limited
A.B.N. 67 001 040 419
Level 1, 184 Adelaide Terrace
East Perth WA 6004
AUSTRALIA

CONTENTS **PAGE**

CONTENTS **PAGE**

Knight Piésold CONSULTING

CONTENTS **PAGE**

KP_svr\...\PE801-139_5 Dargues Reef TSF Final Design Rev 0

CONTENTS **PAGE**

KP_svr\...\PE801-139_5 Dargues Reef TSF Final Design Rev 0

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CONTENTS **PAGE**

1. INTRODUCTION

1.1 GENERAL

Cortona Resources Limited is proceeding with the final design and construction of its Dargues Reef Gold Project, situated in the Lachlan Fold Belt of the Southern Tablelands region of NSW. As part of the Bankable Feasibility Study (BFS), Independent Metallurgical Operations Pty Ltd (IMO) engaged Knight Piésold Pty Ltd (KP) to provide engineering services for tailings and water management and associated works, and this study was presented to IMO in a report titled "Dargues Reef Gold Project – Bankable Feasibility Study. Tailings Management", PE801-00139/03 (Ref. 1) in October 2010. The report detailed a feasibility level design of a tailings storage facility and provided construction quantity and cost estimates for the facility. Subsequently, Cortona Resources appointed Knight Piesold to carry out the final design and construction supervision of the tailings storage facility (TSF).

This report details the final design of the TSF. Interpretation of site conditions is based on the sub-surface lithology revealed during the investigation programme, visual assessments of the in situ materials, the results of in situ field tests, and the results of laboratory testing carried out on selected representative samples collected during fieldwork. The tailings storage facility is designed in accordance with the requirements of the NSW Dams Safety Committee (DSC) guidelines *"DSC3F – Tailings Dams"* (Ref. 2).

1.2 PROJECT DESCRIPTION AND LOCATION

The Dargues Reef Project is a gold prospect located approximately 14 km south of the town of Braidwood and approximately 60 km east-south-east of Canberra as shown on Drg. No. PE801-00139-001. The operation will mine 330,000 tonnes per annum using conventional long hole open stope mining methods via a decline. A paste fill process will be used in the stoped out areas and waste rock will also be used as stope backfill allowing maximum orebody extraction and limiting haulage of waste to surface.

The plant is designed to extract half the gold via a simple gravity process and to produce a gold-silver-pyrite concentrate, containing the other half of the gold. No cyanide will be used at the Dargues Reef process plant. The concentrate will be transported to the London Victoria Mine near Parkes where it will be further processed and refined to produce gold dore.

The proposed infrastructure for the mine comprises the following:

- A plant site incorporating a primary crusher, ball mills, gravity circuits, and flotation cells. This facility will be located approximately 500 m to the west-north-west of the TSF and will be orientated north-south.
- The mine entrance portal will be situated approximately 400 m to the west-south-west of the TSF and will be orientated south to north.
- The tailings storage facility (TSF) will be located relative to the plant site and boxcut and is situated on an unnamed ephemeral creek and surrounding agricultural land.
- A paste hole, vent riser and escapeway are located approximately 150 m, 200 m and 250 m respectively south of the TSF embankment.

1.3 DESIGN OBJECTIVES

The overall aims of the design against which the more detailed design work has been conducted are defined by the design objectives. For final design, these are as follows:

- Provide long-term and safe containment of all solid waste materials.
- Maximisation of tailings density using sub-aerial deposition.
- Minimisation of seepage.
- Enable rapid and effective rehabilitation.
- Provide long-term structural stability of the facility.
- Enable ease of operation.
- Allow staged construction to facilitate modifications required as a result of operational experience.
- Minimum impact on surrounding environment.

1.4 DESIGN PARAMETERS AND ASSUMPTIONS

KP developed the overall design criteria during the Bankable Feasibility Study and the same parameters have been adopted for the final design. The only significant change to the design assumptions is that the plant commissioning date has been amended to February 2013. The design criteria, as summarised in Table 1.1, are in accordance with the following NSW DSC guidelines:

- *"DSC3F – Tailings Dams"* (DSC3F) (Ref. 2).
- *"DSC3A – Consequence Categories for Dams"* (DSC3A) (Ref. 3).
- *"DSC3B – Acceptable Flood Capacity for Dams"* (DSC3B) (Ref. 4).

- *"DSC3C – Acceptable Earthquake Capacity for Dams"* (DSC3C) (Ref. 5).

Table 1.1: Design criteria

HYDRAULIC DESIGN	
Temporary diversion structures during construction	• 1:25 Annual Exceedance Probability (AEP).
Diversion channel erosion protection	• 1:10 AEP (at channel outfalls only).
Diversion channel capacity	• 1:100 AEP.
TSF storm storage capacity	The more onerous of the following scenarios apply: • 1:10 AEP, 72 hour storm event on top of a 1:100 AEP. 72 hour storm event in addition to the maximum pond operating volume for average climatic conditions without the pond abutting the embankment wall. • 1:100 AEP wet sequence in addition to the maximum pond operating volume for average climatic conditions without the pond abutting the embankment wall. • 1:100 AEP, 72 hour storm event in addition to the maximum pond operating volume for average climatic conditions without the emergency spillway operating. • 1:10,000 AEP, 72 hour storm event in addition to the maximum pond operating volume for average climatic conditions without exceeding the capacity of the emergency spillway.
TSF hydraulic structures at closure	• The TSF basin will be graded to be free-draining and the emergency spillway will be designed for a 1:10,000 AEP storm event.
OPERATIONS	
Capacity - Final - Starter	• 890,000 t of dry tails over 63 months. • 170,000 t of dry tails – 1 year initial capacity.
Production Days/Month	• Tailings Discharge Rate 14,200t per month
Production Days/Year	• 330 (90.4% availability).
Design factor for pipes and pumps	• 20%
Slurry Characteristics	• 69% solids by weight. • SG solids = 2.71. • SG liquor = 1.001. • Slurry settled density = 1.35 to 1.40 t/m^3. • Permeability of 1 x 10^{-6} m/s.
Fluid Management	• Partial basin drainage system which gravity drains to a sump and is then pumped back to the supernatant pond. • Decant tower removal of supernatant solution via a pumping system and pressure pipeline back to the plant.
PRIMARY EMBANKMENT	
General	• Deposition from main embankment crest. • Minimum tailings freeboard of 0.3 m. • The supernatant pond will form at the head of the valley. Decant structures will be constructed at Stage 1 and final stage to permit removal of water from the pond.
Construction	• Upstream toe cut-off key trench and drain. • Zoned starter embankment constructed from local borrow comprising an upstream low permeability zone (with HDPE lining on the upstream face) and downstream structural zone. • 6 m crest width.

Table 1.1 (Cont.): Design criteria

Materials	• Remove unsuitable foundation soils from embankment footprint. Structural fill won from mine waste and local borrow. • Low permeability material won from selected local borrow areas within and near to the basin.
TAILINGS BASIN	
Basin Lining	• In situ soils, scarified, moisture conditioned and compacted to form a soil liner. • Composite liner (compacted in situ soil plus 1.5 mm smooth HDPE liner along the creek line and the area beneath the decant pond).
Basin Underdrainage	• Partial basin underdrainage system comprising main collector drains along the basin spine and finger drains at 10m spacing beneath the decant pond area.

1.5 ENVIRONMENTAL COMMITMENTS

The TSF is designed to meet the following environmental objectives:

- Provide permanent and secure confinement of all solid waste materials
  Design of the facility is in accordance with the requirements of the NSW Dam Safety Committee guidelines on the safe design and operating standards for tailings storages as a minimum.
- Limit impact on flora and fauna
  The facilities will be located on historically cleared areas to avoid unnecessary impacts to vegetation, flora and fauna within the area. In addition all necessary work required to limit/control weeds on areas around the facilities will be undertaken.
- Address visual amenity
  The final profile of the TSF (at closure) will resemble a small hill with a cover of light vegetation. The facility will be completely rehabilitated soon after decommissioning. The main focus of the rehabilitation program will be re-spreading of harvested topsoil, re-vegetation, erosion control (with rock armour protection on the embankment face if required) and stormwater management.
- Limit impact on groundwater
  Impact on the local groundwater regime will be controlled by the facility basin liner and seepage minimisation systems. In order to facilitate early detection and seepage remediation, a series of groundwater quality monitoring stations will be installed.
- Minimise seepage
  The primary aim of the seepage management strategy is to control seepage at source by:

i. Limiting the extent of the supernatant pond on the tailings beach, and thus maximising evaporation potential from the tailings beach surface.

ii. Construction of a cut-off trench, excavated into the foundation soils, that is then backfilled with low permeability fill, to reduce seepage loss under the embankment foundations.

iii. Constructing a suitable basin lining system for the facility, comprising a combination of in situ soil liner (in the non-critical areas of the basin) and a composite soil liner / HDPE geomembrane along the valley creek line and the area beneath the supernatant pond.

iv. Capping the TSF, at closure, to reduce the quantity of surface rainfall run-off that infiltrates the facility.

- Limit impact on surface water
  Best management practices will be implemented during operation and closure to ensure that the quality of surface water run-off from the facility is maintained within environmentally acceptable limits.

1.6 CONSEQUENCE CATEGORY

The consequence category for the Dargues Reef TSF has been assessed on the basis of DSC guidelines DSC3A and DSC3F.

A tabulation of the assessed severity of damage and loss, (based on Table 2 of DSC3A) is presented in Table 1.2. The population at risk (PAR) is assessed to be in the range 1 to 10 and the probable loss of life (PLL) is assessed to be 0. On this basis, a consequence category of *Low* would be indicated for the design of the TSF.

DSC guideline DSC3F provides an amplified version of the matrix used for assessment of the consequence category. This matrix recognises the difficulties of quantitatively determining the environmental consequences of dam failure. The matrix is reproduced as Table 1.3. On the basis of the assessment of impacts relative to the guidelines summarised in Table 1.3, the consequence category of the TSF is *Significant*. The DSC acknowledged this classification in writing and a copy of their letter of prescription is provided in Appendix A.

Table 1.2: Consequence categories for dams

Impact	Type	KP Comments	Severity of damage and loss
Social	Loss of cultural amenity	Dam failure could result in significant physical damage to items of local heritage	Minor
Natural Environment	Area of impact	0.1 km^2 to 1 km^2	Minor
	Duration of impact	1 month to 3 years	Minor
	Impacts on conservation value	Physical damage will be limited to areas that are extensively cleared of vegetation.	Negligible
	Impacts on plants and animal habitat	Physical damage will be limited to areas that are extensively cleared of vegetation.	Negligible
	Riverine landscape processes	Localised impacts in river connectivity expected.	Minor
DSC3A Table 2: Consequence Categories Based on Population At Risk (PAR)			
Population at Risk (PAR)		The box cut entrance to underground workings is located approximately 400 m west-south-west of the TSF embankment toe on the opposite side of Spring Creek. The process plant site and nearest offices are located about 500 m to the west-north-west of the TSF. The paste hole, vent riser and escape-way are located 150 m, 200 m, and 225 m downstream (south) of the TSF embankment toe, respectively. The population at risk (PAR) is assessed to be 1 to 10	Low
DSC3A Table 1: Consequence Categories Based on Probable Loss of Life (PLL)			
Probable Loss of Life (PLL)		There will be regular routine inspections of the facility by operating personnel. It is unlikely that dam failure would occur without any warning. Mine staff will be trained, including attendance at DSC Dam Safety Management Courses. Warning systems will be in place. The probable loss of life (PLL) is assessed to be 0.	Low

Table 1.3: Assessment of consequence category

Population at Risk - PAR	Receiving Environment	Severity of Damage or Loss			
		Negligible (Benign Liquid)	Minor (Benign Solid)	Medium (Saline Liquid / Unsightly Solid)	Major (Acid / Toxic Tailings)
<1	Remote / Degraded	Very Low	Very Low	Low	Significant
1-10	Rural / Productive	Low	Low	Significant	High C
10-100	Urban / Sensitive		Significant	High C	High B
100-1000				High A	High A
>1000					Extreme

1.7 SITE SELECTION AND CONSTRAINTS

1.7.1 General

Site selection for the TSF was carried out prior to the BFS, taking account of current land usage, the proximity of borrow for construction materials and the exploration potential within the existing leases. Two potential TSF sites were assessed:

- Option 1 – Nominal square / rectangular paddock facility to the south of the plant area.
- Option 2 – Valley storage to the east of the plant site.

A brief description of the two options is included below. A detailed description of the TSF options is provided in a technical memorandum that is included herewith, as Appendix B.

1.7.2 TSF Option 1

The facility consists of two cells of the following geometry:

Facility Dimensions	Cell 1 - 215 m x 240 m	Cell 2 - 215 m x 215 m
Average tailings depth	8.1 m	
Embankment heights	1 to 17 m (Cell 2 crest level nominally 7 m higher than Cell1)	
Estimated Embankment Volume		
Downstream	666,000 m^3	
Upstream	200,000 m^3	
Distance to plant (mill area to centroid of TSF)	500 m	

Based on the two cell configuration, the water balance modelling indicates a water deficit, with make-up (59%) required from another source.

1.7.3 TSF Option 2

This option comprises a cross valley storage located to the east of the plant. The facility dimensions are as follows:

- Final tailings footprint: 9.3 ha
- Catchment area (inside diversion channels): 12.0 ha
- Maximum embankment height: 30 m
- Estimated embankment volume (downstream construction): 184,800 m^3
- Distance to plant (mill area to centroid of embankment): 480 m

It may be possible to construct the embankment using a modified centreline configuration which would potentially save 30% of the embankment volume. Similar to Option 1, Option 2 TSF water balance is strongly negative with water make-up (57%) required from another source. However it would be possible to reduce the shortfall by modifying the diversion channels.

1.7.4 Evaluation of Options

Both of the options selected are viable tailings storage areas. From an embankment volume point of view Option 2 is more efficient. Option 2 was selected as the preferred location for the tailings storage facility.

2. SITE CHARACTERISTICS

2.1 TOPOGRAPHY

The terrain comprises a series of rolling hills intersected by ephemeral creeks. The base of the main drainage through the TSF basin falls towards the south-west at a gradient of about 8 to 10%. Ground level within the TSF footprint varies between approximately 715 m AHD at the head of the valley and 685 m AHD at the proposed embankment location.

The landscape is predominantly cleared except along the creek lines.

2.2 SURFACE DRAINAGE

The drainage in the area of the TSF is dominated by an ephemeral creek, which becomes very narrow and densely vegetated towards the south-west.

2.3 REGIONAL GEOLOGY

The site is located within the Lachlan Fold Belt and is associated with the Devonian-aged Braidwood Granodiorite. The Braidwood Granodiorite is a large elliptical pluton covering approximately 1,000 km^2. The western contact of the granodiorite dips at a low angle towards the west while the eastern contact dips steeply to the east. This geometry is consistent with the intrusion having been tilted about 20° to the west following displacement during recent Tertiary-aged block faulting.

Westward tilting followed by erosion has led to the eastern portion of the intrusion being exposed at a deeper magmatic level than the western portion. The Braidwood Granodiorite has been geologically mapped as a hornblende-biotite granodiorite, with the eastern phase dominantly biotite granodiorite and the western phase being hornblende granodiorite. The unaltered hornblende granodiorite is a light coloured, equigranular granodiorite containing plagioclase, K-feldspar, quartz, brown-green hornblende, minor chlorite altered biotite and accessory magnetite, apatite, sphene, and zircon, with a trace of pyrite.

The Braidwood Granodiorite intrudes early Devonian-aged Long Flat Volcanics to the west and Ordovician-aged sediments to the east. Regional aeromagnetic data indicates that the Braidwood Granodiorite underlies the Long Flat Volcanics for approximately 10 km to the west of the western contact exposed at the surface.

The Braidwood Granodiorite is cut by a number of east-south-east and south-east trending faults. These faults appear to control drainage patterns within the area where the granodiorite is exposed at surface. The granodiorite is also cut by a second suite of structures striking to the north-north-east. Placer alluvial gold occurs in recent sediments deposited in the east-south-east and south-east drainage systems in the south-west portion of the pluton.

2.4 LOCAL GEOLOGY

The 1:1,000,000 geological plan of the Majors Creek area is reproduced in Figure 2.1. The Dargues Reef project site occurs in the western part of a large granitic pluton, the Braidwood Granodiorite, which trends approximately north-south and extends from north of Braidwood to south of Majors Creek.

Dargues Reef was mined in the 19th century. Major mineralisation occurs on the northern side of a diorite dyke which crops out in Spring Creek, a short way downstream of the confluence of the unnamed creek (TSF creek) with Spring Creek.

Cooling of the pluton appears to have created a conjugate joint set of regional extent, striking 060° and 135°. TSF Creek appears to be related to the 060° set. The fault shown on the geological map between Majors Creek and Araluen strikes at approximately 135°.

The proposed TSF site is typically underlain by residual soils derived from the weathering of the granite to a depth of 2 m to 3 m. The residual soils are clayey in the upper metre, but are essentially non-plastic below this layer. Boulders were not encountered in the residual soils but were encountered in the colluvial and alluvial materials underlying the creek bed.

2.5 SEISMIC ASSESSMENT

An assessment of the seismicity of south-eastern Australia has been carried out and probabilistic seismic hazard analyses have been completed for the Dargues Reef project site. Existing information and historical data, including earthquake catalogues and technical publications on the tectonics and seismicity of the region have been reviewed. The most prominent seismic source in the region that defines the seismic hazard for the project is the Lachlan Fold Belt, an areal source zone thought to be capable of causing earthquakes up to Magnitude 6.1. The Dalton-Gunning zone, located approximately 30 km to the north of the site, is another areal source zone that contributes significantly to the seismic hazard at Dargues Reef. This seismic source zone is also thought to be capable of causing earthquakes of up to M6.1.

The computer program EZ-FRISK was used to develop a seismic hazard model for the Dargues Reef site. Seismic sources defined in the hazard model include shallow crustal earthquake sources such as the Lachlan Fold Belt and Dalton-Gunning zone, but also those located within the wider Sydney Basin area and Tasman Sea Margin. Appropriate attenuation models defining the relationship between earthquake magnitude, source to site distance and peak ground acceleration have been used in the probabilistic analysis.

Seismic design parameters have been determined for use in the design of the TSF. Seismic ground motion parameters (including peak ground acceleration, earthquake magnitude and response spectra) have been determined using the results of the probabilistic seismic hazard analysis.

