WESIZWE PLATINUM MINE

BAKUBUNG TAILINGS STORAGE FACILITY
DESIGN REPORT

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Project Number
RI301-00509/10

BAKUBUNG STORAGE FACILITY
DESIGN REPORT

Rev	Description	Date
2	Issued in Final	March 21

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ABBREVIATIONS

A .... Flow Area
ANCOLD .... Australian National Committee on Large Dams
C .... Runoff coefficient
COLTO .... Committee of Land and Transport Officials
CQA .... Construction Quality Assurance
CSL .... Critical State Line
CCL .... Compacted Clay Liner
EC .... Electrical Conductivity
FoS .... Factor of Safety
GRI .... Geosynthetics Research Institute
HDPE .... High Density Polyethylene
Ha .... Hectare
Ktpm .... Kilo tonnes per month
LCT .... Leachable Concentration Threshold
MDD .... Maximum Dry Density
Mtpa .... Million tonnes per month
Mamsl .... Metres Above Mean Sea Level
MOD .... Modified
OMC .... Optimum Moisture Content
PCD .... Pollution Control Dam
PGA .... Peak Ground Acceleration
Q .... Flow Rate
R .... Hydraulic Radius
RoR .... Rate of rise
RMD .... Ratio of mono-valent to divalent cations
RMD .... Ratio of mono-valent to divalent cations
SANAS .... South African National Accreditation System
SANS .... South African National Standards
T_c .... Time of concentration
TSF .... Tailings Storage Facility
TCT .... Total Concentration Threshold

1.0 INTRODUCTION

1.1 General

The Bakubung TSF is located in the North West Province, Bojanala District, East of Phatsima village and approximately 7 km South-West of Sun City. Figure 1-1 presents the locality of Bakubung Platinum Mine. Knight Piésold (KP) was appointed by Wesizwe Platinum Limited (Wesizwe) in March 2019 to carry out a feasibility level design of a filtered tailings storage facility (TSF) and associated infrastructure at their Bakubung Platinum Mine and in 2020 to develop the feasibility design further. This report described that design.

A site selection study was conducted by KP in December 2018 (KP, 2019) where engineering, economic, environmental, social and cultural aspects were weighted in a site selection matrix where the selected site was scored 2nd. Further the Client has reduced the life of the facility to meet the capacity of the selected site, resulting in the most appropriate site to be considered. Also, the preferred site at the time was since set aside for concentrator plant development.

The objective of the project is to:

- Design a tailings storage facility able to contain an average tonnage profile of 1 Mtpa for a maximum period of 7 years;
- Demonstrate that the environmental impacts of the tailings have been mitigated through a robust barrier system and adequate drainage management design; and
- To size and design a safe and stable tailings storage facility utilizing on site waste rock for the toe wall as part of the new TSF.

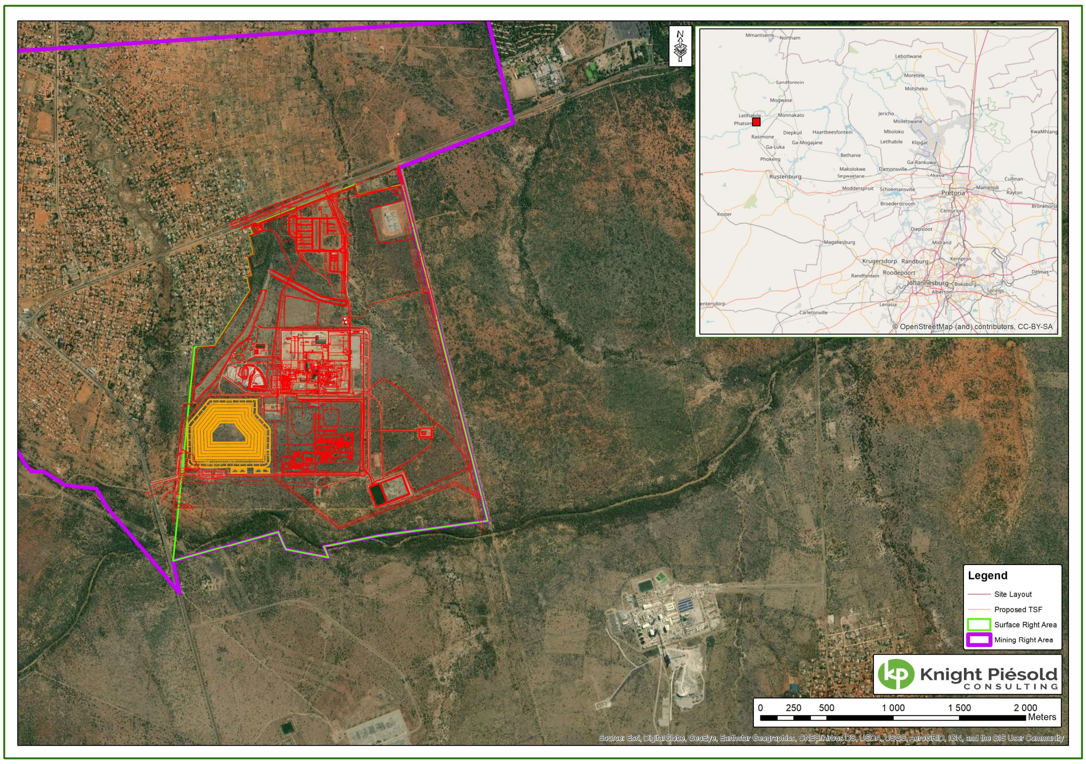

Figure 1-1: Locality of Bakubung Platinum Mine

1.2 SCOPE

The full scope of work to be carried out by KP encompassed the following activities:

- Project management
- Geotechnical Investigation
- Groundwater desktop study, which included:
    - Review of available information
    - Site visit
    - Groundwater recharge calculations
    - Conceptual groundwater site model
    - Numerical flow model
- Design, which included:
    - Confirmation of design criteria
    - Capacity Analysis
    - Geochemical Analysis of Tailings
    - Embankment Design

    - Surface Water Management
    - Pollution Control Barrier System Design
    - Drainage Design
    - Stability and Seepage Analysis
    - Construction Quality Assurance
    - Closure Consideration
- Drawings
- Updated schedule of quantities with costs
- Design Report

1.3 SITE DESCRIPTION

A walk-over site visit was conducted on the 26th March 2019, as per the project scope. The site visit was attended by Mr Katlego Magoro and Mr Shona Vaughan-Williams of KP accompanied by Mr Lungelo Nyandeni of Wesizwe. The purpose of the visit was to familiarise the design team with the site conditions and related aspects which may impact the TSF design.

A 1 m contour interval survey was made available to KP by Wesizwe for the purpose of the design. The Bakubung TSF is positioned on relatively flat land which has a uniform slope of ± 2.4 % in a North-South direction. The vegetation in the footprint area consists of light bush and shrubs with medium-sized trees.

There is an illuminated walkway which runs through the site from the West to the East. Additionally, there is an overhead and a buried powerline which runs through the site from the North substation towards the new entrance for the mine which is currently under construction. Details of the existing site conditions are presented in Drawing No. 301-00509/10-001. A site layout is presented in Figure 1-2.

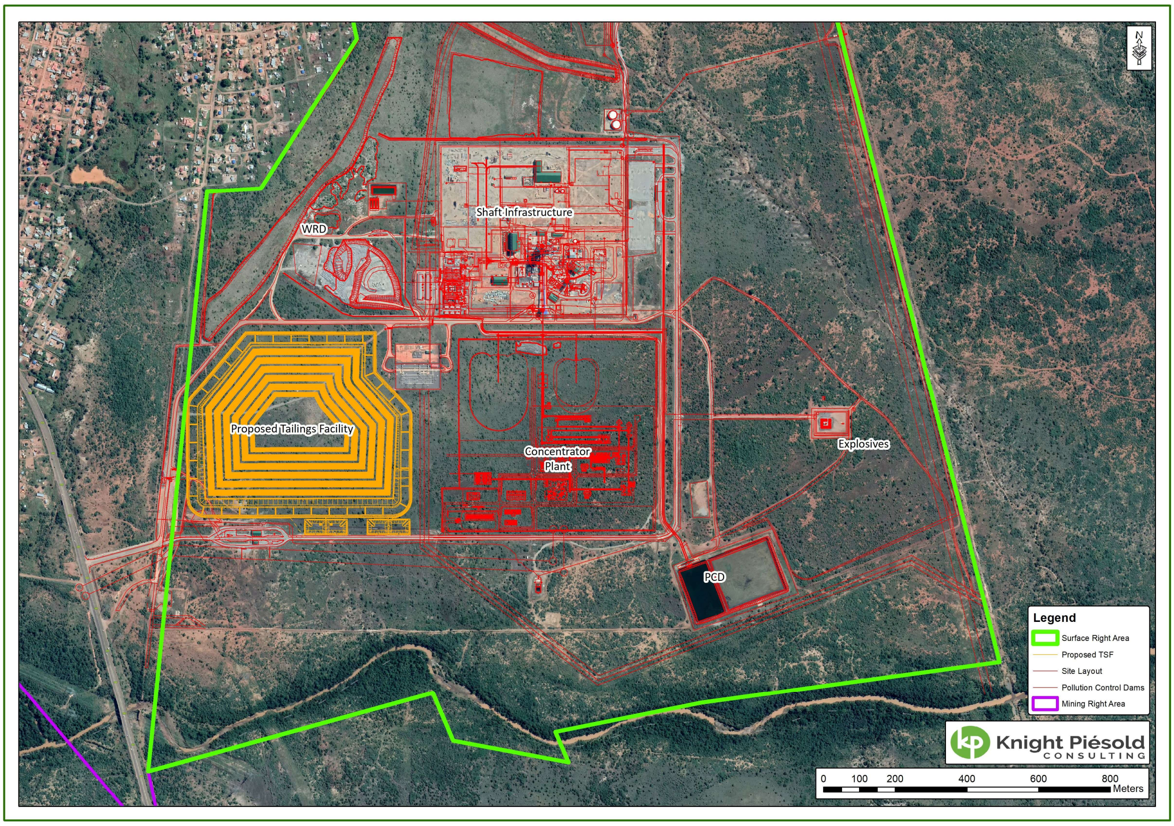

Figure 1-2: Site Layout

1.4 BATTERY LIMITS AND EXCLUSIONS

The battery limits for the design are as follows:

- The upstream battery limit for the incoming tailings will the method of transportation for the filtered tailings onto the TSF;
- The downstream battery limit is the evaporation ponds.

The following are excluded from the scope of this design:

- Relocation of road, powerline and services
- Materials transport, stacking and spreading equipment
- Mechanical and electrical components
- Technical Specifications
- Environmental Impact Assessment and Water Use Licensing processes
- Hydrogeological field work (geophysics, drilling and aquifer testing)
- Design relating to the transportation of the filtered tailings to the TSF
- Electrical requirements

- Topographical survey
- Tailings filter process or equipment specifications

2.0 DESIGN CRITERIA

The design criteria used in the preparation of the design are intended to meet or exceed South African legal requirements and experience worldwide on similar projects. All design criteria have been sourced from work on the site or as cautious estimate which will be finalised during detailed design.