It is recommended that the 1 in 475 year earthquake equivalent peak ground acceleration of 0.07g is adopted as the Operating Basis Earthquake (OBE) for the TSF. For a design operating life of 63 months the probability of exceedance for the OBE event is 1%. A design earthquake magnitude of 5.8 within the Lachlan Fold Belt or Gunning-Dalton Fault Zone at a hypocentral distance of 25 km is appropriate for the OBE. The TSF and appurtenances are expected to remain functional and any damage from the occurrence of earthquake shaking not exceeding the OBE would be easily repairable.

An appropriate Maximum Design Earthquake (MDE) for the TSF has been determined based on the DSC guidelines DSC3A and DSC3C. The facility has been classified as a *Significant* consequence category dam and, therefore, an appropriate MDE would be equivalent to an annual exceedance probability (AEP) of 1 in 500. However, given that this is virtually the same return period as that adopted for the OBE, it is proposed that the adopted MDE should be the 1 in 1000 acceleration of 0.11g. This is a conservative approach that also allows for some of the uncertainty inherent when conducting earthquake hazard assessments within Australia. Considerable damage to the tailings dam is acceptable under seismic loading from the MDE, provided that there is no uncontrolled loss of storage due to partial or complete failure of the dam.

A copy of the detailed seismicity assessment for the site is presented in Appendix C.

3. GEOTECHNICAL SITE INVESTIGATION

3.1 SITE INVESTIGATION

3.1.1 Scope of Work

A geotechnical investigation was carried out at the site of the proposed TSF in order to determine ground conditions and to provide design parameters for design of the facility.

The scope of work was carried out during the period 3rd to 27th May 2010 and comprised the following:

- Walk/drive-over survey of the project area;
- Drilling of three boreholes using diamond coring techniques;
- Test pitting at twenty eight locations;
- In situ permeability testing.

Fieldwork was supervised by an experienced geotechnical engineer from KP. The borehole and test pit locations were pre-determined by KP and agreed with Cortona Resources, and the locations were set out by a Cortona site representative. Minor changes were made to some of the originally proposed locations during fieldwork as a result of access constraints.

The geotechnical fieldwork was undertaken in accordance with the guidelines presented in *"Geotechnical Site Investigations, AS 1726"* (Ref. 6) and samples were collected for laboratory testing.

3.1.2 Walkover Survey

A walkover/drive-over survey of the mine area was carried out by a Senior Geotechnical Engineer from KP on 3rd May 2010 in order to make an assessment of the nature of ground conditions and to inspect specific areas of the investigation. A photographic record of the site inspection is provided in Appendix D.

3.1.3 Borehole Drilling

Three boreholes were drilled to depths of between 27.3 m and 29.8 m using a truck-mounted Edson drilling rig supplied and operated by Terratest Drilling. Each borehole was drilled by auger to approximately 1 m below ground level and then continued using HQ3 size triple-tube coring techniques with either mud or water flush. The core was placed into core trays and logged and photographed by the KP site representative, and was subsequently transported to the mine core shed for storage. On completion of drilling each of the boreholes was grouted to surface.

A summary of the drilling works is presented in Table 3.1.

Table 3.1: Summary of TSF drilling

Borehole	Bottom Depth (m)	Depth to Bedrock (m)	Depth to Groundwater (m)
DRTSF1	27.3	0.2	N/M
DRTSF2	29.8	0.4	N/M
DRTSF3	29.6	2.5	N/M

N/M – not measured

Logs and photographs of the boreholes are presented in Appendix E. The borehole locations are shown in Figure 3.1.

3.1.4 Packer Testing

During the drilling of each borehole, packer tests were undertaken over three depth intervals (typically 3 – 5 m, 10 – 15 m and 20 – 30 m) in order to assess the in situ permeability characteristics of the bedrock. Each test was carried out by inserting a wireline gas-inflated packer at the top of the section to be tested, and then measuring the volume of water acceptance into that section of borehole at 5 minute intervals and at varying water pressures. The results and interpretation of the packer testing are presented in Appendix F and a summary of the test results is presented in Table 3.2.

Table 3.2: Summary of packer test results

Borehole No.	Test Interval (m)	Material Description*	Interpreted Permeability (m/s)
DRTSF1	3 – 6	Granite, XW	Invalid test
	10 – 15	Granite, XW/SW	1.36×10^{-6}
	22 - 27	Granite, SW	2.32×10^{-6}
DRTSF2	3 – 6	Granite , XW	1.53×10^{-7}
	10 – 15	Granite, XW	Invalid test
	20 – 29.8	Granite, SW	No test
DRTSF3	3 – 5	Granite, XW/SW	No test
	9.3 – 14.3	Granite, SW	5.37×10^{-7}
	20 – 29.6	Granite, SW	2.30×10^{-7}

*Refer rock description terminology in Appendix E

3.1.5 Test Pitting

A total of twenty eight test pits were excavated across the TSF footprint to depths of up to 5.5 m using a Sumitomo SH200 tracked excavator supplied and operated by Braidwood Earthmoving. Each test pit was logged and photographed by the KP site representative and samples were collected from selected pits for laboratory testing. All of the tests pits were backfilled with excavated spoil on completion of excavation.

Logs and photographs of the test pits are presented in Appendix G. The test pit locations are shown in Figure 3.1.

3.1.6 Laboratory Testing

Representative samples of the in situ soils and rocks were collected from the test pits and boreholes and delivered to K&H Geotechnical Services in Parkes for testing. The purpose of the testing was to classify and characterise the soils in order to assess their behaviour characteristics and suitability for use in earthworks. Laboratory testing included the following tests:

- Atterberg limits and linear shrinkage.
- Particle size distribution.
- Standard compaction;
- Remoulded permeability.
- Emerson dispersion.

Tests were carried out in accordance with *"Method of testing soils for engineering purposes, AS 1289"* (Ref. 7), where applicable.

A summary of the laboratory tests is presented in Table 3.3 and the test reports are presented in Appendix H.

Table 3.3: Summary of laboratory test results

Test Pit	Depth	Particle Size Distribution			Liquid Limit	Plastic Limit	Plasticity Index	Linear Shrinkage	Emerson Dispersion	Standard Compaction		Remoulded Permeability
		Gravel	Sand	Silt/Clay						MDD	OMC	(k)
	(m)	(%)	(%)	(%)	(%)	(%)	(%)	(%)	Class	(t/m^3)	(%)	m/s
TP12	0.3 - 1.4	6	38	56	37	18	19	6.5	6	1.76	17.5	3×10^{-8}
TP14	0.3 - 1.0	4	41	55	58	23	35	12	5	1.68	19	2×10^{-10}
TP15	0.5 - 2.0	7	58	35	30	20	10	4.5	5	1.86	14	5×10^{-10}
TP16	0.2 - 0.6	3	76	21	25	21	4	2	5	1.82	15	8×10^{-7}
TP17	0.3 - 1.1	3	53	44	38	18	20	6.5	3	1.72	19	3×10^{-10}
TP18	0.5 - 1.1	5	54	41	43	24	19	7.5	3	1.78	15.5	6×10^{-10}
TP20	0.3 - 1.0	4	53	43	51	35	16	10	5	1.73	18.5	2×10^{-8}

3.2 SUB-SURFACE CONDITIONS

3.2.1 Tailings Storage Facility

Three boreholes were drilled and twenty-eight test pits were excavated across the proposed footprint of the TSF in order to assess the founding conditions for the TSF embankment, to provide permeability data for seepage analysis, and to provide further observations on the near surface materials for assessment of constructability and borrow potential.

Extremely weathered granite was encountered from near surface at all locations. This material comprises either clayey sand or sandy gravelly clay within the upper one metre and becomes less plastic with depth. Occasional clayey bands were noted at depth associated with differential weathering and groundwater flow. Significant weathering was encountered to depths of 12 m and 20 m in boreholes DRTSF1 and DRTSF2 respectively, whilst in borehole DRTSF3 (at the base of the creek) the weathering was encountered to a depth of only 6.8 m. Slightly weathered and very to extremely high strength granite was encountered beneath the weathered zone to the base of each borehole.

Alluvium was encountered in one borehole and at five test pit locations, all in proximity to the creek line, from ground level to depths of between 2 m and 3 m. The alluvium typically comprises layers of sand and clay with silt and gravel occasionally lensed or interbedded. The material was generally found to be of soft or loose consistency.

Colluvium was encountered in four test pits from ground level to depths of between 2 m and 3 m. The colluvium typically comprises layers of gravelly sand and clay. However, in TP15 the colluvium mainly consisted of large boulders. The material is of variable consistency.

Made ground was encountered in TP11 (downstream of the proposed embankment) to a depth of 1.6 m and comprised gravelly sand. It is thought that this material could be detritus deposited at the base of the creek and associated with previous mining activities.

A summary of the sub-surface profile within the TSF footprint is presented in Table 3.4 Reference should be made to the borehole and test pit logs for specific information relating to particular locations.

Table 3.4: Summary of ground conditions – TSF

Depth to base of horizon (m)	Material Description	Consistency / Weathering	Location
0 – 0.3	TOPSOIL	Varies	All
0.3 to 1.6	MADE GROUND, gravelly sand, yellow.	Not classified	TP11
0.3 to 3.0	ALLUVIUM, sand and clay with organic or root material occasionally interbedded or lensed, yellow, brown, cream and grey.	Soft/loose consistency.	Creek
0.3 to 3.0	COLLUVIUM, gravelly sand and clay	Soft to firm and loose to very dense consistency.	Creek
6.8 to 16.5	WEATHERED GRANITE, recovered as sand and gravel, occasionally clayey or clay, pale brown, cream and orange.	Extremely weathered. Assessed as firm to stiff where cohesive and medium dense to dense where non cohesive.	All
Not penetrated	GRANITE, medium coarse grained, black, white, grey, cream, pink, with various zones of alteration.	Slightly weathered, generally of high to extremely high strength	All

3.2.2 In situ Permeability

In situ permeability testing yielded a typical permeability range at depth of between 1.5×10^{-7} m/s and 2.3×10^{-6} m/s indicating that the sub-surface profile is relatively permeable. Laboratory permeability testing on samples of the near surface soils remoulded to 98% SMDD yielded permeabilities ranging between 8×10^{-7} to 6×10^{-10} m/s indicating that there is potential to re-work the near surface subgrade to form a compacted soil liner.

3.2.3 Groundwater

Due to the nature of the drilling groundwater level was not readily monitored. However, groundwater was noted at depths of between 1.6 and 2.9 m below the base of the creek in test pits TP16, TP19 and TP24.

4. TAILINGS CHARACTERISTICS

4.1 GENERAL

Tailings testing was undertaken for the Dargues Reef Gold Project based on samples provided by Cortona. Two samples were provided, the first in May 2010 and the second in August 2010. They were described as follows:

> *"The first sample was generated from a composite prepared for bulk testwork. A 40kg sub-sample was ground to a P_{80} of 250µm before being subjected to a single rougher flotation test. Part of this rougher tail was adjusted to 55% solids and delivered to Knight Piesold for testing. This sample should be called 'Composite 6 Rougher Tail – Test no. HS21769'.*
>
> *The second sample was also generated from a composite prepared for bulk testwork. A 45kg sub-sample was ground to a P_{80} of 212µm before a rougher flotation test was completed. The rougher tailings were stored and the rougher concentrate was reground to a P_{80} of 75µm and a cleaner flotation test was performed. The rougher tail and the cleaner tail was combined to generate a Combined Flotation Tail. Part of this Combined Flotation Tail was adjusted to 58% solids and sent to Knight Piesold for testing. This sample should be called 'Stage 3 – Combined Flotation Tail'."*

The first sample (labelled "Rougher") was only 5L in total and thus only basic settlement and air drying characteristics were measured. The testing was carried out at approximately 50 – 55% solids which was the expected tailings discharge percent solids. The rougher sample test data is no longer relevant to this study and consequently the results are not included or discussed herein.

The second sample (labelled "Combined") was provided at approximately 58% solids. The plant design includes a paste thickener capable of thickening the tailings to about 70% solids. The sample was tested both at 70% solids and at a lower percent solids range (55 – 60%). The second sample was indicated by the client as being more representative of the expected tailings product and the results from testing on this sample have been used for design purposes.

The following tests were carried out on both the samples:

i. Classification tests to determine:
   - Particle size distribution of the tailings.
   - Supernatant liquor density.
   - Liquid and plastic limits of the tailings solids.
   - Tailings solids particle density.

ii. Undrained and drained sedimentation tests.

iii. Air drying tests.

iv. Permeability tests.

Consolidation tests were carried out only on the Combined sample.

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

4.2 TESTING PARAMETERS

The Combined sample was tested at a design target percent solids of 69%. A select number of tests for the Combined sample were also performed at a lower percent solids for comparative purposes.

4.3 LABORATORY TESTING RESULTS

4.3.1 Classification Testing

Classification testing was completed at the Perth laboratory of SGS. Where appropriate, classification tests were conducted in accordance with relevant Australian Standards. The results of the classification tests are summarised in Table 4.1 and the laboratory test reports are presented in Appendix I.

Table 4.1: Summary of classification test results – Combined tailings

Test	Combined	AS1289
Tailings Particle Density (t/m^3)	2.71	3.5.1 (1995)
Supernatant Density (t/m^3)	1.001	(hydrometer)
Supernatant pH	7.6	(pH meter)
Liquid Limit (%)	27	3.1.2 (1995)
Plastic Limit (%)	NP	3.2.1 (1995)
Plasticity Index (%)	NP	3.3.1 (1995)
Linear Shrinkage (%)	0.5	3.4.1 (1995)

NP – non plastic

The particle size distribution analysis on the Combined tailings sample was completed in accordance with AS1289 3.6.3 - 2003. The measured particle size distribution is summarised in Table 4.2 and the grading curve is shown in Figure 4.1.

Table 4.2: Particle size distribution - Combined tailings

Particle Size (µm)		Percent Passing (%)
600	Sand	100
200		72
75	Silt	32
20		21
6		11
2	Clay	7
~P_{80}		230µm

The particle size distribution sample indicates a well graded and predominantly fine sand size with silt. The Combined sample consists of approximately 68% sand, 25% silt and 7% clay and is classified as a silty sand with a trace of clay (SM) under the Unified Soil Classification System.

4.3.2 Percent Solids Measurement

During the preparation of each test, two sub-samples of the tailings were subjected to oven drying to determine the percent solids of the sample. In addition to these measurements, the percent solids were back calculated from other test results for comparison. Table 4.3 presents the results of the percent solids tests.

Table 4.3: Percent solids of tailings samples

Sample	Oven Drying Samples		Percent Solids Range Back-Calculated from Tests	Mean Percent Solids Back-Calculated from Tests
	Test 1 (%)	Test 2 (%)	(%)	(%)
Combined (58%)	58.1	58.1	54.5 to 58.6	56.8
Combined (71%)	71.0	71.0	69.2 to 71.8	70.8

The results indicate that there is some variance in the percent solids for the individual tests. The variation in percent solids between tests is, however, within reasonable limits.

4.3.3 Sedimentation Tests

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

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

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

The results of the sedimentation tests are presented in figures 4.2 and 4.3. Table 4.4 presents a summary of the measured sedimentation test data.

All the tests indicate that the tailings are fast settling with water production and final densities achieved within a period of 24 hours. The lower percentage solids Combined sample released approximately 37% of the water in slurry to supernatant for the undrained case, increasing to 43% total water release (both supernatant and underdrainage) for the drained case. There is no significant increase in settled density between the drained and undrained tests for the Combined sample.

The Combined paste-thickened sample releases approximately 14% of the water in slurry to supernatant in the undrained test, reducing to 5% supernatant in a fully drained scenario with an additional 11% reporting to underdrainage. Again there is no significant increase in settled density between the drained and undrained tests for the thickened Combined sample. There is an increase in settled density from 1.21 t/m^3 to 1.41 t/m^3 when the discharge percent solids increases from 58% to 71%.

Table 4.4: Summary of sedimentation test results

Sample	Test	Initial Solids (%)	Supernatant (% of initial water volume)	Underdrainage (% of initial water volume)	Time to Achieve		Final Dry Density (t/m^3)	Figure
					90% of total density increase (days)	Final Density (days)		
Combined (58%)	Undrained	57.6	37	-	0.2	0.3	1.21	4.2
	Drained	55.6	22	21	0.2	0.3	1.21	4.3
Combined (71%)	Undrained	71.6	14	-	0.3	0.9	1.41	4.2
	Drained	70.0	5	11	0.2	0.3	1.41	4.3

4.3.4 Air Drying Tests

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

A direct relationship exists between dry density and moisture content up to a breakaway point, at which the degree of saturation falls below 100%. At this point, negative pore water pressures are developed, which further consolidates the tailings. Drying below a limiting saturation produces no further consolidation and the density at this point represents the maximum that can be achieved via air drying of the tailings.

The results of air drying tests are presented in figures 4.4 and 4.5 and are summarised in Table 4.5.

Table 4.5: Results of air drying tests

Sample	Moisture Content at Breakaway Point (%)	Dry Density at Breakaway Point (t/m^3)	Limiting Saturation Value (%)	Final Dry Density (t/m^3)
Combined	40	1.37	32	1.45

The Combined tailings sample achieved a dry density of 1.45 t/m^3 over a drying time of 5.5 days. The Combined air drying tests increased the final dry density by only 3% over the drained sedimentation density value. This indicates that initial settlement of the tailings provides the majority of the dry density in less than one day, with air drying providing only a marginal improvement in density.

4.3.5 Permeability Tests

Falling head permeability tests were conducted on samples of saturated tailings with measurements of drainage through the drained sedimentation sample being measured. In addition, permeability values were derived from the results of consolidation tests. Measured permeability data are summarised in Table 4.6 and presented in Figure 4.6.

Table 4.6: Summary of permeability test results

Sample	Test Type	Dry Density (t/m^3)	Permeability (m/s)
Combined (58%)	Consolidation Test	1.17	1.8×10^{5}
		1.20	3.8×10^{-6}
		1.25	1.3×10^{-6}
	Falling Head Permeability Test	1.17	1.0×10^{-6}
		1.25	1.0×10^{-6}
Combined (71%)	Falling Head Permeability Test	1.36	3.1×10^{-7}
		1.41	3.1×10^{-7}

These results represent the permeability of saturated tailings prior to further consolidation due to additional deposition loading or air-drying. In the range of expected settled densities the permeability is approximately 4.0×10^{7} m/s indicating a moderately high permeability tailings.