The design criteria used developed for this design is presented in Table 2-1 below:

Table 2-1: Bakubung Platinum Mine Design Criteria

Item	Description	Value	Source/Comment
1	Tailings Type	Filtered Platinum Tailings	Wesizwe Platinum Limited
2	Deposition Rate	1 Mtpa	Wesizwe Platinum Limited
3	Total Capacity Required	7 Mt (min)	Wesizwe Platinum Limited
4	Life of facility	7 Years (min)	Wesizwe Platinum Limited
5	In-situ Dry Density	1.6 t/m^3 (minimum expected)	Knight Piésold
6	Deposition Method	Conveyor belt system	Wesizwe Platinum Limited
7	Design Storm	1 in 100-year, 24 hr event – 134 mm	ICOLD Bulletin 101
8	Embankment Design	Individual slopes between benches = 1:2.5 (V:H) Overall slope = 1:3 (V:H) Benches width = 7m	Knight Piésold
9	Footprint Area	22 ha	Knight Piésold
10	Final Elevation of TSF	1 089 mamsl	Knight Piésold
11	Height of TSF above lowest point	± 50 m	Knight Piésold
12	Minimum Stability Factor of Safety	Operational = 1.5 Closure = 1.5 Pseudo-Static > 1.1 (with 1:475 seismic event)	Chamber of Mines Guidelines, 1996, ANCOLD, 2012
13	Waste Type	Type 3	Regulation 634, 635
14	Dam Barrier System	Class C	Regulation 636
15	TSF Decant System	None	Knight Piésold
16	Return Water Requirements	None	Wesizwe Platinum Limited

2.1 GEOTECHNICAL INVESTIGATION

2.2 GEOLOGY

According to the published 1:250 000 scale geological series, sheet 2526 Rustenburg, the site is underlain by Kolobeng and Pyramid norite of the Rustenburg Layered Suite, western Lobe of the Bushveld Complex. The Rustenburg Layered Suite is categorised into four zones, namely the Upper, Main, Critical and Lower Zones. The site is located on the Main Zone. Typical weathering of norite produces upper black/dark brown, high clay content residual soils and lower sandy residual soils. The upper clayey soils have a high potential for expansion.

No geological features/structures are in the vicinity of the site on the geological map. According to Weinert's climatic N-value the site falls in an area classified as N<5. This is associated with more humid areas. Chemical weathering is the predominant mode of weathering as opposed to mechanical disintegration, which is associated with arid regions.

2.3 TOPOGRAPHY AND DRAINAGE

The planned TSF sites are flat and the surface topography slopes slightly in a southerly direction between elevations 1 050 m above mean sea level (mamsl) and 1 029 mamsl, at a slope of 2.4%. The site is situated within the A22F quaternary catchment.

2.4 SOIL PROFILES

The general soil profile at the TSF is summarised as follows:

- Colluvium covers the site and generally comprises dark brown/black sandy silty clay with a soft consistency. This material has an intact soil structure. However, a slickensided soil structure was occasionally encountered where the colluvium was deep. This horizon occurs to an average depth of 0.7 m except at 3 test pits where the colluvium occurs to depths of between 1.4 m and 2.4 m.
- Alluvium is present at the most southern and south-western parts of the TSF. The alluvium is generally dense gravelly silty sand to silty gravely sand with minor sub-rounded cobbles. The horizon was encountered below the colluvium to depths varying between 1.4 m and more than 3.1 m below the ground surface.
- Fine-grained residual norite underlies the colluvium. The horizon comprises sandy silty clay with a slickensided soil structure with a generally stiff consistency. The fine-grained residual norite depth is mostly between 0.5 m and 1.4 m, except at 3 test pits, where the horizon extends to depths of between 2.6 m and more than 3.0 m.
- Coarse-grained residual norite was encountered below the fine grained residual norite and is dense becoming very dense with depth. The soil comprises gravelly silty sand. The horizon extends to excavation refusal of TLB (between 2.1 m and 2.6 m) or maximum reach of TLB of more than 3.1 m.

- Norite bedrock was encountered at the northern and eastern parts of the site at six test pits. It occurs as highly weathered soft rock and refusal was at depths varying between 2.1 m and 2.6 m.

No groundwater seepage was encountered in any of the test pits.

Additional information can be found in the KP Geotechnical Investigation Report (KHH2542) and letter Wesizwe Foundation Geotechnical Testing Factual Letter Report (KP2659) hereby presented in **Appendix A**.

2.5 LABORATORY TEST RESULTS

Representative soil samples were taken from soil horizons in the test pits and submitted to SGS Matrolab in Pretoria for laboratory testing. A bulk sample of the waste rock stockpile was also collected. The laboratory tests conducted on the soil samples are as follow:

- Foundation Indicator tests (grading, Atterberg limits and clay content).
- Standard proctor compaction tests to determine maximum dry density and optimum moisture content.
- Specific gravity tests
- Dispersivity tests
- Modified AASHTO compaction tests to determine maximum dry density and optimum moisture content.
- California Bearing Ratio (CBR) tests.
- Falling head permeability tests on remoulded samples.
- Geochemistry tests.
- Shear box tests.
- pH and corrosivity tests.
- Clay Mineralogy tests (X-ray diffraction).
- Consolidation tests to determine settlement on remoulded samples.

The laboratory results are summarised in Table 2-2.

Table 2-2: Summary of Laboratory Test Results

SAMPLE ID		GRADING				ATTERBERG LIMITS (%)			GM	PE	USC	STANDARD PROCTOR COMPACTION		MOD AASHTO COMPACTION		CBR (%)			PH	CONDUCTIVITY	PEAK SHEAR STRENGTH PARAMETERS		Falling Head Permeability (cm/s)	MATERIAL DESCRIPTION
Sample No.	DEPTH (m)	Gravel	Sand	Silt	Clay	LL	PI	LS				MDD (kg/m^3)	OMC (%)	MDD (kg/m^3)	OMC (%)	98	95	93		mS/m	Friction Angle (%)	Cohesion (kPa)		
TP1/1	0.5 – 1.3	3	22	33	42	70	33	13.5	0.27	High	MH	-	-	-	-	-	-	-	-	-	-	-	-	Residual Norite (Fine)
TP3/1	1.5 – 3.0	53	31	9	7	60	27	10.5	2.01	Low	SM	-	-	2003	9	15	10	8	-	-	30	27	-	Residual Norite (Coarse)
TP6/1	0.6 – 2.6	2	27	31	40	68	34	13	0.25	Very high	MH	1266	-	-	-	-	-	-	-	-	-	-	9.96×10^{-7}	Residual Norite (Fine)
TP7/1	1.0 – 1.4	5	28	28	39	69	38	12.5	0.37	Very high	CH	-	-	1507	18	2	1	1	-	-	-	-	-	Colluvium
TP7/2	1.4 – 2.4	34	18	28	20	76	34	8	1.16	Medium	MH	-	-	-	-	-	-	-	-	-	-	-	-	Residual Norite (Fine)
TP9/1	0.5 – 1.3	2	22	33	43	69	33	14.5	0.24	High	MH	-	-	-	-	-	-	-	-	-	-	-	-	Residual Norite (fine)
TP10/1	1.1 – 2.6	44	44	10	2	32	10	6	1.94	Low	SC	-	-	2076	10	24	16	12	-	-	35	31	-	Residual Norite (Coarse)
TP12/1	0.9 – 3.1	55	30	10	5	64	27	10	2.11	Low	SM	-	-	1891	10	3	3	2	-	-	-	-	-	Alluvium
TP14/1	1.3 – 2.3	74	22	4	0	-	NP	0	2.51	Low	GW	-	-	-	-	-	-	-	-	-	-	-	-	Residual Norite (Coarse)
TP16/1	0.4 – 1.2	43	19	18	20	73	25	12	1.51	Medium	SM	1310	-	-	-	-	-	-	-	-	-	-	$4.23\times^{-7}$	Colluvium
TP17/1	1.6 – 3.1	7	49	31	13	66	17	8.5	0.76	Medium	MH	-	-	-	-	-	-	-	-	-	-	-	-	Residual Norite (Fine)
TP19/1	1.9 – 3.1	3	24	24	49	84	37	14	0.32	High	MH	1366	-	-	-	-	-	-	-	-	-	-	1.88×10^{-6}	Residual Norite (Fine)
TP20/1	2.0 – 3.0	4	27	21	48	78	48	15.5	0.35	Very high	CH	-	-	-	-	-	-	-	-	-	-	-	-	Residual Norite (Fine)
TP21/1	0.4 – 1.4	-	-	-	-	-	-	-	-	-	-	1394	28.5	-	-	-	-	-	-	-	-	-	-	Residual Norite (Fine)
TP22/1	0.4 – 1.4	0	35	28	37	64	32	15	0.30	High	CH	1374	27.5	-	-	-	-	-	7.64	26.60	-	-	$3x10^{-7}$	Residual Norite (Fine)
TP23/1	0 – 1.0	2	35	26	37	71	31	15	0.36	High	CH	-	-	-	-	-	-	-	-	-	-	-	-	Colluvium
TP24/1	0 – 0.7	1	29	30	40	65	32	15	0.30	High	CH	1407	22.3	-	-	-	-	-	7.62	32.30	-	-	$1.5x10^{-6}$	Colluvium
TP26/1	0 – 0.8	2	32	28	38	67	27	13	0.34	High	CH	1413	27.1	-	-	-	-	-	7.71	34.60	-	-	$2.6x10^{-7}$	Colluvium
TP28/1	0.2 – 0.8	2	26	31	41	66	26	13	0.27	Medium	CH	1373	25.2	-	-	-	-	-	7.66	27.60	-	-	$4.2x10^{-7}$	Residual Norite (fine)
1320	-	0	16	79	5	-	NP	0	0.11	Low	CL	1918	-	-	-	-	-	-	-	-	-	-	1.61×10^{-4}	Tailings

SAMPLE ID		GRADING				ATTERBERG LIMITS (%)			GM	PE	USC	STANDARD PROCTOR COMPACTION		MOD AASHTO COMPACTION		CBR (%)			PH	CONDUCTIVITY	PEAK SHEAR STRENGTH PARAMETERS		Falling Head Permeability (cm/s)	MATERIAL DESCRIPTION
Sample No.	DEPTH (m)	Gravel	Sand	Silt	Clay	LL	PI	LS				MDD (kg/m³)	OMC (%)	MDD (kg/m³)	OMC (%)	98	95	93		mS/m	Friction Angle (%)	Cohesion (kPa)		
1321	-	52	38	7	3	30	10	6	2.07	Low	SC	-	-	2390	4	152	57	30	-	-	-	-	-	Waste Rock

R denotes TLB refusal.

Notes:

LL	:	Liquid Limit	PE	:	Potential Expansiveness	OMC	:	Optimum Moisture Content	SC	:	Clayey sand
PI	:	Plasticity Index	USC	:	Unified Soil Classification	MH	:	Sandy silty clay	SM	:	Silty sand
LS	:	Linear Shrinkage	CBR	:	California Bearing Ratio	SM	:	Silty sand	GW	:	Well graded gravel
GM	:	Grading Modulus	MDD	:	Maximum Dry Density	CH	:	Clay of high plasticity	CL	:	Inorganic clays of low to medium plasticity

2.6 FOUNDATION GEOTECHNICAL APPRAISAL

2.6.1 TSF

The TSF site is covered by 1.5 m thick colluvium and fine residual norite which comprises silty clay. The upper colluvium has an average thickness of 0.7 m and mostly has a soft consistency. The underlying fine residual norite mostly has a stiff consistency and is slickensided which is indicative of an expansive soil.

The soft colluvium is highly compressible and unsuitable as a foundation layer as it will cause excessive strain in the geomembrane due to the load of the tailings, which will settle more in the centre than on the side, further enhanced by locally differential settlement due to the depth variation of the layer. It is therefore recommended that the material should be excavated to a minimum depth of 0.7m below the ground surface within the entire TSF footprint and removed to spoil and re-used as low permeability medium in the barrier system in two 150 mm layer compacted to 93% Proctor density at a moisture content of -0 +2% of optimum moisture content (omc). The material should be kept close to omc due to its high swell potential. The falling head permeability testing highlighted a permeability of colluvium and the fine residual norite lower than required clay barrier specified in Minimum Requirement (1998) with an upper value of $1.5x10^{-6}$ cm/s.