4.4 CONSOLIDATION TESTS

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

A consolidation test was undertaken on the Combined sample at low densities. The settlement with respect to time and the variation in permeability with density for the test is presented in Figure 4.6 and the results of the consolidation tests are summarised in Table 4.7.

Table 4.7: Summary of consolidation test results

Dry Density (t/m^3)	Stress Range (kPa)	Coefficient of Consolidation C_V (m^2/y)	Coefficient of Volume Decrease M_V (m^2/kN)	Compression Index C_C	Permeability (C_V based) (m/s)
1.14 – 1.25	1.57 – 4.44	321.0	0.030	0.46	1.06 x 10^{-7}

These results indicate that the tailings are moderately compressible but will consolidate very quickly.

4.5 PREDICTED PHYSICAL BEHAVIOUR OF TAILINGS

Based on the results of the testing on the Combined sample at 71% solids, the following behaviour is predicted for the tailings.

4.5.1 Water Production

The release of water following deposition of the tailings can be estimated from the results of the drained and undrained sedimentation tests. The rate of release will determine the amount of liquor available in the decant pond for collection, treatment and return to the process plant. The design is based on the thickened tailings results only.

The quantity of underdrainage release in the field would be expected to be lower than the values indicated by the laboratory testing, due to the thickness of the deposited tailings and further consolidation of the tailings.

The rate of supernatant release for the tests was extremely quick, taking less than a day to complete. It is expected that water release would be in the order of 5 to 10% of the water in slurry. Underdrainage could be as high as 7% of the water in slurry but would likely average around 2 to 3%.

4.5.2 Tailings Density

The settled dry density of tailings deposited into the TSF can be predicted from laboratory testing. The test results indicated that the tailings reached moderately high final dry densities at a relatively quick rate from settling, with only an incremental increase due to drying. It has been observed over a number of years that densities achieved in the field are generally lower than those obtained in the laboratory. In addition, field densities achieved are dependent on the area available for drying and the thickness of deposited layers. A suitable deposition plan and efficient operation of the facility can greatly improve settled density. Assuming that the facility is efficiently operated, it is estimated that the average settled density of the tailings will be between 1.35 and 1.40 t/m^3.

4.6 TAILINGS GEOCHEMISTRY

4.6.1 General

A single sample of tailings slurry was submitted for geochemical analysis of both the solids fraction and the supernatant water.

4.6.2 Geochemical Methods

Acid Base Accounting

Acid base accounting (ABA) assesses the sample's potential to form acid from oxidation of sulphides and the ability to neutralise acid by the dissolution of minerals, especially carbonates, contained in the sample.

The ABA test work was conducted by Genalysis in Perth. Total sulphur, total carbon and total inorganic carbon was determined by LECO induction furnace, with infrared detection. Sulphate sulphur was determined by 10% Na_2CO_3 extraction, with $BaSO_4$ precipitation. The testing work methods used are based on the ABA methodology defined in the *"Acid Rock Drainage Prediction Manual"* (Ref. 8) and *"Guidelines for Metal Leaching and Acid Rock Drainage at Mine Sites in British Columbia"* (Ref. 9).

Acid Neutralising Capacity (ANC) was determined by digestion in a standard solution of HCl, followed by back titration with NaOH to determine the amount of acid consumed. The technique used was based on the publication *"Field and laboratory methods applicable to overburden and mine soils"* (Ref. 10).

The results of the ABA testing are used to calculate the Maximum Potential Acidity (MPA) which is a measure of the maximum amount of sulphuric acid which can be produced from the total oxidation of all sulphides within the sample, assuming all sulphide is present as pyrite.

The Net Acid Producing Potential (NAPP) is the balance between the Maximum Potential Acidity and the Acid Neutralising Capacity. A negative NAPP indicates that there is an excess neutralising capacity and a positive NAPP indicates there is excess potential acidity.

Static Net Acid Generation

Static Net Acid Generation (NAG) testing provides a direct measure of the sample's ability to produce acid through sulphide oxidation. The addition of hydrogen peroxide to samples causes rapid oxidation of the contained sulphides to produce sulphuric acid.

The NAG testing was conducted by Genalysis. The procedure employed is based upon the Static NAG Test described in *"Advances in acid drainage prediction using the net acid generation test"* (Ref. 11) and *"Evaluation of the Net Acid Generation (NAG) Test for Assessing the Acid Generating Capacity of Sulfide Minerals"* (Ref. 12). Fifteen percent hydrogen peroxide solution was adjusted to pH 4.5 prior to addition to the samples. The samples were then left to stand overnight at room temperature before taking pH measurements. The samples were then boiled for 2 to 3 hours and allowed to cool before being made back up to 250 mL followed by pH measurement and titration to pH 7.0.

Acid Formation Potential

The acid formation potential of a sample is calculated based on acid base accounting, i.e. the balance between a sample's ability to produce acid from the oxidisation of sulphide minerals (MPA) and the sample's ability to neutralise acid by the dissolution of alkaline minerals contained within the sample (ANC). The balance between the MPA and the ANC is termed the net acid producing potential (NAPP), with a negative NAPP indicating an excess of acid neutralising capacity and a positive NAPP indicating an excess potential acidity.

Historically a safety margin was applied to the ratio between the ANC and MPA to allow for variability in the rates of acid production and neutralisation processes, and the potential for geographic separation of the acid producing and acid neutralising phases. This safety margin was generally set by industry at 2 in North America and 3 in Australia.

With recent advances in the understanding and acceptance of the NAG test there has been a move away from this method of classifying materials based solely on the ANC and MPA as these calculated parameters do not take into consideration the true availability of acid producing and acid neutralising phases.

Knight Piésold prefers to utilise the results of the Acid Base Accounting in combination with the NAG testing results to classify the acid formation potential of materials, and utilises the classification system as presented in Table 4.8. This is based on the Australian Government guideline *"Managing Acid and Metalliferous Drainage"* (Ref. 13) and is broadly similar to the classification system contained within *"ARD Test Handbook"* (Ref. 14) which is advocated by *"Global Acid Rock Drainage Guide (Version 0.7)"* (Ref. 15).

Table 4.8: Acid formation potential classification system

Acid Formation Potential Class	NAPP (kg H_2SO_4 /t)	NAG pH
Potentially Acid Forming (PAF)	>10	<4.5
Potentially Acid Forming – Low Capacity (PAF-LC)	0 to 10	<4.5
Non Acid Forming (NAF)	Negative	≥4.5
Acid Consuming (AC)	Less than -100	≥4.5
Uncertain	Positive	≥4.5
	Negative	<4.5

Multi-Element Analysis

Multi-element analysis of the samples was conducted to assess elemental enrichments within the samples. The testing was conducted by Genalysis in Perth. Digestion methods employed resulted in near total digestion of the samples to assess the whole rock geochemistry of the samples

Multi-element analysis results were compared to the average crustal abundance to give the geochemical abundance indices. The Geochemical Abundance Index (GAI) quantifies an assay result for a particular element in terms of average crustal abundance. The GAI is calculated from the following formula:

$$GAI = Log_2 (C_n / (1.5 \times B_n))$$

where:

Cn = measured concentration of element in sample

Bn = average crustal abundance as described in "Environmental Chemistry of the Elements" (Ref. 16).

The GAI is expressed on a scale of 0 to 6, with 0 indicating that the concentration of the element is less than or similar to average crustal abundance. A GAI of 3 corresponds to a 12 fold increase above the average crustal abundance, and so forth up to a GAI of 6 which represents a 96 fold increase or greater.

KP has assigned an arbitrary scale to the GAI with indices of 0 and 1 classified as unenriched, an index of 2 classified as slightly enriched, indices of 3 and 4 classified as significantly enriched. and indices of 5 and 6 classified as highly enriched.

Supernatant Water Quality

Characterisation of the tailings supernatant was conducted to assess the potential for the supernatant to cause environmental impacts to surface water or near surface groundwater. These tests differ from the multi-element tests in that they only record the readily soluble elements whereas the multi-element tests give the total elemental enrichment of the tailings solids.

The supernatant characterisation was conducted on a sample of slurry sent to Genalysis in Perth. The pH and the conductivity of the slurry were measured and the bottles left to stand for a minimum of 3 hours. The supernatant was siphoned off and filtered through 0.45 µm membrane before preservation of the solution by acid addition prior to analysis. The analysis was by Inductively Coupled Plasma Mass Spectrometry or Inductively Coupled Plasma Optical Emission Spectrometry depending on the element being analysed and the detection limits required.

The supernatant water quality test results were compared to a set of reference water quality standards which are detailed in the following section.

Reference Water Quality Standards

To allow assessment of the results of the supernatant analysis a set of reference values has been established. These reference values were compiled from internationally accepted guidelines for water quality for release from mining operations: *"IFC Environmental, Health and Safety Guidelines for Precious Metal Mining"* (Ref. 17) *"IFC Environmental. Health and Safety Guidelines for Mining"* (Ref. 18) and the *"Australian and New Zealand Guidelines for Fresh and Marine Water Quality"* (Ref. 19). The use of several guidelines is required as no single guideline contains target concentrations for all parameters. Where a target concentration for a specific element is at different levels in more than one guideline, the lowest concentration has been selected. The reference values adopted are summarised in Table 4.9. The water quality results of the supernatant analysis have also been compared to *"Australian Drinking Water Guidelines"* (Ref. 20) which are summarised in Table 4.10.

The establishment of these reference water quality values is to allow for evaluation only and it is not implied by reproduction of the reference water quality values that the Dargues Reef Gold Project is required to meet these reference levels or that these reference levels are used as the regulatory framework.

Table 4.9: Reference release water quality standards

Parameter	Unit	ANZECC Livestock	IFC 2004	IFC 2007	Adopted Reference Level
pH	S.U.		6 to 9	6 to 9	6 to 9
TDS	mg/kg	2000			2000
Aluminum	mg/L	5			5
Antimony	mg/L				N/G
Arsenic	mg/L	0.5	0.1	0.1	0.1
Barium	mg/L				N/G
Boron	mg/L	5			5
Cadmium	mg/L	0.01	0.1	0.05	0.01
Calcium	mg/L	1000			1000
Chloride	mg/L				N/G
Chromium	mg/L	1			1
Chromium (Cr^{+6})	mg/L		0.1	0.1	0.1
Cobalt	mg/L	1			1
Copper	mg/L	0.4	0.5	0.3	0.3
Cyanide-Total	mg/L			1	1
Cyanide-Free	mg/L			0.1	0.1
Cyanide-WAD	mg/L		0.5	0.5	0.5
Fluoride	mg/L	2	20		2
Iron	mg/L		3.5	2	2
Lead	mg/L	0.1	0.1	0.2	0.1
Magnesium	mg/L	2000			2000
Manganese	mg/L				N/G
Mercury	mg/L	0.002	0.01	0.002	0.002
Molybdenum	mg/L	0.15			0.15
Nickel	mg/L	1	0.5	0.5	0.5
Phosphorus	mg/L				N/G
Selenium	mg/L	0.02	0.1		0.02
Silver	mg/L		0.5		0.5
Sodium	mg/L				N/G
Sulphate	mg/L	1000			1000
Tin	mg/L				N/G
Uranium	mg/L	0.2			0.2
Vanadium	mg/L				N/G
Zinc	mg/L	20	2	0.5	0.5

Table 4.10: Reference drinking water quality standards

Parameter	Unit	Health	Aesthetic	Adopted Reference Level
pH	S.U.		6.5 to 8.5	6.5 to 8.5
TDS	mg/kg		500	500
Aluminum	mg/L		0.2	0.2
Antimony	mg/L	0.003		0.003
Arsenic	mg/L	0.007		0.007
Barium	mg/L	0.7		0.7
Boron	mg/L	4		4
Cadmium	mg/L	0.002		0.002
Calcium	mg/L			N/G
Chloride	mg/L		250	250
Chromium	mg/L			N/G
Chromium (Cr^{+6})	mg/L	0.05		0.05
Cobalt	mg/L			N/G
Copper	mg/L	2	1	1
Cyanide-Total	mg/L	0.08		0.08
Cyanide-Free	mg/L			N/G
Cyanide-WAD	mg/L			N/G
Fluoride	mg/L	1.5		1.5
Iron	mg/L		0.3	0.3
Lead	mg/L	0.01		0.01
Magnesium	mg/L			N/G
Manganese	mg/L	0.5	0.1	0.1
Mercury	mg/L	0.001		0.001
Molybdenum	mg/L	0.05		0.05
Nickel	mg/L	0.02		0.02
Phosphorus	mg/L			N/G
Selenium	mg/L	0.01		0.01
Silver	mg/L	0.1		0.1
Sodium	mg/L		180	180
Sulphate	mg/L	500	250	250
Tin	mg/L			N/G
Uranium	mg/L	0.02		0.02
Vanadium	mg/L			N/G
Zinc	mg/L		3	3

4.6.3 Results

Laboratory test reports for the tailings geochemical testing conducted by Genalysis are provided in Appendix I. The results are presented and discussed in the following sections. As part of the quality control and assurance programme conducted by the laboratory, duplicates of all tests were conducted on the tailings sample. The results presented in the following sections are the average values of the duplicate tests.

Acid Base Accounting

Total sulphur content of the tailings was determined by LECO combustion. In addition the sample was analysed for Na_2CO_3 soluble sulphate. The difference between these two values was assumed to be equal to the sulphide content of the tailings. The results of the analysis are presented in Table 4.11.

Table 4.11: Sulphur analysis results

Sample	Total Sulphur (%)	Sulphate Sulphur (%)	Sulphide Sulphur (%)	Maximum Potential Acidity (kg H_2SO_4 /t)
Tailings Solids	0.095	<0.01	0.085	2.6

The results of the analysis indicate that the tailings sample had a very low total sulphur content. The sulphate sulphur content was below detection limit indicating that all the sulphur is likely to be present in the form of sulphide sulphur. The maximum potential acidity was calculated from the sulphide sulphur content at 2.6 kg H_2SO_4 / tonne of tailings which is considered very low.

The acid neutralising capacity (ANC) of the sample was determined along with the carbonate content. The two results can be used as a check against one another and to identify the contribution of ANC from carbonates and other non-carbonate minerals. The results of the analysis are summarised in Table 4.12.

Table 4.12: Summarised carbonate and acid neutralising capacity results

Sample	Carbonate Carbon (%)	CO_3-ANC [1] (kg H_2SO_4 /t)	ANC [2] (kg H_2SO_4 /t)
Tailings Solids	0.99	82	89

[1] Calculated ANC from carbonate content
[2] Measured Sobek ANC

The results of the acid neutralising capacity and carbonate content indicate that significant carbonate is present and the acid neutralising capacity correlates well with the carbonate content.

The Net Acid Producing Potential (NAPP) of the sample was calculated from the MPA and the ANC, and is presented in Table 4.13 along with the ANC/MPA ratio. The net acid producing potential is strongly negative with a high ANC/MPA ratio indicating that there is substantial excess neutralising capacity in the sample.

Table 4.13: Summary of net acid producing potential results

Sample	ANC (kg H_2SO_4 /t)	MPA (kg H_2SO_4 /t)	NAPP (kg H_2SO_4 /t)	ANC/MPA (ratio)
Tailings Solids	89	2.6	-86	34

Net Acid Generation

The net acid generation (NAG) test aids in interpretation of acid formation potential classifications. It also identifies if the sulphides and neutralising minerals contained in the samples are readily available to produce or consume acid.

The results of the net acid generation test are summarised in Table 4.14 and indicate that under extreme oxidising conditions no measurable acid is produced and the pH of the NAG solution remains alkali. This correlates well with the calculated net acid producing potential.

The final NAG pH of the tailings after complete oxidation was 9 indicating that alkali conditions are likely to prevail within the tailings pore waters should complete oxidation of the tailings solids occur.

Table 4.14: Summary of net acid generation results

Sample	NAG (7.0) (kg H_2SO_4/t)	NAG pH
Tailings Solids	0	9

Acid Formation Potential

The sample's acid formation potential is calculated based on the acid base accounting results and the NAG test. The acid base accounting yielded a net acid producing potential of -86 kg H_2SO_4/t and a NAG pH of 9. The tailings are therefore classified as

Non Acid Forming. Figure 4.7 presents a graphical representation of the classification.

Tailings Solids Geochemical Enrichments

Whole rock multi-element analysis of the tailings solids was conducted to assess elemental enrichments within the solid portion of the tailings material. Multi-element analysis results were compared to the average crustal abundance to give the geochemical abundance indices. The Geochemical Abundance Index (GAI) quantifies an assay result for a particular element in terms of average crustal abundance.

The assay results, average crustal abundance (ACA) and corresponding geochemical abundance indices (GAI) are presented in Table 4.15. The results of the analysis show that the tailings solids contain a small number of elemental enrichments. Molybdenum and Antimony are classified as significantly enriched, and Silver as slightly enriched. Boron is classified as slightly enriched, but this is a result of the high detection limit for the test and therefore the sample may not actually be enriched in Boron.

The results of the analysis have been compared also to *"National Environment Protection Measure – Assessment of Site Contamination"* (Ref. 21). Guideline values for Antimony and Molybdenum are not available in this reference. To allow for assessment of the Antimony and Molybdenum concentrations, the concentrations contained in the samples have been compared to the Netherlands National Institute of Public Health and Environment intervention levels for soil as given in *"Proposals for intervention values for soil and groundwater, including for calculation of human-toxicological serious soil contamination concentration: fourth series of compounds"* (Ref. 22) and "*Ecological threshold concentrations for antimony in water and soil*" published by the European Centre for Risk Assessment (Ref. 23). The results of this comparison are shown in Table 4.16 and indicate that the concentration of enriched elements are below ecological or health based investigation levels for all parameters except Sulphur. The Sulphur is present in relatively low concentrations; however, there is sufficient neutralising capacity such that this does not present a risk of acid generation, indicating therefore that the material is unlikely to present a risk to the environment or to human health.