2.6.2 STARTER WALL

The starter wall will require to be founded on competent ground, it is therefore recommended to remove until encountering the coarse residual norite or the excavation hits refusal, replacing the material with dump rock, increasing the stability of the TSF.

2.7 WASTE ROCK DUMP

The waste rock dump is located to the north of the proposed TSF (refer to Plate 9). The dump is approximately 200 m in length by 120 m in width with an average height of 4 m. A sample of the waste rock dump material was taken for laboratory testing. It has a high Modified AASHTO MDD of 2 390 kg/m³ at an OMC of 4%. The material has a COLTO classification as a G5 material, although it is possible that poorer quality materials may be present within the stockpile. Oversized material will have to be removed, as it is not suitable for construction.

The waste rock can be used for starter wall construction and is a better-quality material than the alluvium. There is also a large quantity of waste rock available for construction.

2.8 PRIMARY CLAY LAYERS

From the geotechnical investigation it was indicated that the site is covered by colluvium and residual norite. Falling head permeability tests have highlighted a permeability of 1.0×10^{-6} cm/s which is in line the maximum permeability of a clay layer specified in a Class C barrier system. The material will be ripped and stockpiled and consequently placed in 2 layers of 150 mm compacted to 95% Standard Proctor density at a moisture between 0 and 2% of optimum moisture content. Although the potential expansiveness is medium to high, the site is characterised by a coarse grained residual norite or alluvium layer, which are characterised by a low expansiveness and they will not be disturbed, with a depth of 1.5 m at its shallowest which will reduce seepage from the barrier system into the ground.

2.8.1 CHEMICAL TEST RESULTS

The results of the chemical dispersivity tests yielded relatively high Cation Exchange Capacities but low exchangeable sodium percentages. The results plotted in Figure 2-1 indicates that the material is generally non-dispersive with one result (TP26) as marginally dispersive.

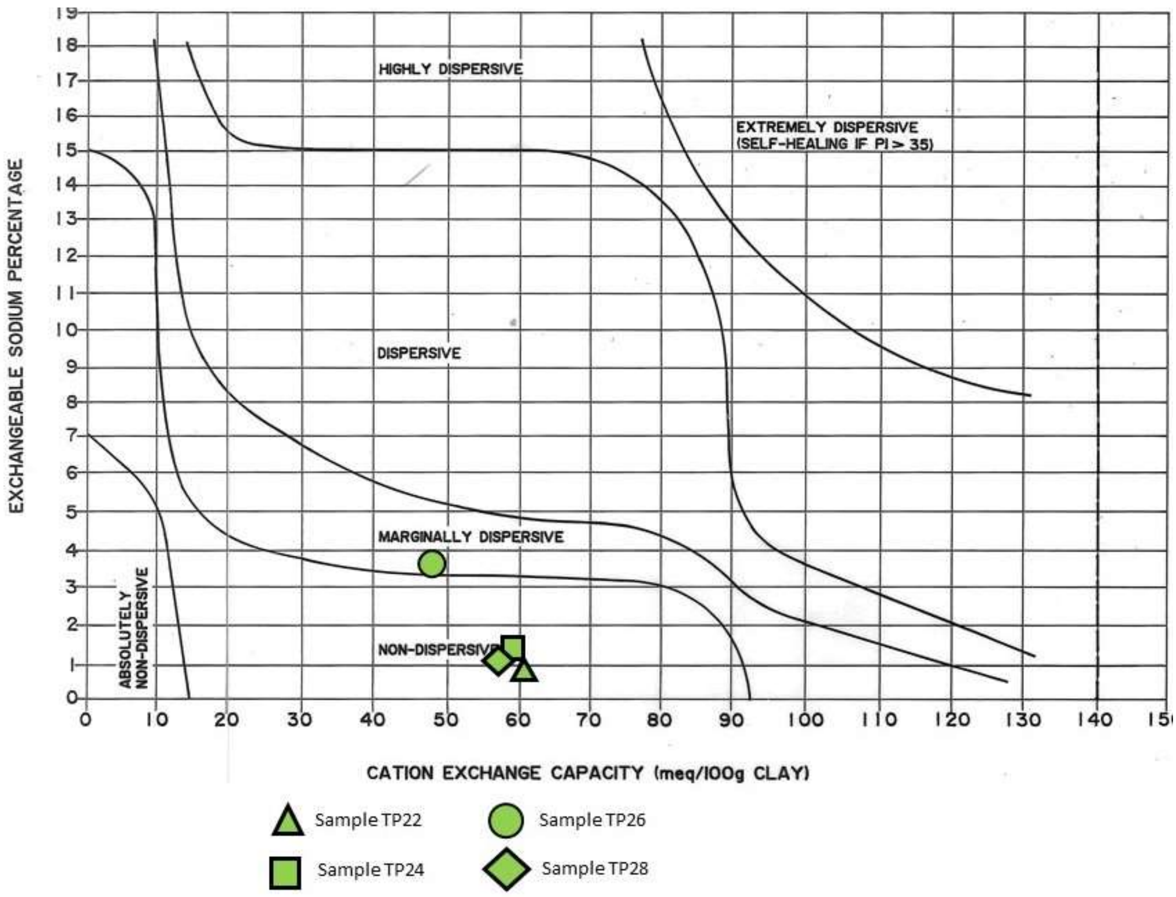

Figure 2-1: Chemical Dispersivity Tests

The results indicate very similar composition between the colluvium and the fine-grained residual norite, where the samples comprise generally montmorillonite clay with abundant quartz, and to a lesser extent k-feldspar and plagioclase.

3.0 GROUNDWATER STUDY

Geo Pollution Technologies (Pty) Ltd (GPT) where appointed to conduct a groundwater impact assessment for the TSF in 2020. The complete report is presented in **Appendix B**.

The objectives of the study were:

- Determine potential groundwater impacts from the TSF considering the proposed barrier system design to capture TSF seepage
- Assess the unsaturated and saturated flow below the TSF
- Design a monitoring network for the planned TSF

3.1 HYDRAULIC CONDUCTIVITIES

The following was taken from the GPT Groundwater Impact Assessment Report (March 2020), Chapter 9.3:

The average hydraulic conductivity (K) of the clayey soils is in the region of 0.09 m/d. The shallow weathered aquifer has a hydraulic conductivity of between 1.2x10^{-2} and 5.75x10^{-5}m/d. These values are given in the geohydrological assessment report (Africon 2008) and was measured in situ on site using double ring infiltrometer tests and falling head tests respectively. The K-value for the preferential pathway encountered in MBH04D was 1.47 m/d. The higher K-value could act as a preferential pathway for groundwater and contaminant migration.

3.2 POTENTIAL GROUNDWATER IMPACTS RESULTING FROM A LEAKING BARRIER

The following was taken from the GPT Groundwater Impact Assessment Report (March 2020), Chapter 10.8:

It is understood that the TSF will be lined using a Class C barrier system. The typical barrier system includes the following layers from excavation level upwards:

- *Substrate preparation layer: The substrate will be ripped and re-compacted to 95% MOD AASHTO with a moisture content of -2 to +2% of optimum moisture content.*
- *Subsoil Drainage Layer: A drainage layer is installed below the barrier system to relieve pressure that may be caused by shallow ground water. It also collects any leakage that may penetrate the barrier system.*
- *Primary low permeability layer: 2 x 150 mm layers of clay compacted to 98% Standard Proctor with a moisture content of +1 to +3% of optimum moisture content in order to have a permeability co-efficient (k) of less than 1x10^{-6} cm/s.*

- *Primary geomembrane layer: 1.5 mm High-density polyethylene (HDPE) geomembrane layer.*
- *Protection layer: 100 mm layer of fine sand that will protect the geomembrane against damage.*
- *Leachate collection layer: 300 mm thick finger drains of geotextile covered aggregate with HDPE pipe drainage network.*

Should the lining remain undamaged, no impact on groundwater receptors can be expected. But linings are often damaged during construction or operations and leakage to the subsurface are thus possible. Three scenarios were modelled to cater for leakage, namely a 10% and 50% and 100% leakage. As dry deposition of material will be done, the only flow to the TSFs is recharge from rainwater. Recharge from rainfall to the TSF was estimated at 20% of mean annual rainfall. The scenarios modelled were thus:

Table 3-1: Potential Groundwater Impacts

Scenario	Leakage (%)	Effective recharge (m/day)	Option 1 leakage volume (m^3/day)	Option 2 Leakage Volume (m^3/day)	Option 3 Leakage Volume (m^3/day)
Minor liner leakage	10	0.00003	6.95	5.8	11.5
Major liner leakage	50	0.0016	37.1	31.0	61.4
No liner	100	0.0003	69.5	58.2	115.1

3.2.1 RECOMMENDATIONS

The following recommendations are put forward:

- A system of storm water drains must be designed and constructed to ensure that all water that falls outside the area of the TSF is diverted clear of the deposit.
- The boreholes MONBH1 to 6 should be added to the current monitoring network. These should be monitored on a quarterly basis, monitoring should start prior to construction, and continue during the operational phase of the TSF for the parameters analysed in the groundwater study report. The following parameters should be monitored:
  - Abbreviated analysis (pollution indicators)
  - Physical Parameters:
    - Groundwater levels
  - Chemical Parameters:
    - Field measurements:
      - pH, EC

- Laboratory analyses:
  - Major anions and cations (Ca, Na, Cl, SO4)
  - Other parameters (EC)
  - Full analysis
- Physical Parameters:
  - Groundwater levels
- Chemical Parameters:
- Field measurements:
  - PH, EC
- Laboratory analyses:
  - Anions and cations (Ca, Mg, Na, K, NO3, Cl, SO4, F, Fe, Mn, Al, & Alkalinity)
  - Other parameters (pH, EC, TDS)

- The monitoring boreholes should be sited using geophysical methods in order to identify geological structures that may act as preferential flow paths for contaminant transport.
- The monitoring boreholes drilled into the inferred fault should be constructed so that the flow of the fault can be compared to the flow of the host rock material.
- Monitoring boreholes drilling should be supervised by a qualified hydrogeologist and care should be taken to accurately log the geology during drilling and construct the boreholes appropriately
- The aquifer parameters should be measured by conducting an aquifer test (pump test =, slug test etc.) on each of the newly drilled boreholes. 24-Hour pumping tests are recommended. This information can be used to update the numerical model with accurately measured parameters.
- A hydrocensus within a radius of 5 km around the boundary of the TSF site should be conducted every 2 years.

A re-evaluation of the risk to the aquifer should be conducted every 2 years.

4.0 TSF DESIGN

4.1 GENERAL

The Bakubung TSF consists of the following design elements:

- A 1 m high toe wall comprising of rockfill from the existing waste rock dump founded on the coarse residual norite providing containment during the early deposition into the facility.
- A class C barrier system beneath the TSF, paddocks and evaporation ponds.
- A network of seepage collection drains constructed in the basin of the TSF and immediately upstream of the toe wall
- Toe paddocks to contain runoff and silt eroded from the outer slopes of the facility
- A concrete lined solution trench to channel filter discharge and runoff from the outer slopes to the evaporation pond.
- Two evaporation ponds with two compartments positioned at the lowest point of the solution trenches situated at the South Eastern side of the TSF to contain the seepage discharge.
- A perimeter access road to allow suitable access around site
- A stone pitched clean water diversion channel to divert clean stormwater around the TSF

4.2 CAPACITY

4.2.1 INTRODUCTION

A 2019 survey of the existing site conditions was made available to KP by Wesizwe for the purposes of this design. Infrastructure situated within the boundaries of the proposed site for the TSF will need to be cleared for the development of the TSF as illustrated in Figure 4-1 and Figure 4-2 (Drawing No. 301-00509/10-001 and Drawing No. 301-00509/10-002). The TSF is required to contain an average tonnage profile of 1 Mtpa for a maximum period of 7 years. The tailings delivery system will be using conveyor coming from the filter plant on skid footings. From the conveyor, wheeled & telescopic stackers and then dozers will be implemented for the final placing.