Table 4.15: Tailings solid multi-element results and geochemical abundance indices

Element	Unit	Multi-Element Analysis Result	Average Crustal Abundance	Geochemical Abundance Index
Silver	ppm	0.45	0	2
Aluminium	ppm	82890	82000	0
Arsenic	ppm	<2	2	0
Boron	ppm	<50	10	2
Barium	ppm	334	500	0
Beryllium	ppm	2.7	3	0
Calcium	ppm	34771	41000	0
Cadmium	ppm	0.1	0.1	0
Cobalt	ppm	4.1	20	0
Chromium	ppm	159	100	0
Copper	ppm	48	50	0
Fluorine	ppm	976	950	0
Iron	ppm	14800	41000	0
Mercury	ppm	0.1	0.1	0
Potassium	ppm	19222	21000	0
Magnesium	ppm	6298	23000	0
Manganese	ppm	630	950	0
Molybdenum	ppm	25	2	3
Sodium	ppm	30025	23000	0
Nickel	ppm	125	80	0
Phosphorus	ppm	712	1000	0
Lead	ppm	6	14	0
Antimony	ppm	3.8	0.2	4
Selenium	ppm	0.06	0.1	0
Tin	ppm	3.3	2	0
Uranium	ppm	3.13	2	0
Vanadium	ppm	88	160	0
Zinc	ppm	34	190	0

significantly enriched
slightly enriched

Table 4.16: Tailings solid multi-element results and site contamination guidelines

Element	Ecological Investigation - Interim Urban[1] (ppm)	Health Investigation Levels - Category A[1] (ppm)	Intervention Values for Soil [2] (ppm)	Multi-Element Analysis Result (ppm)
Antimony			15	3.8
Arsenic	20	100	55	<2
Barium	300		625	333.9
Beryllium		20	30	2.7
Boron		3000		<50
Cadmium	3	20	12	<0.1
Chromium (III)	400	12%		N/D
Chromium (VI)	1	100		N/D
Chromium (total)			380	159
Cobalt		100	240	4.1
Copper	100	1000	190	48
Lead	600	300	530	6
Manganese	500	1500		630
Methyl Mercury		10		N/D
Mercury (inorganic)	1	15	10	0.1
Molybdenum			200	25
Nickel	60	600	210	125
Phosphorus	2000			712
Selenium			100	0.06
Silver			15	0.45
Sulphur	600			950
Sulphate	2000			100
Tin			900	3.3
Thallium			15	N/D
Vanadium	50		250	88
Zinc			720	34

1 = National Environmental Protection Council – National Environmental Protection Measures – Soil Investigation levels for assessment of site contamination

2 = European Soil Intervention Levels.

Supernatant Water Quality

The supernatant water quality was assessed to examine the solubility of the various parameters which will occur when the ore is processed within the process plant. The results of the testing give an indication of the water quality which is likely within the supernatant pond during operation, but cannot be used to predict long term seepage quality from the facility (this would require kinetic testing of the tailings solids). However, based on the fact that the tailings have very low sulphur content, major changes in water chemistry as the tailings weather are not anticipated.

The results of the supernatant testing are presented in Table 4.17 and have been compared to the reference water quality standard for release of water from mining operations and livestock drinking water as detailed in Table 4.9. The supernatant quality meets the guidelines for release and for livestock drinking water for all parameters analysed. It is not proposed that supernatant will be released from the facility. Likewise, if stock or wildlife gain access to the facility they should not be adversely affected if they drink the process water. However, it is recommended that access is restricted by fencing of the facility.

Although not directly relevant to the Dargues Reef Gold Project the tailings supernatant has been assessed based on *"Technical guidelines for the environmental management of exploration and mining in Queensland"* (Ref. 24). These guidelines contain a useful assessment tool for assessing the liner requirements for a tailings storage facility based on the supernatant water quality. These guidelines recommend that the supernatant water is compared to drinking water standard and classified based on the criteria shown in Table 4.18, and provide guidance on the general liner types to be considered based on water quality and presence or not of significant groundwater resources which are exploited in proximity to the site.

Table 4.17: Tailings supernatant and comparison to release and livestock guidelines

Parameter	Reference Value (mg/L)	Assay results (mg/L)	Exceedance of Reference (%)
pH	6 to 9	7.8	-
TDS	2000	630	-
Aluminum	5	0.16	-
Antimony	N/G	0.035	N/G
Arsenic	0.1	0.001	-
Barium	N/G	0.098	N/G
Boron	5	0.12	-
Cadmium	0.01	0.00012	-
Calcium	1000	55.88	-
Chloride	N/G	157.5	N/G
Chromium (total)	1	<0.01	-
Cobalt	1	0.0002	-
Copper	0.3	0.01	-
Fluoride	2	1	-
Iron	2	0.22	-
Lead	0.1	<0.0005	-
Magnesium	2000	14.12	-
Manganese	N/G	0.16	N/G
Mercury	0.002	<0.0001	-
Molybdenum	0.15	0.01	-
Nickel	0.5	<0.01	-
Phosphorus	N/G	<0.1	N/G
Selenium	0.02	0.0016	-
Silver	0.5	0.00001	-
Sodium	N/G	137	N/G
Sulphate	1000	115.1	-
Tin	N/G	0.0036	N/G
Uranium	0.2	0.028	-
Vanadium	N/G	<0.01	N/G
Zinc	0.5	0.015	-

N/G – no guideline

Table 4.18: Supernatant water classification system and liner requirements

Supernatant Class	Concentration of contaminating substances	Liner Requirements	
		Significant Groundwater Area	No Significant Groundwater
Non toxic	< drinking water	No specific requirements	
Low Toxicity	<10 x drinking water	Soil liner or proven depth of low permeability soils	
Sub Lethal	10 – 100 x drinking water	Double liner	Soil Liner
Toxic	>100 x drinking water	Double liner with leak collection	Soil liner

The results of the comparison to drinking water standards are presented in Table 4.19 and indicate that the tailings supernatant should be classified as sub-lethal based on the high concentration of Antimony in the water. Table 4.18 indicates that the tailings storage facility should incorporate a soil liner if no significant groundwater resources are present in the vicinity of the facility. A double liner would be required if significant groundwater resources were present in the vicinity of the facility which may be impacted by seepage from the facility.

Project approval conditions stipulated by the Planning Assessment Commission of New South Wales state that "The Proponent shall ensure that walls, floor and final capping of the tailings storage facility is designed to be equivalent to 600 mm of clay or permeability 1 x 10^{-8} m/s".

Table 4.19: Supernatant water classification based on drinking water standards

Parameter	Reference Value (mg/L)	Assay results (mg/L)	No of Times Drinking Water Standard	Toxicity Class
pH	6.5 to 8.5	7.8	<	-
TDS	500	630	1.3	Low Toxicity
Aluminum	0.2	0.16	<	-
Antimony	0.003	0.035	11.5	Sub-lethal
Arsenic	0.007	0.001	<	-
Barium	0.7	0.098	<	-
Boron	4	0.12	<	-
Cadmium	0.002	0.00012	<	-
Calcium	N/G	55.88	N/G	-
Chloride	250	157.5	<	-
Chromium (total)	0.05	<0.01	<	-
Cobalt	N/G	0.0002	N/G	-
Copper	1	0.01	<	-
Fluoride	1.5	1	<	-
Iron	0.3	0.22	<	-
Lead	0.01	<0.0005	<	-
Magnesium	N/G	14.12	N/G	-
Manganese	0.1	0.16	1.6	Low Toxicity
Mercury	0.001	<0.0001	<	-
Molybdenum	0.05	0.01	<	-
Nickel	0.02	<0.01	<	-
Phosphorus	N/G	<0.1	N/G	-
Selenium	0.01	0.0016	<	-
Silver	0.1	0.00001	<	-
Sodium	180	137	<	-
Sulphate	250	115.1	<	-
Tin	N/G	0.0036	N/G	-
Uranium	0.02	0.028	1.4	Low Toxicity
Vanadium	N/G	<0.01	N/G	-
Zinc	3	0.015	<	-

< Indicates assay results below drinking water guidelines and therefore non toxic
N/G – no guideline

5. WATER MANAGEMENT

5.1 GENERAL

Management of water for the project site is critical in terms of the TSF design and decant return water pumping requirements. A site water management model was developed in order to understand and control the flow of water around the site and to determine design embankment crest levels to cater for extreme storm events.

Water management of the TSF consists of three major components:

- Tailings storage facility.
- External stormwater runoff.
- Plant site.

The model uses the design tailings throughput together with estimated settled tailings densities to determine the tailings level at various stages in the facility life. The model then examines a range of extreme rainfall events to determine supernatant pond volumes and the required embankment stage crest levels. A range of extreme dry rainfall events was also analysed to determine the water shortfall that could potentially occur.

The model was run on a monthly time-step for the duration of the operating life. Modelled flows do not represent the design duties for pumps and pipelines or peak flows for rainfall as they are averaged over the month and do not take into account efficiency and availability of the infrastructure.

5.2 WATER BALANCE MODELLING PARAMETERS

5.2.1 General

The water management model requires a number of input parameters. The following sub-sections outline the selection of parameters used for the water management modelling.

5.2.2 Climatic Conditions

Climatic data for the site was obtained from the Australian Bureau of Meteorology (BOM) *"Climatic Atlas of Australia"* (Ref. 25). The rainfall data used is from the Braidwood weather station which is located approximately 14 km south of the town of Braidwood. Climate data is available from 1920 to 2009 with some minor gaps. The data was analysed and design monthly rainfall parameters were generated as summarised in Table 5.1.

Table 5.1: Summary of rainfall data

Month	Average Rainfall (mm)	1 in 100 AEP\1 Wet Year (mm)	1 in 100 AEP Dry Year (mm)
January	65	91	9
February	43	69	78
March	64	261	69
April	38	300	23
May	48	164	17
June	52	42	14
July	63	104	6
August	80	49	19
September	54	71	24
October	70	19	8
November	74	260	10
December	73	140	48
Total	724	1570	326

\1 AEP = Annual Exceedance Probability

The evaporation data utilised is presented in Table 5.2.

Table 5.2: Evaporation data

Month	Average Evaporation (mm)
January	230
February	180
March	150
April	100
May	80
June	65
July	80
August	90
September	110
October	130
November	165
December	235
Total	1615

Precipitation intensity-duration-frequency (IDF) data was derived for the Dargues Reef site using procedures given in *"Australian Rainfall and Runoff, Volume 1 – A Guide to Flood Estimation"* (ARR) (Ref. 26) for Frequent to Large storms. IDF data for Rare to Extreme storms was derived using storm interpolation procedures given in ARR between the 1:100 AEP storm and the Probable Maximum Precipitation (PMP) storm event. PMP was estimated using procedures given in *"The Estimation of Probable Maximum Precipitation in Australia; Generalised Short-Duration Method (GSDM)"* (Ref. 27) and *"Generalised Southeast Australia Method (GSAM) for Estimating Probable Maximum Precipitation"* (Ref. 28). A summary of resulting IDF data is presented in Table 5.3, and IDF curves are shown in figures 5.1 and 5.2.

Table 5.3: Storm intensity-duration-frequency data

Storm Category	Storm Frequency		Point Rainfall Intensity (mm/h) for given Storm Duration				
	Return Period (yrs)	AEP %	6 min	1 h	12 h	24 h	72 h
Frequent	5	20	116	38	9.3	6.1	3.0
	10	10	133	43	11	7.1	3.5
	20	5	155	51	13	8.4	4.2
Large	50	2	185	60	15	10	5.2
	100	1	209	68	17	12	6.1
Rare	200	0.5		78	20	14	7.0
	500	0.2		94	24	16	8.6
	1,000	0.1		107	28	19	10
	2,000	0.05		121	31	21	11
Extreme	10,000	0.01		156	41	27	13
	50,000	2E-03		199	52	34	16
	200,000	5E-04		241	64	40	18
PMP	10,000,000	1E-05		360	96	58	24

5.2.3 Runoff Coefficients

The area around the facility is cleared ground formerly used for agricultural purposes. The adopted runoff coefficients used in the modelling for various ground surface conditions were calculated using the rational method in accordance with the guidelines given in ARR. The runoff coefficients used for water balance modelling are presented in Table 5.4.

Table 5.4: Adopted runoff coefficients

Condition	Runoff Coefficient
Undisturbed Bush	0.09
Cleared Agricultural Land	0.2
Topsoil Stripped Areas within Basin	0.5
Drying Tailings Beach	0.8
Active Tailings Beach (Supernatant Producing Areas)	1.0
Ponds	1.0

5.2.4 Tailings Beach Slope

The viscous nature of the tailings and high slurry density means that the tailings flow will generally be laminar with minimal segregation of material. The adopted beach slope used for design is 1.25% ±0.4%, based on observed tailings beach slopes at other sites and calculations from the viscosity data.

5.2.5 Additional Modelling Parameters

The tailings slurry design parameters are provided in Section 1.4. The modelling parameters such as tailings properties and facility design characteristics are discussed elsewhere.

5.3 TSF WATER BALANCE

5.3.1 Model

The TSF water balance has been modelled using specially developed computer software. The program is a computer model written in Visual Basic/Excel specifically for tailings storage facilities and incorporates a database of information derived from both laboratory and field data accumulated over the past 20 years by KP Australia. The program calculates tailings densities achieved in the storage, and determines the volume of water available for return to the process plant taking into account rainfall, evaporation, and supernatant and underdrainage release from the tailings due to consolidation.

5.3.2 Modelling Runs

The model was run under average climatic conditions. In addition, the effects of 1 in 100 Annual Exceedance Probability (AEP) wet and dry years were assessed. The effects of storm events on the facility were also examined.

5.3.3 Results of Modelling Runs

Four different conditions were modelled as follows:

5.3.3.1 Average Climatic Conditions

The model was run with a repeating sequence of average conditions. The estimated water balance for average conditions is summarised on a monthly basis in Table 5.5. The plots of tailings density and rate of rise are presented in figures 5.3 and 5.4. Pond volume and percent recycle are plotted on Figure 5.5.

Based on the modelling the following conclusions can be made:

- The tailings storage facility operates with a water deficit under average conditions. The pond remains at or close to minimum pond size (specified in the modelling as 5,000 m^3). The make-up water required in Year 1 is approximately 81% of the initial water in the slurry, which ranges from 4,200 m^3 to 6,400 m^3 per month.
- The recycle from the TSF back to the Process Plant in Year 1 varies from 0% to 34% of water in slurry and from 0% to 43% in Year 2. The average recycle volume is 28% of the water in slurry. The supernatant contributes approximately 22% of this volume with rainfall providing the remaining 78%. The low rate of recycle is due to the low supernatant release as a result of the high percent solids of the tailings and the high evaporation losses relative to rainfall.
- The initial tailings dry density is approximately 1.33 t/m^3. The average settled dry density gradually increases as a result of consolidation of the underlying tailings, achieving a final average density of approximately 1.41 t/m^3.

Table 5.5: Water balance – Average conditions

Year	Month	Rainfall	Evaporation	Water in Slurry	Supernatant Runoff	Rainfall Runoff	Evaporation Losses	Pond Volume	Consolidation Volume	Available TSF Recycle		Discharge	Make Up Requirement
		mm	mm	m³	m³	m³	m³	m³	m³	m³	%	m³	m³
	Jan-13	65	230	0	0	0	0	0	0	0			0
	Feb-13	43	180	6373	892	806	429	5363	0	0	0		6373
	Mar-13	64	150	6373	892	1453	856	5251	0	1602	25		4771
	Apr-13	38	100	6373	892	964	784	5190	0	1133	18		5241
	May-13	48	80	6373	892	1312	779	5234	0	1380	22		4993
1	Jun-13	52	65	6373	892	1504	724	5247	0	1659	26		4714
	Jul-13	63	80	6373	892	1877	994	5207	0	1815	28		4558
	Aug-13	80	90	6373	892	2384	1144	5170	1	2171	34		4203
	Sep-13	54	110	6373	892	1597	1407	5060	17	1210	19		5163
	Oct-13	70	130	6373	873	2142	1675	5041	29	1388	22		4985
	Nov-13	74	165	6373	843	2330	2172	5000	34	1076	17		5297
	Dec-13	73	235	6373	801	2368	3219	5000	36	0	0		6373
	Jan-14	65	230	6373	788	2165	3138	5000	42	0	0		6373
	Feb-14	43	180	6373	790	1448	2379	5000	45	0	0		6373
	Mar-14	64	150	6373	807	2214	1946	5000	37	1111	17		5262
	Apr-14	38	100	6373	828	1354	1252	5020	37	948	15		5426
	May-14	48	80	6373	839	1734	990	5053	32	1582	25		4791
2	Jun-14	52	65	6373	845	1919	798	5071	25	1973	31		4400
	Jul-14	63	80	6373	833	2375	994	5073	22	2234	35		4140
	Aug-14	80	90	6373	823	3065	1128	5086	18	2764	43		3609
	Sep-14	54	110	6373	801	2094	1396	5019	16	1582	25		4791
	Oct-14	70	130	6373	784	2768	1666	5005	15	1914	30		4460
	Nov-14	74	165	6373	746	2946	2162	5000	14	1549	24		4825
	Dec-14	73	235	6373	685	2952	3210	5000	13	440	7		5934
	Jan-15	65	230	6373	685	2674	3131	5000	14	243	4		6131
	Feb-15	43	180	6373	709	1776	2374	5000	19	130	2		6243
	Mar-15	64	150	6373	751	2698	1944	5000	17	1522	24		4851
	Apr-15	38	100	6373	793	1638	1251	5004	14	1190	19		5183
	May-15	48	80	6373	814	2082	988	5043	13	1882	30		4491
3	Jun-15	52	65	6373	826	2286	797	5064	15	2309	36		4064
	Jul-15	63	80	6373	812	2813	993	5065	13	2644	41		3729
	Aug-15	80	90	6373	800	3617	1127	5077	12	3289	52		3084
	Sep-15	54	110	6373	774	2463	1394	5008	10	1923	30		4450
	Oct-15	70	130	6373	755	3247	1664	5000	9	2355	37		4019
	Nov-15	74	165	6373	711	3447	2159	5000	9	2007	31		4367
	Dec-15	73	235	6373	639	3446	3207	5000	8	887	14		5487
	Jan-16	65	230	6373	642	3114	3128	5000	8	636	10		5737
	Feb-16	43	180	6373	673	2061	2372	5000	7	370	6		6003
	Mar-16	64	150	6373	726	3121	1943	5000	7	1911	30		4462
	Apr-16	38	100	6373	777	1888	1250	5000	6	1421	22		4953
	May-16	48	80	6373	802	2392	987	5038	6	2175	34		4199
4	Jun-16	52	65	6373	816	2619	796	5060	6	2622	41		3751
	Jul-16	63	80	6373	801	3208	993	5060	5	3021	47		3352
	Aug-16	80	90	6373	789	4103	1126	5073	5	3758	59		2616
	Sep-16	54	110	6373	761	2780	1393	5002	5	2223	35		4150
	Oct-16	70	130	6373	741	3649	1663	5000	4	2733	43		3640
	Nov-16	74	165	6373	692	3866	2159	5000	4	2403	38		3970
	Dec-16	73	235	6373	615	3864	3205	5000	4	1277	20		5096
	Jan-17	65	230	6373	619	3490	3126	5000	3	986	15		5387
	Feb-17	43	180	6373	653	2310	2371	5000	3	596	9		5777
	Mar-17	64	150	6373	711	3497	1942	5000	3	2269	36		4104
	Apr-17	38	100	6373	767	2115	1250	5000	3	1635	26		4739
	May-17	48	80	6373	795	2678	987	5035	3	2453	38		3920
5	Jun-17	52	65	6373	810	2932	796	5058	3	2926	46		3447
	Jul-17	63	80	6373	794	3597	992	5057	3	3401	53		2972
	Aug-17	80	90	6373	781	4611	1126	5070	2	4255	67		2118
	Sep-17	54	110	6373	751	3124	1393	5000	2	2554	40		3819
	Oct-17	70	130	6373	730	4091	1662	5000	2	3161	50		3212
	Nov-17	74	165	6373	678	4319	2158	5000	2	2842	45		3532
	Dec-17	73	235	6373	597	4294	3204	5000	2	1689	27		4684
	Jan-18	65	230	6373	602	3858	3125	5000	2	1338	21		5035
6	Feb-18	43	180	6373	640	2540	2370	5000	2	813	13		5560
	Mar-18	64	150	6373	702	3827	1941	5000	2	2590	41		3784
	Apr-18	38	100	6373	761	2308	1250	5000	2	1821	29		4552

KP_svr\...\PE801-139_5 Dargues Reef TSF Final Design Rev 0

5.3.3.2 1 in 100 AEP Wet Sequence

Effects of a 1 in 100 AEP Wet year were analysed by inserting a wet year independently into each year of the model. As the pond level can return to a minimum each year, the water balance impact is independent of the previous year's rainfall.

The maximum pond volume of 26,000 m^3 was generated by inserting a 1 in 100 AEP Wet year towards the end of the operation as shown in Table 5.6. The storage volume available on the tailings without encroaching on the embankment at that time is 150,000 m^3 and the maximum pond level for the 1 in 100 AEP Wet year precipitation is only 17% of the capacity available on the tailings. The maximum recycle rate of 85% of water in slurry occurs for 4 months beyond the end of the Wet year. No spillway flows are expected under these conditions. The size of the pond and the effect on the recycle rate are shown in Figure 5.5.

5.3.3.3 Storm Events

The design elevation of the TSF embankment is a function of the required storm capacity of the facility in excess of the tailings beach level. DSC guideline DSC3F was noted as defining various freeboard requirements related to the flood handling capacity of the facility. A rainfall-runoff model, created using *"Hydrologic Modeling System HEC-HMS, Version 3.4"* (HEC-HMS) (Ref. 29), was employed to model various storm scenarios for the purpose of verifying that the proposed design meets DSC3F freeboard criteria. These are discussed below:

- Beach Freeboard – runoff (volume) from 1:10 AEP, 72 hour and 1:100 AEP, 72 hour storm events were added to the decant pond, starting with the maximum pond operating volume under average climatic conditions at a reduced level of RL709.0 m. The resulting pond volume (116,410 m^3) corresponds to a predicted pond level of 710.6 m which lies *below* the predicted tailings level of RL711.0 m. Accordingly, beach freeboard is satisfied.
- Pond Recovery Time – the design of the decant pumping system falls outside of KP's design scope. However, it is recommended that the decant pumping system is designed such that the 1:100 AEP, 72 hour storm event can be removed within 14 days. The 1:100 AEP, 72 hour storm run-off added to the maximum pond operating volume under average climatic conditions gives a resulting total decant pond volume of 86,810 m^3, which corresponds to an Operational Pond Limit of RL710.2 m.
- Operational Freeboard – this is the vertical distance between the top of the tailings (RL711.0 m) and the adjacent embankment crest (RL712.0 m). The

minimum recommended value suggested by DSC3F is 300 mm. The TSF design provides 1,000 mm of Operational Freeboard, far exceeding the specified minimum value.

- Environmental Containment Freeboard – this is the vertical distance between the Operational Pond Limit (RL710.2 m) and the pond level resulting from a 1:10 AEP, 72 hour storm. The modelling shows that the resulting pond level is RL710.6 m which lies *below* the proposed final stage spillway level (RL711.5 m). Consequently, Environmental Containment Freeboard is satisfied.
- Total Freeboard – this is the vertical distance between the Operational Pond Limit (RL710.2 m) and the crest of the embankment (RL712.0 m). The design storm event for a *Significant* consequence category is the 1:10,000 AEP, critical duration storm. Starting with the maximum pond operating level under average climatic conditions (RL709.0 m), the peak decant pond level during passage of a 1:10,000 AEP, 72 hour storm is 711.2 m. Accordingly, Total Freeboard is satisfied and without discharge from the emergency spillway.

5.3.3.4 1 in 100 AEP Dry Sequence

The results for the 1 in 100 AEP Dry year simulations are summarised in Table 5.7. The table summarises the results of multiple individual modelling runs for a single 1 in 100 AEP Dry event. Each year is independent, as the pond level reverts to its minimum volume each year, allowing multiple individual modelling runs for the Dry event to be carried out without impacting on one another. The 1 in 100 AEP Dry year precipitation is 326 mm. The average recycle volume under 1 in 100 AEP Dry year conditions is 10% of the water in slurry, which yields 370,000 m^3 shortfall in total, ranging from 3,500 m^3 and 6,400 m^3 per month during the operation.

There may be periods of several months when no water return should be expected from the TSF and during which time all process water will have to be supplied from alternative sources.

Table 5.6: Water balance – 1 in 100 AEP Wet conditions

Year	Month	Rainfall	Evaporation	Water in Slurry	Supernatant Runoff	Rainfall Runoff	Evaporation Losses	Pond Volume	Consolidation Volume	Available TSF Recycle		Discharge	Make Up Requirement
		mm	mm	m³	m³	m³	m³	m³	m³	m³	%	m³	m³
1	Jan-13	65	230	0	0	0	0	0	0	0			0
	Feb-13	43	180	6373	892	806	429	5363	0	0	0		6373
	Mar-13	64	150	6373	892	1453	856	5251	0	0	0		6373
	Apr-13	38	100	6373	892	964	784	5190	0	0	0		6373
	May-13	48	80	6373	892	1312	779	5234	0	0	0		6373
	Jun-13	52	65	6373	892	1504	724	5247	0	0	0		6373
	Jul-13	63	80	6373	892	1877	994	5207	0	0	0		6373
	Aug-13	80	90	6373	892	2384	1144	5170	1	0	0		6373
	Sep-13	54	110	6373	892	1597	1407	5060	17	0	0		6373
	Oct-13	70	130	6373	873	2142	1675	5041	29	1919	30		4454
	Nov-13	74	165	6373	843	2330	2172	5000	34	1573	25		4801
	Dec-13	73	235	6373	801	2368	3219	5000	36	722	11		5651
2	Jan-14	65	230	6373	788	2165	3138	5000	42	397	6		5976
	Feb-14	43	180	6373	790	1448	2379	5000	45	175	3		6198
	Mar-14	64	150	6373	807	2214	1946	5000	37	1045	16		5329
	Apr-14	38	100	6373	828	1354	1252	5020	37	881	14		5492
	May-14	48	80	6373	839	1734	990	5053	32	1521	24		4853
	Jun-14	52	65	6373	845	1919	798	5071	25	1923	30		4450
	Jul-14	63	80	6373	833	2375	994	5073	22	2193	34		4181
	Aug-14	80	90	6373	823	3065	1128	5086	18	2742	43		3632
	Sep-14	54	110	6373	801	2094	1396	5019	16	1600	25		4773
	Oct-14	70	130	6373	784	2768	1666	5005	15	1959	31		4414
	Nov-14	74	165	6373	746	2946	2162	5000	14	1617	25		4756
	Dec-14	73	235	6373	685	2952	3210	5000	13	517	8		5856
3	Jan-15	65	230	6373	685	2674	3131	5000	14	321	5		6052
	Feb-15	43	180	6373	709	1776	2374	5000	19	189	3		6184
	Mar-15	64	150	6373	751	2698	1944	5000	17	1606	25		4767
	Apr-15	38	100	6373	793	1638	1251	5004	14	1255	20		5119
	May-15	48	80	6373	814	2082	988	5043	13	1963	31		4410
	Jun-15	52	65	6373	826	2286	797	5064	15	2397	38		3976
	Jul-15	63	80	6373	812	2813	993	5065	13	2755	43		3619
	Aug-15	80	90	6373	800	3617	1127	5077	12	3434	54		2939
	Sep-15	54	110	6373	774	2463	1394	5008	10	2029	32		4344
	Oct-15	70	130	6373	755	3247	1664	5000	9	2500	39		3874
	Nov-15	74	165	6373	711	3447	2159	5000	9	2169	34		4205
	Dec-15	73	235	6373	639	3446	3207	5000	8	1057	17		5316
4	Jan-16	65	230	6373	642	3114	3128	5000	8	798	13		5575
	Feb-16	43	180	6373	673	2061	2372	5000	7	483	8		5890
	Mar-16	64	150	6373	726	3121	1943	5000	7	2084	33		4289
	Apr-16	38	100	6373	777	1888	1250	5000	6	1529	24		4845
	May-16	48	80	6373	802	2392	987	5038	6	2314	36		4059
	Jun-16	52	65	6373	816	2619	796	5060	6	2778	44		3596
	Jul-16	63	80	6373	801	3208	993	5060	5	3220	51		3153
	Aug-16	80	90	6373	789	4103	1126	5073	5	4026	63		2348
	Sep-16	54	110	6373	761	2780	1393	5002	5	2414	38		3959
	Oct-16	70	130	6373	741	3649	1663	5000	4	2994	47		3379
	Nov-16	74	165	6373	692	3866	2159	5000	4	2686	42		3687
	Dec-16	73	235	6373	615	3864	3205	5000	4	1557	24		4816
5	Jan-17	91	230	6373	619	4865	3132	5000	3	1237	19		5136
	Feb-17	69	180	6373	653	3722	2376	5000	3	761	12		5613
	Mar-17	261	150	6373	741	14614	2670	12271	3	2517	39		3856
	Apr-17	300	100	6373	829	17790	2767	22709	3	1784	28		4589
	May-17	164	80	6373	863	10142	2741	25556	0	2643	41		3730
	Jun-17	42	65	6373	865	2632	2176	21460	0	2546	40		3827
	Jul-17	104	80	6373	851	6443	2516	20820	0	5417	85		956
	Aug-17	49	90	6373	837	3045	2630	16655	0	4483	70		1890
	Sep-17	71	110	6373	805	4376	2827	13592	0	4290	67		2083
	Oct-17	19	130	6373	765	1145	2667	7418	0	435	7		5939
	Nov-17	260	165	6373	729	15825	3532	15023	0	5417	85		956
	Dec-17	140	235	6373	695	8699	6145	12856	2	5417	85		956
6	Jan-18	65	230	6373	658	3982	4896	7183	0	5417	85		956
	Feb-18	43	180	6373	652	2561	2689	5000	0	5417	85		956
	Mar-18	64	150	6373	702	3831	1941	5000	3	5417	85		956
	Apr-18	38	100	6373	761	2311	1250	5000	5	5417	85		956

Table 5.7: Water balance – 1 in 100 AEP Dry conditions

Year	Month	Rainfall	Evaporation	Water in Slurry	Supernatant Runoff	Rainfall Runoff	Evaporation Losses	Pond Volume	Consolidation Volume	Available TSF Recycle		Discharge	Make Up Requirement
		mm	mm	m³	m³	m³	m³	m³	m³	m³	%	m³	m³
1	Jan-13	0	230	0	0	0	0	0	0	0			0
	Feb-13	78	180	6373	892	1481	430	5396	0	0	0		6373
	Mar-13	69	150	6373	892	1562	856	5262	0	1732	27		4641
	Apr-13	23	100	6373	892	589	784	5141	0	817	13		5557
	May-13	17	80	6373	892	477	779	5114	0	618	10		5756
	Jun-13	14	65	6373	892	392	724	5089	0	585	9		5788
	Jul-13	6	80	6373	892	181	991	5013	0	157	2		6216
	Aug-13	19	90	6373	892	562	1120	5034	1	314	5		6059
	Sep-13	24	110	6373	892	722	1391	5017	17	257	4		6116
	Oct-13	8	130	6373	872	241	1661	5000	29	0	0		6373
	Nov-13	10	165	6373	842	307	2155	5000	34	0	0		6373
	Dec-13	48	235	6373	801	1559	3213	5000	36	0	0		6373
2	Jan-14	9	230	6373	788	293	3125	5000	42	0	0		6373
	Feb-14	78	180	6373	790	2661	2385	5000	45	1110	17		5263
	Mar-14	69	150	6373	807	2378	1947	5000	37	1275	20		5098
	Apr-14	23	100	6373	828	827	1250	5000	36	441	7		5933
	May-14	17	80	6373	839	630	986	5008	32	507	8		5866
	Jun-14	14	65	6373	844	501	792	5014	25	572	9		5801
	Jul-14	6	80	6373	832	230	985	5000	22	113	2		6261
	Aug-14	19	90	6373	822	729	1117	5000	18	452	7		5921
	Sep-14	24	110	6373	800	950	1384	5000	16	382	6		5991
	Oct-14	8	130	6373	783	312	1656	5000	15	0	0		6373
	Nov-14	10	165	6373	746	388	2150	5000	14	0	0		6373
	Dec-14	48	235	6373	685	1944	3204	5000	12	0	0		6373
3	Jan-15	9	230	6373	685	362	3117	5000	13	0	0		6373
	Feb-15	78	180	6373	709	3264	2381	5000	18	1610	25		4763
	Mar-15	69	150	6373	751	2899	1945	5000	15	1720	27		4653
	Apr-15	23	100	6373	793	1000	1249	5000	14	559	9		5815
	May-15	17	80	6373	814	757	985	5000	13	599	9		5775
	Jun-15	14	65	6373	825	597	792	5006	15	640	10		5733
	Jul-15	6	80	6373	811	272	984	5000	13	118	2		6255
	Aug-15	19	90	6373	800	860	1116	5000	12	556	9		5817
	Sep-15	24	110	6373	774	1118	1383	5000	10	519	8		5855
	Oct-15	8	130	6373	755	366	1654	5000	9	0	0		6373
	Nov-15	10	165	6373	711	454	2148	5000	9	0	0		6373
	Dec-15	48	235	6373	639	2269	3201	5000	8	0	0		6373
4	Jan-16	9	230	6373	642	421	3114	5000	8	0	0		6373
	Feb-16	78	180	6373	673	3788	2379	5000	7	2090	33		4284
	Mar-16	69	150	6373	726	3352	1943	5000	7	2142	34		4231
	Apr-16	23	100	6373	777	1153	1249	5000	6	687	11		5686
	May-16	17	80	6373	802	870	985	5000	6	693	11		5681
	Jun-16	14	65	6373	816	684	791	5001	6	713	11		5661
	Jul-16	6	80	6373	801	311	984	5000	5	134	2		6239
	Aug-16	19	90	6373	788	976	1116	5000	5	654	10		5719
	Sep-16	24	110	6373	760	1262	1383	5000	5	645	10		5729
	Oct-16	8	130	6373	741	411	1654	5000	4	0	0		6373
	Nov-16	10	165	6373	692	509	2147	5000	4	0	0		6373
	Dec-16	48	235	6373	615	2544	3199	5000	4	0	0		6373
5	Jan-17	9	230	6373	619	472	3112	5000	3	0	0		6373
	Feb-17	78	180	6373	653	4245	2378	5000	3	2524	40		3850
	Mar-17	69	150	6373	711	3756	1943	5000	3	2527	40		3846
	Apr-17	23	100	6373	767	1292	1248	5000	3	813	13		5561
	May-17	17	80	6373	795	974	984	5000	3	787	12		5587
	Jun-17	14	65	6373	810	766	791	5000	3	787	12		5586
	Jul-17	6	80	6373	793	348	983	5000	3	161	3		6212
	Aug-17	19	90	6373	780	1097	1115	5000	2	764	12		5609
	Sep-17	24	110	6373	750	1418	1383	5000	2	788	12		5585
	Oct-17	8	130	6373	730	461	1653	5000	2	0	0		6373
	Nov-17	10	165	6373	678	569	2146	5000	2	0	0		6373
	Dec-17	48	235	6373	597	2827	3198	5000	2	229	4		6144
6	Jan-18	9	230	6373	602	522	3111	5000	2	0	0		6373
	Feb-18	78	180	6373	640	4667	2377	5000	2	2933	46		3440
	Mar-18	69	150	6373	702	4111	1942	5000	2	2873	45		3500
	Apr-18	23	100	6373	761	1410	1248	5000	2	924	15		5449

5.4 SURFACE WATER MANAGEMENT

Following discussions with the client it was agreed that the catchment area of the TSF would be kept as small as possible and on this basis catchment diversion channels would be utilised. The total catchment area reporting to the facility is approximately 20 Ha, and of this total approximately 8.5 Ha lies within the perimeter defined by the diversion channels. From a design perspective, it was assumed that the run-off from the upstream catchment is diverted during the various water balance simulations. For the storm storage and freeboard calculations, i.e. for extreme flood events, it was conservatively assumed that the diversion channels were not operational.

Run-off is expected to sheet across the landscape rather than form discrete watercourses; hence the diversion channels have been designed to intercept surface run-off along their entire lengths. The diversion channels will be trapezoidal in shape: 1 m deep, 3 m wide, with side slopes of 3:1 (H:V) and a channel gradient of 0.5%. The channels are sited to drain to existing natural drainage channels on either side of the TSF embankment abutments. The diversion channels are designed to have sufficient capacity to convey the peak runoff from a 1:100 AEP, critical duration (2 hours) storm event using the solution of Manning's equation for normal depth, with an additional freeboard allowance of 200 mm. The results of the peak flow estimation and critical duration determination for the areas contributing runoff to the two diversion channels are illustrated on figures 5.6 and 5.7.

The hydraulic results (critical velocities) predicted during passage of the 1:100 AEP, 2 hour storm indicate that erosion protection is not required within the diversion channels except at the respective outfalls where rip-rap could be justified. However, given the non-critical location of the diversion channel outfalls, no erosion protection is provided for, and any consequential erosion will be repaired as necessary.