Due to this being a filtered tailings facility, it has been assumed that the facility will be constructed by a system of conveyors and spreader, the TSF will be constructed in 7 m lifts until the final height is reached. The equipment will stack the tailings and the tailings will be spread and compacted using mobile equipment. To achieve the required capacity a total of seven lifts will be required. Each lift will have a 7 m wide bench, a typical section of this can be seen on drawing No. 301-00509/10-005.

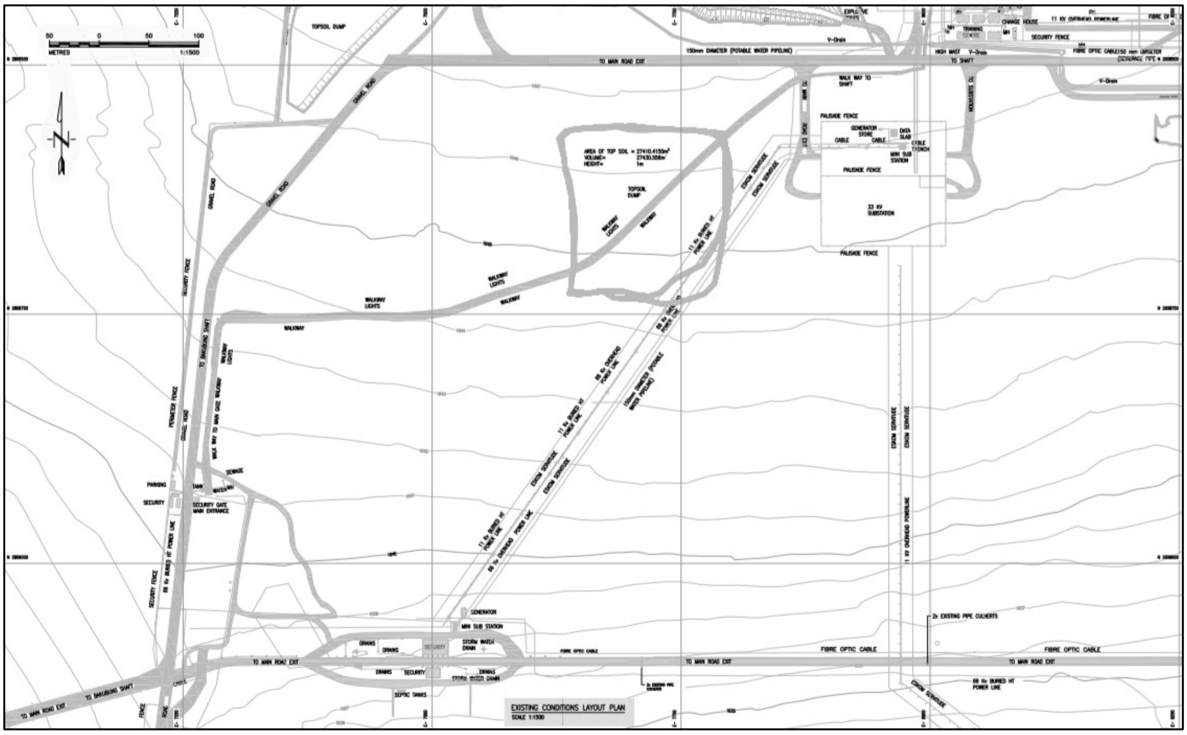

Figure 4-1: Existing Site Conditions

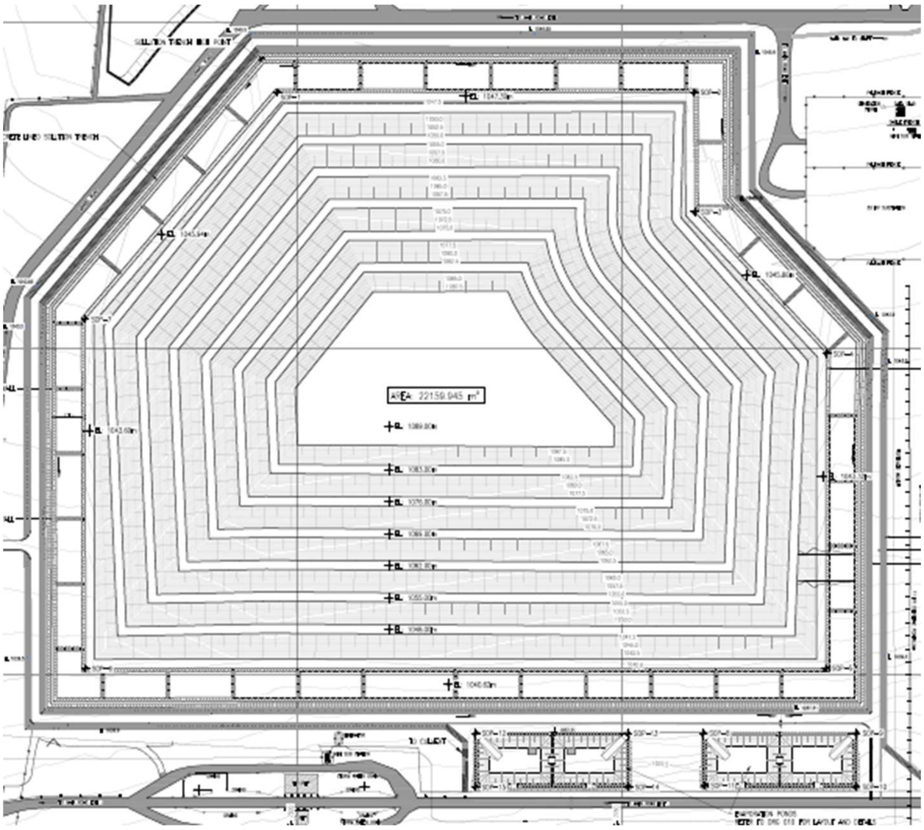

Figure 4-2: Proposed TSF Layout

4.2.1.1 TAILINGS PRODUCTION

The TSF is required to contain an average tonnage profile of 1 Mtpa for a minimum period of 7 years.