A general arrangement of the proposed surface water management layout is shown in Drg. No PE801-00139-010. Sections and details of the diversion channels are presented in Drg. No PE801-00139-032.

6. TAILINGS FACILITY DESIGN

6.1 GENERAL DESCRIPTION

The facility will comprise a cross-valley storage with a zoned embankment. The design incorporates a basin underdrainage system to reduce seepage, and a toe drain located at the upstream toe to lower the phreatic surface adjacent to the embankment. The upstream toe drains and underdrainage system drain by gravity to a collection sump located at the upstream toe of the embankment. Supernatant water will be decanted from the facility via a decant tower located at the head of valley. Solution recovered from the underdrainage and decant systems will be pumped back to the plant for re-use in the process circuit. An emergency spillway will be constructed for each raise to control the discharge of any extreme storm events exceeding the design event.

Tailings will be discharged into the facility by sub-aerial deposition methods, via spigots spaced at regular intervals along the embankment crest, so as to maximise tailings density and evaporation of water. Deposition will occur mainly from the embankment towards the valley in order to form a supernatant pond towards the north-eastern perimeter.

The general layout and typical details of the TSF are shown on Drg. Nos. PE801-00139-005 and PE801-00139-010.

6.2 EMBANKMENT CONSTRUCTION

The TSF embankment will be constructed in three stages. Stage 1 will be constructed initially and will provide for the first 12 months of operation. The Stage 1 embankment will be constructed downstream.

After constructing the starter embankment, the facility will be raised in two stages, using modified centreline construction to achieve its final height. The typical embankment cross section is shown on Drg. No. PE801-00139-012. A more detailed description of the embankment is outlined below.

Embankment construction will comprise a zoned embankment constructed of selected local borrow. The embankment consists of an upstream low permeability zone (Zone A) and a downstream structural zone (Zone C). Typical material specifications for the embankment are summarised below:

- Zone A material will be selected local borrow with a hydraulic conductivity not greater than 1 x 10 ^{8} m/s. Zone A material will be won from the basin and will comprise extremely weathered granite.
- Zone C material will comprise mine waste from the box cut, supplemented by local borrow, if required. Sufficient mine waste will be available at Stage 1 to allow completion of the embankment to the final downstream toe. Zone C material for subsequent stages will comprise either mine waste, or material borrowed locally.

The initial embankment will have upstream and downstream slopes of 1V:3H with a crest width of 6 m. The same crest width will be adopted for the modified centreline embankment construction.

It is anticipated that the tailings will attain sufficient strength adjacent to the embankment to allow modified centreline construction for the subsequent stages. However, testing of the tailings adjacent to the embankment in Stage 1 will be required to confirm this assumption. It is expected that the tailings will not be suitable as a construction material and the design is based on all lifts being constructed using local borrow.

Construction of the stage raises will commence before the current stage is full so that there is adequate storage volume available throughout the life of mine and to minimise construction delays. A summary of the proposed embankment staging is provided in Table 6.1.

Table 6.1: Staged embankment construction

Stage	Duration (months)	Cumulative Production (t) [1]	Embankment Design Crest (mRL) [2]	Construction Schedule
1	12	170,400	701.0	February to June 2012
2	25	530,000	708.0	September 2013 to January 2014
Final	26	890,000	712.0	October 2015 to February 2016

Notes: 1. Production based on the Mining Plan issued in September 2010.
2. Embankment crest levels based on tailings beach slope of 1 in 80.

6.3 SEEPAGE CONTROL

In order to mitigate seepage losses through the basin area and increase the settled density of the deposited tailings, a number of seepage control and underdrainage collection features have been integrated into the design. The seepage control and underdrainage collection systems will consist of the following components:

i. Cut-off trench.

ii. Low permeability soil liner.

iii. Partial geosynthetic liner.

iv. Basin underdrainage collection system.

v. Underdrainage collection sump.

vi. Embankment upstream toe drain.

6.3.1 Cut-Off Trench

Primary seepage control from the tailings facility will comprise the construction of a cut-off trench excavated into the foundation soils and backfilled with low permeability fill to minimise seepage loss through the embankment foundation.

The cut-off trench will be located beneath the upstream toe of the embankment and will be cut to a depth of approximately 2 m – 3 m (depending on ground conditions). The cut-off trench will be constructed continuously along the upstream toe of the embankment to the full deposition elevation to limit potential seepage at any level. If the cut material is suitable as Zone A fill it may be replaced in the excavation in compacted layers; alternatively, suitable low permeability material will be won, conditioned, placed and compacted in the trench.

The location and details of the embankment cut-off trench are shown on Drg. No. PE801-00139-012.

6.3.2 Low Permeability Liner

The deeply incised creek will be widened to approximately 5 m width and the creek banks cut back to a slope of 1V:3H. The surplus material will be used for embankment construction. The basin will be compacted to form a low permeability soil liner to tie into the low permeability zone of the embankment. The liner will be constructed by scarifying the surface soils, moisture conditioning, and re-compacting to a target permeability of 1×10^{8} m/s. Some cross movement of material may be required for areas with insufficient clay material in the subgrade.

6.3.3 Partial Geosynthetic Liner

The excavated area along the creek alignment, the Stage 1 embankment upstream face, and the area under the decant pond will be lined with a 1.5 mm HDPE geomembrane liner in order to mitigate any seepage from the tailings and the supernatant pond. The extent of the liner will cover the maximum decant pond area determined from water management modelling for a 1 in 25 AEP, 24 hour storm event. The liner will be installed in Stage 1. Approximately 50% of the total basin area will be covered by HDPE liner.

The HDPE liner will be placed on top of the compacted soil liner forming a composite liner system. Smooth geomembrane will be utilised except at the location of the decant towers, where a textured geomembrane will be placed to provide additional stability to the causeway and decant towers.

Drg. No. PE801-00139-015 shows the proposed extent of the HDPE geomembrane.

6.3.4 Basin Underdrainage Collection System

The underdrainage collection system is designed to reduce the phreatic surface on the tailings basin area under the decant pond and immediately upstream of the embankment. The underdrainage has several benefits as follows:

- Minimises seepage through the basin and under/through the embankment;
- Drains the tailings mass, thus increasing the density of the tailings and providing a more efficient facility in terms of storage;
- Increases the strength of the tailings mass immediately adjacent to the embankment.

The design of the underdrainage system takes advantage of the natural fall of the ground and thus minimal re-shaping of the basin will be required. The underdrainage system will consist of three drainage networks, namely the main collector drains, branch drains and finger drains.

The collector drain will be constructed along the main drainage line. The drain will consist of a 7 m wide sand layer (Zone F) with a nominal thickness of 300 mm, with 4 no. 160 mm draincoil pipes running for the entire underdrainage length. The sand will be covered by an erosion protection layer (Zone D) of 150 mm thickness in order to minimise erosion losses and damage to the drains. The collector drain pipes will feed directly into the underdrainage collection sump located at the upstream toe of the embankment.

The branch drains will be constructed across the basin along the minor drainage lines. The finger drains will be constructed in the HDPE lined area at approximately 10 metre spacings. Both branch and finger drains will be of triangular profile, with a 100 mm draincoil pipe along the centreline and a sand layer (Zone F) wrapped by geotextile and will be held in place by welding HDPE straps to the geomembrane liner. The branch drains will feed directly to the collector drains and the finger drains will connect into the branch and collector drains.

The layout of the facility underdrainage system is shown on Drg. No. PE801-00139-015 and relevant sections and details are shown on Drg. Nos. PE801-00139-017 and PE801-00139-019.

6.3.5 Underdrainage Collection Sump

An underdrainage collection sump will be constructed against the upstream toe of the TSF embankment. This sump will collect solution from the toe drains and underdrainage system and consists of the following components:

- An excavated sump, filled with clean gravel wrapped in geotextile. The sump will be located on top of the geomembrane liner.
- A 450 mm diameter HDPE (SDR11) solid riser pipe, slotted only at the base. The pipe is located on top of the geomembrane liner (protected with a wearsheet) in a trench that runs up the upstream embankment face.
- A submersible pump.
- A hoist and pulley to raise and lower the pump.

The underdrainage system details are shown on Drg. No. PE801-00139-020.

6.3.6 Embankment Upstream Toe Drain

In addition to the basin drainage system, a toe drain will be constructed along the upstream toe of the embankment. The toe drain has two purposes. The main purpose is to increase the stability of the embankment by providing drainage of the tailings and hence lowering the phreatic surface adjacent to the embankment. The second purpose of the toe drain is to act as an underdrainage collection pipe.

The toe drain will be similar in design to the collector drains and will comprise a 160 mm draincoil pipe laid at the base of the drain within 300 mm of drainage material (Zone F) wrapped by geotextile. The toe drain will drain into the underdrainage collection sump for recycling back into the facility.

Details of the embankment toe drain are shown in Drg. No. PE801-00139-012.

6.4 ADDITIONAL SEEPAGE CONTROL MEASURES

The facility is designed with a number of different seepage control measures. If the designed seepage control measures do not provide sufficient seepage control, there are a number of additional seepage control measures which can be incorporated into the facility at a later stage. The two main additional seepage control measures are:

- Downstream seepage interception trench (0 - 5 m zone). The trenches can be either open or closed (i.e. backfilled with drainage material) with sumps to collect the seepage and return it into the facility.
- Water recovery bores – these are used to intercept seepage flows at depths greater than 5 m.

6.5 DECANT AND RETURN WATER SYSTEM

The TSF will operate with two decant towers, both located towards the top of the valley. An initial decant tower will be constructed for Stage 1. It is expected that it will take approximately 3 months for the tailings beach to develop sufficiently that the pond will come into contact with the Stage 1 decant tower, at which stage it can become operational and water can be returned to the plant. The Stage 1 decant tower will become redundant later in the life of the TSF as the pond level rises and migrates further up the valley. The second decant tower will be constructed at the final stage and will operate for the remainder of the life of the facility. The decant towers will be raised as required with each embankment lift and will consist of the following components:

- An access causeway constructed of Zone C material;
- A slotted concrete decant tower consisting of a 1.8 m diameter slotted concrete pipe surrounded by clean waste rock (Zone G) with a minimum size of 100 mm;
- A submersible pump with float control switches mounted on a lifting hoist.

The decant pump will be raised on a regular basis to ensure that no tailings enters the pump intake.

The location of the decant towers are shown on Drg. No. PE801-00139-015 and sections and details are shown on Drg. No. PE801-00139-022.

6.6 EMERGENCY SPILLWAY

The tailings storage facility has been designed to completely contain storm events up to and including annual exceedance probabilities (AEP) of 1:10,000 on top of predicted maximum average conditions during operations (see Section 5.3.3.3). Consequently, exceeding the storm storage capacity of the facility at any stage of operation is highly unlikely. Regardless, in the event that the storage capacity of the facility is exceeded, water which cannot be stored within the facility will discharge via an engineered spillway. The emergency spillway during operation is designed to convey runoff from a 1:1,000 AEP critical duration storm, assuming that the decant pond level is at the spillway invert level at commencement of the storm event.

As part of closure of the facility, a permanent spillway will be constructed at the location of the final stage spillway. In order to size the closure spillway, a rainfall / runoff flood routing model was created using HEC-HMS. Sub-catchments contributing runoff to the TSF were de-lineated for use in this model and are shown in Figure 6.1. Additional key methods and inputs used with the emergency spillway model include:

- Precipitation inputs taken from the IDF curves developed for the site, as discussed in Section 5.2.2;
- Temporal distribution of design precipitation events estimating using methods and procedures as discussed in ARR;
- Elevation / storage / outflow rating curves corresponding to the end of operational lifetime conditions and closure conditions, respectively. Both curves were developed using the projected tailings surface results (for the elevation / storage portion of the curve) and an outflow rating curve computed using *"HEC-RAS River Analysis System, Version 4.1"* (HEC-RAS) (Ref. 30);
- Initial loss / continuing loss (IL / CL) model for calculating rainfall excess with parameters taken from ARR that vary according to AEP; and
- Transformation of rainfall excess to runoff hydrographs performed using the Clark synthetic hydrograph method, with times of concentration and basin storage coefficients assigned to each identified sub-catchment using relationships taken from ARR.

The HEC-HMS emergency spillway model was employed to develop full flood frequency curves during the last month of operations (when storm storage is at a minimum) and under closure conditions. Flood frequency curves express the peak predicted outflows (at various critical durations) for a range of potential annual exceedance probabilities. Flood frequency curves, as required by DSC3B for the aforementioned conditions, are presented on figures 6.2 and 6.3 along with selected critical duration diagrams that were used in their preparation, as shown on figures 6.4 and 6.5.

The flood frequency curve development results indicate the following:

- The 1:10,000 AEP, 72 hour storm results in a peak spillway outflow of 0.6 m^3/s at a peak pool RL of 711.2 m under operational conditions. This satisfies the minimum required embankment dam freeboard of 0.3 m for a *Significant* category facility as specified in DSC3B.
- During the last month of operations, a probable maximum precipitation design flood (PMPDF) results in a peak spillway outflow of 3.5 m^3/s at a peak decant pond level of RL 711.5 m.
- Procedures given in ARR do not allow for extrapolation of extreme rainfall for events less frequent than the PMP, thus the Dam Crest Flood (DCF) as required in DSC3B cannot be estimated under operational conditions.
- The 1:10,000 AEP, 2 hour storm results in a peak spillway outflow of 6.2 m^3/s at a peak pond level of RL711.7 m under closure conditions. This also satisfies the minimum required dam freeboard given in DSC3B.
- Under closure conditions, a PMPDF results in a peak outflow of 17.3 m^3/s at a peak decant pond level of RL712.0 m. This means that under closure conditions, the PMPDF and the DCF are synonymous.

A new spillway will be constructed at each stage of construction and will be excavated into the ridge directly to the east of the east end of the TSF embankment. The general layout and channel dimensions are shown in Drg. No. PE801-00139-030. The placement of channel revetment will be omitted during operation owing to the transient nature of the spillway. At closure, the spillway will be deepened, widened and extended into the facility, and channel revetment will be placed as shown. Under closure conditions the emergency spillway has sufficient capacity to control the discharge from a PMPDF (or DCF) without overtopping the TSF embankment.

6.7 TAILINGS AND DECANT RETURN TRENCH

The tailings delivery and decant return pipelines will be located within a bunded corridor between the process plant and the TSF in order to mitigate any spillage of tailings or decant water into the surrounding environment. Typical sections and details are shown in Drg. No. PE801-00139-035.

6.8 SEEPAGE ASSESSMENT

6.8.1 General

Seepage analyses were undertaken on the TSF to assess the following aspects of the design:

- Estimate the position of the phreatic surface within the embankments. This indicates how much of the embankment material could be saturated and is therefore a consideration for slope stability. A high phreatic surface (and consequent high pore water pressures) is a key consideration in the assessment of embankment stability.
- Estimate the total seepage losses from the TSF. It is common to estimate the maximum possible seepage loss by making conservative assumptions. This result has implications for the potential environmental impact of the TSF.
- Estimate the influence of the basin underdrainage system on the phreatic surface within the TSF. This modelling indicates how critical the underdrainage system is to the performance of the TSF and what the consequences would be if the underdrainage system were to fail.

6.8.2 Geometry

The seepage model used for this analysis was based on a south-west to north-east aligned long section through the TSF. The section was aligned with the main creek along the valley floor. The sub-surface conditions beneath the facility are based on the geotechnical information derived from the site investigations. Beneath the spine of the valley the layer of alluvial sand at the surface has some impact on the seepage flows. The depth of this alluvial layer varies along the creek, as does the underlying soil profile. Embankment zoning is based on the design symmetry shown in Drg. No. PE801-00139-012.

In the TSF basin there are two separate underdrainage systems. The first series of drains (referred to as the basin underdrainage system) is above the geosynthetic liner and reports to the underdrainage sump. This system extends along the creek to the sump located at the toe of the TSF embankment. The second series of drains (referred to as the seepage collection system) is located at the upstream toe of the TSF embankment. The full length of the upstream toe drain reports to the underdrainage sump.

The seepage analysis program Seep/W was used to evaluate seepage losses for the TSF.

6.8.3 Material Types and Properties

The assumed cross-section through the facility is illustrated in Figure 6.6 and the adopted materials properties of the facility are summarised in Table 6.2.

Table 6.2: Adopted material types and properties used in seepage model

Material Type	Permeability, k (m/s)	Source
Zone C – Structural Fill	1.0×10^{-7}	Assumed
Zone A – Low Permeability	3.0×10^{-8}	Assumed
Alluvial/Colluvium	5.0×10^{-5}	In Situ Field Data
HDPE Geomembrane*	1.0×10^{-11}	Specification
Compacted Soil Liner	3.0×10^{-8}	Assumed
Weathered Rock	8.0×10^{7}	In Situ Field Data
Granite	1.0×10^{9}	Assumed
Tailings	5.0×10^{-7}	Laboratory Data

* HDPE geomembrane is used in combination with a compacted soil liner.

The water table was based on observations made during the site investigation and was modelled at 2.0 to 2.5 m below natural ground level at the base of the valley.

6.8.4 Scenarios Modelled

Seepage from the facility at the end of the operation has been modelled as the critical scenario. This model was used to determine seepage levels and pressures at the maximum tailings and pond levels. The scenario was broken down into two cases as follows:

- Case 1 – Expected Operational Conditions
  This model assumes an operational basin underdrainage system and HDPE liner. The decant pond is assumed to be that arising from average rainfall conditions. The results of this case are shown in Figure 6.7.
- Case 2 – Underdrainage System Not Operational
  The purpose of this model was to examine the effect of the underdrainage system on the performance of the TSF. The model is identical to Case 1 but with no underdrainage system. The results of this case are shown in Figure 6.8.

6.8.5 Boundary Conditions

The following boundary conditions were assumed in the analysis:

- The supernatant pond is represented by a constant head boundary condition, where the head is equal to the elevation of the pond surface.

- At the left edge of the model (i.e. at the embankment) the water level was set at 2.5 m below ground level.
- At the right side of the model the water level was set at 2 m below ground level.
- Drainage systems were modelled as a series of free draining points (or zero pressure nodes). These nodes were placed at the design underdrainage spacing to account for the infiltration rate.
- The downstream toe was modelled using flux (Q) review nodes, by maximum pressure (seepage may pass through the downstream toe).

6.8.6 Results of Seepage Assessment

The seepage modelling results are summarised in Table 6.3. The seepage rates tabled do not include discharge from the drainage systems (i.e. the rates listed are actual seepage losses from the TSF rather than water circulated through the tailings mass). The seepage through the basin is pro-rated by the ratio of the basin area to the length of the model.

Table 6.3: Results of seepage modelling for final stage

Case	Water flow through basin (L/s/m)	Water flow through basin (L/s)	No. of Times Case 1	Equivalent permeability (m/s)	Figure No.
Underdrainage Functioning Partially Saturated Tailings	1.78×10^{-4}	0.036	-	3.9×10^{-10}	6.7
Underdrainage Not Functioning Partially Saturated Tailings	7.29×10^{-4}	0.148	4.1	1.6×10^{-9}	6.8

The results of the two cases are discussed below:

- Case 1 – Expected Operational Conditions
  Case 1 was modelled with the basin underdrainage system operational. At the left side of the model (i.e. at the main embankment) the phreatic surface is at the level of the underdrainage system, and thus the tailings in this area are unsaturated. In the area beneath the decant pond the tailings are saturated. However, the underdrains reduce the pressure at the HDPE liner, indicating that the drainage is effective in lowering the pressure on the liner and reducing the seepage loss.
  The seepage collection system at the TSF embankment upstream toe acts to capture seepage and mitigates seepage into the downstream environment. The seepage rate of 0.036 L/s is equivalent to a basin permeability of 3.9×10^{-10} m/s.
- Case 2 – Underdrainage System Not Operational
  The impact of having the underdrainage non-operational is that the phreatic surface which extended from the edge of the pond to the top of the soil liner in Case 1 now extends across the tailings to the toe of the embankment. Over the liner itself the pressure head is equal to the height of the pond. The increase in pressure results in an increase in seepage rate to 0.15 L/s, which is equivalent to a basin permeability of 1.6×10^{-9} m/s.

As shown in Table 6.3 the seepage rate increases by 4.1 times when the basin underdrainage system is non-operational. This indicates that water previously collected by the underdrainage now seeps through to the TSF embankment. At the embankment, the low permeability Zone A and the seepage collection system largely intercept this seepage, though obviously the seepage collection system intercepts a greater volume of flow. However the increased flow rate is within the capacity of the seepage collection system.

6.8.7 Conclusions

The results of the seepage modelling provide the following conclusions:

- The phreatic surface will remain well away from the TSF embankment under expected operational conditions.
- The proposed arrangement of the spine and basin HDPE geomembrane liner with underdrainage system will result in significantly reduced seepage from the facility, by about 78% compared to the case where the underdrainage system is not operational.

- Inevitably some seepage will occur through the TSF basin. However, the seepage rates are equivalent to an overall basin permeability of between 1.6 x 10^{-9} m/s and 3.9 x 10^{-10} m/s, well below that required by the Government of NSW PAC operating limits.

6.9 STABILITY ASSESSMENT

6.9.1 Embankment Stability

The stability of the tailings storage facility embankment was assessed in order to confirm the factors of safety against shear failure under the range of possible operating conditions. In accordance with Australian National Committee on Large Dams (ANCOLD) guidelines, the assessment covered the following steps:

- Analysis under static conditions.
- Analysis under seismic conditions (pseudo-static analysis).
- Deformation analysis under earthquake loading (Makdisi and Seed method).
- Liquefaction potential assessment.

The computer program XSTABL (Ref. 31) was used for the static and pseudo-static analyses, which were carried out using the modified Bishop method. XSTABL calculates the magnitude of the de-stabilising forces in the embankment slope and compares this to the total strength of the soil structure. The calculated ratio of these two parameters is the factor of safety against slope failure. When the de-stabilising forces are equal to the strength of the structure, this ratio (the factor of safety) is equal to one and the embankment is said to be "just stable". As the factor of safety increases, the probability of an embankment failure is reduced.

The stability of the embankments under earthquake loading conditions was assessed using pseudo-static methods of analysis. A horizontal ground acceleration of 0.07g was adopted as the operating basis design acceleration (OBE) for the analysis, based on the seismicity assessment for the site area. An event of this magnitude is calculated to have a return period of 1 in 475 years or, in effect, a 10% probability of occurring in 50 years.

In addition to the above the stability of the facility was examined by applying a 1 in 1000 year return period maximum design earthquake (MDE) acceleration of 0.11g in the analysis.

The stability of the embankments was measured under each load case against the minimum recommended factors of safety against failure, as provided in the ANCOLD

"Guidelines for Design of Dams for Earthquake" (Ref. 32). These factors are summarised in Table 6.4.

Table 6.4: Minimum factors of safety for design.

Case being Analysed	Minimum FOS
Long-term static loading stability (after closure)	1.5
Short-term static loading stability (during operation)	1.3
Pseudo-static loading stability (under OBE conditions)	1.1

6.9.2 Material Properties

The properties of the materials to be used for embankment construction are based on the results of the site investigation and laboratory testing of typical samples. The strength properties selected are considered to be representative of the various types of materials identified during the investigation and proposed to be used in the embankment, and are based on laboratory test data where this is available. The adopted shear strength parameters for the various material groups are defined in Table 6.5.

Table 6.5: Shear strength parameters

Material	γ_{moist} (kN/m^3)	γ_{sat} (kN/m^3)	c' (kPa)	Ø' (°)
Zone A low permeability fill	18	19	5	28
Zone C structural fill	18	18	0	30
Tailings	14	16	0	20
Alluvium/ Colluvium	18	20	0	30
Weathered rock	21	22	0	33
Granite	23	24	0	40

6.9.3 Embankment Stability

The stability of the TSF embankment was assessed under both static and pseudo-static conditions for a number of possible failure modes, and at both Stage 1 and final height. The models were analysed using conservative assumptions regarding the level of the tailings and the phreatic surface. For example, when downstream stability was being considered, the model assumed that the TSF was at full capacity. In addition, the effect of the pore water pressures on embankment stability was also modelled very conservatively by incorporating high phreatic surfaces in order to analyse the worst case scenario. In practice, it is expected that the decant pond will be located well away from the main embankment and that, even in the event of a storm event, the rise in pond level will be temporary only and should not cause a permanent rise in the phreatic surface. On this basis the analysed phreatic surfaces are considered to be higher than would be experienced in practice. Obviously, this will be monitored and the stability of the embankments will be reviewed regularly as part of on-going monitoring of the facility.

The analysed sections were derived from Drg. No. PE801-00139-012 and represent the critical sections where the embankment height is greatest. The results of the stability analyses are presented in Table 6.6. The modes of failure and the geometry of the analysed sections are shown on figures 6.9 through 6.14.

Table 6.6: Summary of TSF embankment stability results

Case	Description	Static Factor of Safety	Pseudo-static Factor of Safety (OBE)	Pseudo-static Factor of Safety (MDE)	Figure No.
1	Starter, Downstream failure, High Pond.	1.95	1.55	1.38	6.9 & 6.10
2	Starter, Upstream failure, Intermediate pond.	1.98	1.53	-	6.11
3	Final, Downstream failure, High Pond.	1.84	1.46	1.3	6.12 & 6.13
4	Final, Upstream failure, Intermediate pond.	1.3	1.0 (see Note 1)	-	6.14

Note1: Estimated crest deformation of 0.5 cm.

Comparison of the results with the ANCOLD minimum factors of safety indicates that the embankment sections are stable under both static and pseudo-static loadings.

For the final stage upstream face, factors of safety for the static and pseudo-static conditions of 1.3 and 1.0 were calculated. As this is a short-term and very conservatively modelled scenario (tailings will be placed in the facility thus reducing the height of the free-standing face and the pond level would not be expected to reside at such a high level) the factors of safety are considered to be acceptable.

6.9.4 Deformation Analysis

The methods given in *"Simplified Procedure for Estimating Dam and Embankment Earthquake Induced Deformations"* (Ref. 33) were used to estimate the crest deformation under the OBE pseudo-static conditions. In this case, the estimated deformation is less than 1 cm, well below the embankment freeboard, and therefore the potential for uncontrolled loss of storage is insignificant.

Relating satisfactory dam performance to earthquake induced deformation is very subjective, and generally depends on dam specific criteria about the allowable loss of freeboard, or the tolerable extent of horizontal displacements. Whilst the calculated magnitude of displacements is fairly insignificant, there are a number of additional reasons why the stability of the embankment under earthquake loading conditions is considered to be acceptable:

- Historically, even at short distances from an earthquake epicentre, there have been no complete failures of embankments built of clay soils, but several dams have come close to failure.
- Dams which have suffered complete failure as a result of earthquake shaking have been constructed primarily with saturated sandy materials or on saturated sand foundations. Liquefaction was a major contributing factor in these failures.
- Well-constructed dams of clay soils on clay or rock foundations not susceptible to strain weakening can withstand extremely strong shaking resulting from earthquakes of up to magnitude 8.25 with peak ground acceleration ranging from 0.35g to 0.8g.
- The foundation soils and proposed embankment construction materials are not subject to strain softening, and are not liquefiable. The static factor of safety of the critical failure surfaces involving loss of crest elevation are greater than 1.5 under working conditions expected prior to an earthquake.
- The minimum horizontal thickness of the constructed embankment will be 6 m, which is relatively thick in relation to potential movements of the embankment.
- There are no outlet works or low strength seams passing through the embankment or foundation which could produce leakage or potential piping erosion in the embankment.

In addition, it should be noted that under most conditions there will only be a limited amount of water in the facility.

6.9.5 Liquefaction Assessment

The embankment foundation comprises weathered rock and the embankment construction materials comprise clay and rock materials, and therefore neither are considered to be liquefiable.

The liquefaction potential of the tailings may be classified according to its particle size distribution. In general terms, saturated sands, silty sands, silts and gravelly sands are most susceptible to liquefaction, whilst finer grained soils are usually less susceptible.

However, experience has shown that even soils with small amounts of clay may liquefy. In addition, mine tailings are more susceptible to liquefaction than natural soils, possibly reflecting their uniform size and recent deposition. There is some evidence that tailings will "age" and develop greater resistance to liquefaction with time.

Figure 6.15 shows the particle size distribution of the combined tailings sample in comparison to the particle size envelopes for slimes with low resistance to liquefaction and potentially liquefiable soils. This shows that the particle size distribution of the combined tailings sample lies within the boundaries of potentially and most liquefiable soils.

Following the guidance provided in *"Ground Motions and Soil Liquefaction During Earthquakes"* (Ref. 34), liquefaction can only occur if all three of the following conditions are met:

- The clay content (particles less than 5 microns) is less than 15% by weight.
- The liquid limit is less than 35%.
- The moisture content is no less than 0.9 times the liquid limit.

Based on Atterberg Limit tests, the liquid limit of the combined tailings is 27% and the material is non-plastic. According to the particle size distribution test, the clay-sized particle fraction of the tailings is approximately 7% by weight. It is estimated that the moisture content of the tailings will generally remain above the liquid limit. Thus, the tailings properties fulfil all three criteria for liquefaction potential. This analysis together with the tailings particle size distribution, suggests that liquefaction of the tailings is a possibility.

Testing of the deposited tailings in situ using electric friction cone penetrometer (EFCP) methods will be carried out towards the end of Stage 1 deposition to assess the suitability of the tailings beach adjacent to the upstream embankment face as a foundation for modified centreline embankment raises, and to confirm the preliminary assessment of liquefaction potential of the tailings material. The latter uses parameters derived from the EFCP testing to calculate the Cyclic Resistance Ratio (CRR) discussed in *"Liquefaction in Tailings and its Foundation"* (Ref. 35), which is then compared with the Cyclic Stress Ratio (a function of the earthquake magnitude). Cyclic softening (or liquefaction) is considered to be possible if, at any depth, CSR is greater than CRR. The results of this programme will be used to assist in design of the Stage 2 embankment raise.

6.10 TAILINGS STORAGE FACILITY MANAGEMENT

6.10.1 Tailings Deposition System

The deposition of tailings into the storage facility will be primarily from the TSF embankment. The tailings delivery pipeline will be routed from the process plant up to the crest of the TSF embankment. The tailings distribution pipeline will be located on the embankment crest and will be raised with each stage.

Deposition will occur from single offtakes inserted along the tailings distribution pipeline. The deposition location will be moved on a daily basis to one of the deposition points, or as required to control the location of the supernatant pond. All of the valves in the deposition system will be provided with pneumatic actuators for ease of operation.

6.10.2 Deposition Technique

Tailings deposition will be carried out using the sub-aerial technique in order to promote the maximum amount of water removal from the facility by the formation of a large beach for drying and draining. Together with keeping the pond size to a minimum, sub-aerial deposition will increase the settled density of the tailings and hence maximise the storage potential and efficiency of the facility.

The tailings will be deposited into the facility from the embankment in such a way as to encourage the formation of beaches over which the slurry will flow along the spine of the basin in a laminar non-turbulent manner. Limited settlement and water release will occur. The released water will form a thin film on the surface of the tailings. This water will flow to the supernatant pond from where it will be removed from the storage area via a decant tower. The Stage 1 decant tower is located such that it will first receive water approximately 3 months after commissioning the facility.

Deposition of the tailings will be carried out on a cyclic basis with the tailings being deposited over one area of the storage until the required layer thickness has been built up. Deposition will then be moved to an adjacent part of the storage to allow the deposition layer to dry and consolidate. This will facilitate maximum storage to be achieved across the whole valley.

After deposition on a particular area of beach ceases and settling of the tailings has been completed, further de-watering will take place due partly to drainage into the underdrainage system, but mainly due to evaporation. As water evaporates and the moisture content drops, the volume of tailings will reduce to maintain a condition of full saturation within the tailings. This process will continue until interaction between the tailings particles negates volume reduction.

7. DAM BREACH ASSESSMENT

A dam breach assessment was performed in order to assess the effects of a dam failure downstream of the facility. The assessment is detailed in KP report "*Dargues Reef Gold Project, Tailings Storage Facility, Dam Breach Assessment*", PE801-000139/8 (Ref. 36) a copy of which is included as Appendix J, and is summarised herein.

There are two types of consequence categories, which indicate the conditions that exist in the vicinity of the facility immediately prior to onset of a dam breach:

- Sunny Day Consequence Category (SDCC), which refers to failures that occur without any attendant natural flooding;
- Flood Consequence Category (FCC), which refers to failures that occur in association with a natural flood.

Three major failure scenarios were assessed through the implementation of dam breach modelling:

- An overtopping water breach occurring during a sunny day (i.e. without coincident precipitation) - the initial decant pond level for this scenario corresponds with the invert elevation of the main spillway, which defines the SDCC;
- An overtopping water breach initiated by a PMPDF - the initial decant pond level for this scenario corresponds with average conditions during the last month of planned operations, which defines the FCC;
- An embankment failure which precipitates a tailings run-out, which is expected to occur *in addition to* either of the other two major failure scenarios.

Analysis of breach formation parameters led to the selection of the overtopping breach mechanism over the piping breach mechanism because it resulted in more conservative breach outflows.

Water inundation maps were prepared as a result of the first two major failure scenarios and a tailings run-out map was prepared for the third failure scenario. These maps were inspected and compared to determine the incremental consequences associated with each failure scenario. The results of this assessment indicate that any decant release and potential tailings run-out following a dam breach would not be expected to impact the downstream mine infrastructure (box cut entrance to underground workings, process plant site, offices, labs, workshops, paste hole, vent riser and escapeway). Additionally, the town of Majors Creek is not expected to be impacted by a breach of the TSF.

From comparison of inundation occurring under both FCC and SDCC scenarios, KP recommends that the FCC is used for consequence assessment. For the FCC, which is the PMPDF for this facility, the results indicate that the initial consequence category assessment of *Significant* is reasonable.

8. MONITORING

8.1 TAILINGS STORAGE FACILITY

8.1.1 Introduction

A comprehensive monitoring programme will be developed to monitor for any potential problems. The monitoring will include survey pins to check embankment movements, piezometers in the embankment and monitoring bores downstream of the embankment. The piezometers and bores will be monitored monthly for water levels and quarterly for water quality. Typical details of the instrumentation are shown on Drg. Nos. PE801-00139-010 and PE801-00139-028.

If the monitoring programme indicates that potential problems are developing, then an increase in monitoring frequency should be implemented and a response plan developed.

8.1.2 Seepage Monitoring

The TSF design incorporates a number of measures to minimise the amount of seepage which will occur form the facility in order to mitigate the extent of any effects on the downstream environment. However, some seepage from the facility is inevitable and, to this end, five groundwater quality monitoring stations are proposed to be installed downstream of the facility to facilitate early detection of changes in groundwater level and/or quality during the operating life and following de-commissioning. These are notated as MB-01 to MB-05 and are shown on Drg. No. PE801-00039-010. Each monitoring installation consists of one shallow hole, extending to a depth of approximately 5 to10 m, and one deep hole terminating below the groundwater table at approximately 15 to 25 m depth. Each borehole will be cased and screened over an interval set in the field during installation, and sealed back to surface with low permeability grout. The casing for the monitoring bores will be either 50 mm or 100 mm diameter so that each monitoring bore can be converted to a de-watering bore if required. The bores will be installed before commissioning the TSF in order to accumulate baseline data specific to the storage location.

8.1.3 Stability Monitoring

Pore water pressures should be monitored within the embankment to ensure that stability is not compromised. To this end standpipe piezometers will be installed at five locations along the embankment (refer Drg. No. PE801-00039-010). Each standpipe will consist of a 25 or 50 mm diameter PVC tube slotted at the base or supplied with a filter tap. The slotted section will be surrounded by sand, and bentonite pellets will be placed above the sand to provide a seal. The remainder of the hole will be sealed with a bentonite / cement grout. The top of the piezometer will be provided with a lockable cap to prevent tampering or vandalism. As an alternative to standpipe piezometers, pneumatic or vibrating wire piezometers could be used. The base of each piezometer will be located within the embankment fill to ensure that the phreatic surface within the embankment, as opposed to groundwater level. is measured. Additional piezometers may be installed as the embankment is raised to monitor the development of the phreatic surface in the embankments.

The piezometers will be monitored at regular intervals as outlined in the monitoring programme and any rises in water level noted. Increases of greater than 10% of the embankment height should be referred to a qualified geotechnical engineer for further investigation. The piezometer levels should be monitored to ensure that the phreatic surface does not reduce the overall stability of the embankments below acceptable levels. Remedial action will be undertaken if increases in pore water pressure are unacceptably high.

8.1.4 Survey Pins

Survey pins will be installed along the crest and downstream faces of the embankment, where greater than 10 m in height, in order to monitor embankment movements so as to be able to assess effects of any such movement on the embankment. The survey pins will be located at 50 m intervals along the downstream side of the embankment crest. The details of each pin (date of installation, survey pin No., Northing, Easting and RL) will be recorded on installation.

Each pin will be monitored for movement at regular intervals as outlined in the monitoring programme. Any displacement of the embankment which is considered excessive or on-going may indicate embankment stability problems and will require investigation by a qualified geotechnical engineer. Remedial action will be undertaken if required based on the conclusions drawn from such an investigation.

8.1.5 Tailings Performance Monitoring

Tailings performance monitoring will include monitoring of the following variables on a continuous basis:

- Solids tonnage to the tailings storage facility.
- Water volume to the tailings storage facility.
- Rainfall and evaporation at the facility.

- Collection efficiency of the underdrainage system based on underdrainage sump pump monitoring.

Monitoring of tailings moisture contents and densities, and survey of the tailings beach and supernatant pond locations should be conducted four times a year.

8.1.6 Emergency Controls

Under normal operating conditions the following systems should be in place:

- The tailings pipeline will be located on the upstream crest of the embankment, which will have a minimum cross fall to the tailings beaches of 2%. Any leakage from the pipeline will therefore flow towards the tailings storage facility.
- The facility is protected by a spillway so that in the unlikely event of an overflow situation, water will be discharged into the local creek system and the embankment will not be overtopped.
- Between the plant site and the tailings storage facility, the tailings pipeline and decant return line will be contained within a bunded easement, and equipped with an automatic pressure drop cut-out.

These systems should greatly reduce the likelihood of uncontrolled spillages from the tailings storage facility.

A detailed monitoring programme will be provided as part of the operating manual for the facility.

8.2 ANNUAL INSPECTION REQUIREMENTS

In accordance with the requirements of DSC guideline *"DSC2C – Surveillance Reports for Dams"* (Ref. 37), the facility will require an annual audit by a suitably qualified geotechnical engineer to ensure that the facility is operating in a safe and efficient manner. The audit should include, but not be limited to, the following items (depending upon the facility):

- Current survey plan of facility showing spot elevations along walls and across tailings beaches, if possible.
- Reconciliation of stored tailings volume and calculated densities with the expected values given in the design, and assessment of available capacity remaining in terms of volume and time.
- Assessment of in situ tailings properties, including particle size distribution, in situ strength, density and moisture properties.

- Water balance studies with approximate reconciliations of slurry volumes, solids content, decant recovery, site rainfall and evaporation. In conjunction with contained moisture information, this will provide an indication of possible seepage losses.
- Validation of storage design, using input parameters derived from site measurements and testing, implications for future storage if present trends are continued, and recommendations for any necessary operational or design modifications.
- Presentation of interpretation of monitoring results, proposals for additional monitoring of identified areas, changes to operational procedures resulting from monitoring results and proposals for any necessary seepage recovery systems.
- General description of facility, complete review of residue and water management practices and operating manual procedures, their problems, failures and successes, and any alteration to the facility or operating procedures that are proposed.
- A complete description of the previous embankment lift with as-built drawings and design proposals for the next embankment lift based on the recorded data.

8.3 MONITORING AND MAINTENANCE PROGRAMME

8.3.1 Monitoring Programme

As part of the operation of the facility, extensive monitoring of all aspects of the operation should be undertaken. This monitoring falls into three basic categories:

i. Short-term operation monitoring – this includes items such as offtake location, whether pipe joints are leaking, etc., which are part of ensuring that the facility is operating smoothly.

ii. Compliance monitoring – this includes items such as checking survey pins for movement and monitoring bores for contamination, etc., which are used to ensure that the project is meeting all of its commitments in regard to a safe, secure operation.

iii. Long-term performance monitoring – this includes such items as tailings level surveys and water flow measurements, etc., which are used to monitor the long term performance of the facility and refine future embankment lift levels and final residue extent.

Table 8.1 summarises the monitoring requirements for the TSF.

Table 8.1: Monitoring programme

Area	Monitoring Requirement	Frequency
Section 1:	Short term operation monitoring	
Tailings Facility	Pipeline integrity	Daily
	TSF liner integrity	Daily
	Visual check on tailings level versus embankment crest	Daily
	Offtake location	Daily
	Blockage of discharge	Daily
Decant	Size of decant pond	Daily
	Location of decant pond	Daily
Section 2:	Compliance Monitoring	
Embankment	Survey pins	Three months
Monitoring bores	General inspection by suitably qualified engineer	Annually
	Water volume	Monthly
	Water level	Three monthly
	Water quality – conductivity	Three monthly
	Water quality – major component analysis	Annually
Piezometers	Water level	Monthly
Section 3:	Performance Monitoring	
Climatic	Precipitation	Daily
	Evaporation	Daily
	Maximum - minimum temperatures	Daily
	Wind direction and speed	Daily
Tailings	Tailings solids (tonnes)	Daily
	Water in tailings (tonnes or m^3)	Daily
	Average tailings flow (m^3/s)	Daily
	Tailings surface survey	Three monthly
Decant water from pumps	Outflow from decant	Daily
	Outflow from underdrainage	Daily
	Outflow from toe drains	Weekly
Technical Audit (Section 8.2)	Independent geotechnical engineer	Annually

8.3.2 Maintenance Programme

Inspection and maintenance of the TSF is largely aimed at mitigating potential problems by dealing with them before they can develop into major problems.

Some aspects of the maintenance programme such as inspections can be integrated into the monitoring programme. If problems are detected then the problem is either to be corrected immediately or is to be noted and a maintenance request form filled in. The form will allow for different levels of urgency depending on how quickly the maintenance is required. The assessment of urgency should be based on the potential for the problem to affect the operation or integrity of the facility. The maintenance will then be integrated with the overall site maintenance programme.

Table 8.2 outlines the maintenance requirements for each area of the facility. Modifications to the maintenance programmes as a result of emergency situations or annual reviews should be reviewed regularly.

Table 8.2: Maintenance programme

Area	Monitoring Requirement	Frequency
Tailings Facility Pipeline	Inspect pipeline for pipe bursts, leaking joints.	Daily
	Inspect offtake(s) for blockages, failure etc. Repair and/or replace as necessary. Re-locate offtake(s) to new location and check that flow at new location is acceptable.	Daily
	Break pipeline at selected locations and observe wear on valves, fittings and pipes. Remove any accumulated material from valves etc. Rotate pipeline if excessive wear is occurring along invert of pipe.	Annually during programmed plant shut down
	Carry out maintenance on valves and fittings as recommended by suppliers.	As recommended
Decant	Inspect decant pond and location. Adjust discharge location to maintain pond around decant tower.	Daily
	Inspect decant for damage or debris.	Weekly or after significant rain events
Embankment	Check general structural integrity and visual signs of seepage through embankment.	Daily
	Visual inspection for slips. erosion problems including around survey pins, tension cracks etc. Problems to be referred to qualified geotechnical engineer for assessment.	Weekly
Underdrainage System	Check for erosion or other damage. Repair and/or replace as necessary.	Monthly or after significant rain events
Toe Drains	Check for damage, collapse or ingress of material.	Monthly or after significant rain events
Instrumentation	Inspect all instrumentation and repair/ replace as required.	Frequency as per instrumentation instructions
General	General inspection of tailings facility and all structures	Prior to wet season
	Geotechnical audit and report.	Annually

9. EMERGENCY ACTION PLANS

9.1 GENERAL

Emergency situations and unforeseen natural disasters can have serious effects on the operation of the tailings storage facility. Potential consequences and remedial procedures are reviewed in this section for the following events:

- Tailings pipeline failure.
- Power failure.
- Earthquake events.
- Extreme rainfall.
- Dam break/overtopping.

The consequences of the various situations and some remedial procedures are provided.

9.2 TAILINGS PIPELINE FAILURE

9.2.1 General

For the tailings discharge system, three potential emergency situations are possible:

i. Rupture in the pipeline (including joint failure).
ii. Blockage of the delivery line, distribution line and/or offtakes.
iii. Tailings overflow.

The tailings pipeline should be contained in a bunded corridor in areas where the pipeline doesn't drain into the tailings facility. As a result, emergency situations should not result in the release of tailings from the facility or contamination of any local surface water systems. However, the capacity of such systems to store tailings is limited and regular inspections should be carried out.

The potential emergencies and the appropriate responses are outlined in the following sections.

9.2.2 Rupture of the Delivery/Distribution System

The most likely location for rupture is at valves, joints and fittings along the pipeline(s). Regular inspection of the pipeline(s) should be carried out. All minor leakages should be noted and the joint disassembled, checked and repaired as required during the next programmed maintenance period or plant shutdown. This procedure will prevent major problems from developing.

Should a major rupture occur, local erosion of the bunded corridor or embankment might occur. Any tailings spillages should be cleaned up and the tailings placed into the tailings facility. Any damage to the embankment, bund, etc., should be restored and made good.

9.2.3 Distribution Pipeline Blockage

The procedures to be undertaken if a blockage occurs in the distribution line will consist either of moving the deposition location back along the line closer to the plant, or shutting off the flow completely prior to undertaking the maintenance works.

The failure of an offtake will be treated in a similar manner to the procedures for a blockage in the distribution line. The offtake containing the blockage should be removed and the new offtake assembled. On replacement, deposition should be returned to its original position.

9.2.4 Tailings Overflow

The tailings facility is designed to operate with a minimum of 0.3 m tailings freeboard at the commencement of construction of the next stage of embankment lifts. The purpose of this freeboard is to ensure that during normal operations the potential for tailings to overflow the embankment is minimal. Areas where the tailings surface is close to the embankment crest level (i.e. less than 300 mm below the crest level) should be noted and deposition from these areas prevented.

9.3 POWER FAILURE

The tailings pumps will be connected to the process plant power distribution system. In the event of a total loss of power, all pump systems and automatic valves not connected to standby generators will cease operations. Systems affected could include:

- Water control and removal in the decant pond.
- Tailings pumping system.

i. Water control and removal in the supernatant pond

Shut-down of the decant pump will mean that the level of the water in the decant pond will begin to rise. Visual inspection of the supernatant pond level will be required. However, the available storage volume is very large and, even under worst case conditions, would take approximately 2 days to fill. In this time power would be restored or the plant would be shutdown. In addition, there is a temporary spillway to downstream of the embankment if the pond level reaches the maximum capacity of the facility.

ii. Tailings pumping system

Upon loss of power, the tailings pumping system will shut down. In the event that the tailings cannot be re-mobilised, the pipeline(s) will need to be drained by manual valves into sumps constructed at the pipeline(s) low points (which should be designed to store 110% of the pipeline volume).

Discontinuing tailings discharge will have no significant effect on the facility. Decant water and rainfall runoff will continue to collect in the decant pond.

9.4 EARTHQUAKE EVENTS

Significant earthquake events have occurred in the Dargues Reef project area (as outlined in Section 2.5); accordingly the tailings storage facility has been designed to stringent stability criteria. However, the stability of the facility embankments should be checked annually as part of the review of the facility.

Contingency procedures to be adopted in the event of an earthquake are dependent on the intensity of the earthquake event at the site and are outlined below:

a) Major earthquake event (Modified Mercalli Intensity VI or greater)

Recognition: Difficulty standing. hanging objects quiver. masonry cracks, waves on ponds, some minor injuries – refer *"Guidelines for Design of Dams for Earthquake"* (Ref. 32).

Response i) Immediately terminate deposition of tailings into the TSF and pumping from the decant.

ii) Immediate inspection of facility embankments and decants for obvious deformation or movement.

iii) Immediate inspection of all pipes including tailings pipeline(s) for rupture or leakage.

iv) Arrange for immediate inspection and report of storage by a suitably qualified geotechnical engineer.

v) Deposition can be recommenced if no major damage to facility and pipelines has occurred.

vi) Survey pins and monitoring bores are to be read immediately after the event and all instrumentation to be read daily.

vii) A detailed daily inspection of the facility is to be undertaken until completion of the geotechnical engineer's report.

b) Minor earthquake event (Modified Mercalli Intensity V or less)

Recognition: Felt outdoors as well as indoors, liquid disturbed, small objects displaced, doors swing open or closed, pictures move - refer *"Guidelines for Design of Dams for Earthquake"* (Ref. 32).

Response

i) Inspect the tailings pipeline for rupture or leakage. If required by damage to the lines stop tailings pumping until repairs are complete.

ii) Inspect embankments and decants for obvious deformation or movement.

iii) Monitor all instrumentation immediately after the event and weekly thereafter until readings return to normal.

iv) If any damage or leakage is observed. immediately arrange for an inspection by a suitably qualified geotechnical engineer.

9.5 EXTREME RAINFALL EVENTS

The tailings storage facility has a minimum flood storage capacity equivalent to a 1:100 AEP, 72 hour storm event, in addition to a minimum design freeboard of 300 mm. Consequently the flood storage capacity will be well in excess of the design capacity. The spillway at each stage is designed to attenuate a 1:1,000 AEP storm event, assuming that the facility is full at commencement of this storm event. On this basis it is not expected that overtopping of the embankment will occur.

Major storm events can cause serious erosion problems as well as other damage. A system of inspections should be carried out to mitigate any potential adverse consequences of extreme rainfall (see tables 8.1 and 8.2).

9.6 DAM FAILURE/OVERTOPPING

9.6.1 General

The TSF embankment is designed for defined operating capacities and specific storm event return periods. The operators working within the facility area will be trained in the correct method and approach to be used for the deposition of tailings. In addition, they will be provided with an appropriate operating manual and trained to recognise problem situations before they become critical. On this basis it is expected that the risk of a dam failure or overtopping event will be extremely small.

Although this risk is very small, it is not absolute zero. Therefore, an assessment has been made of the potential consequences of a release of contaminants (either tailings or seepage water). Details of the assessment are given in "*Dargues Reef Gold Project, Tailings Storage Facility, Dam Breach Assessment*", PE801-000139/8 (Ref. 37). Typical emergency action plans are provided for each type of release with specific details provided for each situation.

The analysis is based on an assessment of potential failure mechanisms, the consequences of each type of failure, and remedial measures that should be undertaken for each situation.

9.6.2 Emergency Action Plan – Contaminated Seepage

The release of seepage from the facility and detection in the monitoring system will be handled as part of the general maintenance and monitoring programme. The levels and chemical constituents of the groundwater in the bores will be monitored on a regular basis in accordance with the lease operating conditions.

If unacceptable changes in the concentration of the chemical constituents of the groundwater are detected in the monitoring bores, the following plan will be initiated:

i. A repeat sample will be taken to check that the initial measurement was correct.

ii. Frequency of monitoring will be increased from monthly to weekly.

iii. An action plan will be developed for the specific situation and, if necessary, modified until the contamination problem has been controlled or removed. This may include the development of a groundwater model, installation of additional bores, conversion of monitoring bores to production bores, and/or modification of tailings deposition or water management control in this area.

If seepage is occurring at the surface either through the embankment or foundation zone, an immediate inspection and review by a qualified geotechnical engineer should be arranged. The review would assess the following areas:

- Stability of the embankments (including the potential for piping failure).
- Modifications of the operating procedures to reduce or eliminate the seepage.
- Control of the emerging seepage water.

For all situations where seepage occurs, the relevant authorities will be notified of the situation and all proposed modifications/remedial work to be undertaken.

9.6.3 Emergency Action Plan – Embankment Overtopping

As discussed in Section 9.6.1, the risk of embankment overtopping is very low. Regular inspections will ensure that the volume of material which may potentially escape will be small. However, in the event of this situation occurring, the following procedures should be initiated:

- The deposition point should be re-located so that the loss of tailings out of the facility stops.
- An inspection should be made of the extent of the release.
- Relevant authorities should be contacted/notified.
- The tailings after drying should be picked up and stored in the facility.
- Any environmental or other damage should be made good.
- The area of the crest at which the spill occurred should be flagged off to indicate to the operators that no further deposition is to occur from this area until after the next embankment lift.

9.6.4 Emergency Action Plan – Dam Failure

This is the most critical type of emergency situation with the greatest hazard potential for damage. If such a dam failure occurred the following steps should be taken:

- Shut down the plant and cease deposition into the facility.
- Immediately on determining that a dam break has occurred, check on the location and safety of any personnel known to be in the area. If it is possible that someone has been caught in the flow slide, inform the emergency services (police rescue, etc.) and follow their recommendations.

- Report the incident to the relevant authorities.
- Inspect the flow area; determine the extent of the flow slide and the damage it has caused.
- Inspect the facility to determine the area of the embankment which has been damaged.
- If the tailings line has been buried or damaged disassemble the line, remove or repair the damaged sections and re-lay the pipeline into another area of the facility (if possible) to allow continuation of tailings deposition when safe to do so.
- Fix any damage to the decant structure, pipelines, access roads, etc. as quickly as possible.
- The breach and the overall facility should be inspected by a competent geotechnical engineer and a repair plan developed.
- After the tailings have dried and the breach is stable, the tailings should be picked up and placed into the facility.
- The breach should be repaired and all damage to the environment made good.
- The conditions before the failure should be determined and the operating procedure modified so that the same situation does not occur again.
- Re-commence tailings deposition when safe to do so.

9.6.5 TSF Embankment

- Description

The TSF embankment will be constructed downstream for the first stage, and then modified centreline for the subsequent two stages. The embankment will end up at a maximum of about 25 m to 30 m in height. The major failure scenario is a failure of the embankment, either due to stability failure or overtopping.

- Potential Consequences

Whilst a remote possibility, failure of the TSF embankment could result in the release of the decant pond and a significant quantity of tailings into the downstream environment. Tailings and decant water would enter the unnamed creek and contaminate the area downstream. There is also a very remote possibility of loss of human lives in this failure scenario.

- Response

The TSF embankment is designed so as to ensure that the level of risk of a failure scenario is extremely small. There are a number of tasks that should be undertaken in situations where the facility is operated close to its maximum capacity in order to ensure that this type of failure does not occur, or that the maximum duration of warning time is available. These are:

i. Ensuring that the storm storage capacity is always maintained.

ii. Minimising the size of the supernatant pond by controlling the decant discharge rate to the process plant.

iii. Monitoring embankment survey pins and piezometers on a regular basis, especially during the wetter months when the volume of water stored in the facility is highest.

If this type of failure does occur, the action plan presented in Section 9.6.4 should be implemented.