A summary of the size parameters of the TSF and evaporation pond is given in Table 4-1 and Table 4-2.

Table 4-1: TSF Size Parameters

Parameter	Value
Area within the toe wall	21.6 Ha
Final Elevation of TSF	1 090 mamsl
Area of TSF at final elevation	2.8 Ha
Height of TSF above lowest point	± 50 m
Rate of rise	3.4 m – 20 m
Storage capacity available	7.6 Mt @ 1 090 mamsl

Table 4-2: Evaporation Pond Size Parameters

Parameter	Value
Area within the crest	0.88 Ha
Depth of evaporation pond above lowest point	0.9m
Storage capacity available	27 128 m^3 @ Max level

4.2.2 DESIGN CONSTRAINTS

The construction and geometry of the facility will be determined by the following constraints:

- Tonnages to the facility,
- Delivery system of tailings (assumed to be conveyors and spreader),
- Properties of the tailings,
- Short- and long-term stability,
- Infrastructure requirements.

4.2.2.1 EQUIPMENT OPERATING ON THE TSF

The movement and constraints of the mechanical equipment has a major influence on the geometry of the facility and the loading will affect the behaviour of the barrier system. The equipment considered during the capacity modelling are conveyors, telescopic stackers and dozers. It has been assumed that the maximum operating slope for the spreader is 1:20 and for the conveyor 1:10. At this stage, a Caterpillar D6 dozer with a ground pressure of 56 KPa has been considered by the Client in the operation while telescopic stackers have not yet finalised, however they will be on rubber tyres, with an estimated ground pressure of 350 KPa.

4.2.2.2 TAILINGS PROPERTIES

Based on knowledge of the area and work undertaken in nearby operations, the following properties were used for the modelling of tailings.

Table 4-3: Tailings Properties

Parameter	Value
Density	1.6 t/m^3
Friction angle	32^o
Cohesion	0 kPa

4.3 UNDERDRAINAGE LAYER AND MONITORING SYSTEM

Although no groundwater was encountered during the geotechnical investigation, the under-drainage layer will form part of the barrier system to collect any leakage that may penetrate the barrier system and any water seeping underneath the TSF.

The under-drainage layer will comprise of finger drains at 50 m centre to centre. The finger drain will comprise a 110 mm diameter slotted pipe connecting into a main 250 mm solid pipe, a nonwoven geotextile wrapped around the pipe and clean river sand of at least 150 mm thick. The pipes have been designed for the normal leakage rate considering a 10% capacity and a factor of safety higher than 3, whilst a 50% diameter water level and a factor of safety 5 for the highest leakage rate.

The finger drains will be arranged in a herringbone system and the trenches will be 300 mm wide by 300 mm deep. As the drainage layer is below the compacted clay layers of the barrier system, the drainage material will need to be from a clean source to avoid pollution of groundwater. The herringbone system will discharge into the solution trenches running outside the perimeter of the TSF monitoring the leakage rate and the efficiency of the barrier system.

For the evaporation pond the same herringbone drain will be applied, with pipes daylighting in a monitored sump.

4.4 BARRIER SYSTEM DESIGN

4.4.1 REGULATORY REQUIREMENTS AND SITE CLASSIFICATION

The regulatory requirements governing the development of a landfill includes but is not limited to the following:

- National Environmental Management: Waste Act, 2008 (Act 59 of 2008) (NEMWA);
- Waste Classification and Management Regulations (GN R634 of 23 August 2013);

- National Norms and Standards for Assessment of Waste for Landfill Disposal (GN R635 of 23 August 2013);
- National Norms and Standards for Disposal of Waste to Landfill (GN R636 of 23 August 2013); and

4.4.2 TAILINGS CLASSIFICATION

A sample of the tailings was submitted to Waterlab (Pty) Ltd for analysis and the results to EnChem Consultants to classify the tailings. Additional information and analysis are presented in **Appendix C**.

The following was noted from the analysis:

- The platinum tailings are neutral with paste pH of 7.4, which is within the landfilling limits of pH >6 and <12.
- The final pH of the leach solution was measured as 7.4, i.e. the sample only slightly increased the starting pH of the distilled water used to extract the sample.
- The average concentrations of cobalt, copper, manganese, nickel and vanadium, exceed the lowest threshold value of TCT0 but are all less than TCT1.
- None of the species of concern leached at a concentration greater than their LCT0 value.
- According to waste regulations GN 635, the sample classifies as a **Type 3 waste**, as for some elements TC is greater than TCT0 but ≤TCT1. However, no species leached at concentrations ≥LCT0, which is equal to the South African Drinking Water Standard or, if a value is not defined in SA, an International Standard. In addition, the sample leached very low soluble solids as the TDS value was measured as only 60 mg/l.
- The moisture content of the sample was 12.5% and would classify as a dry waste as the value is well below the landfill limit of 40%.
- The sample consists of the following major elements, i.e. at concentrations >1%; Al, 2.12%; Ca, 1.68%; Cr, 1.36%; Fe, 6.48%; Mg, 8.32%; and Si. 14.88%. These elements will be present as their oxides.

GN 636 requires a Class C barrier system for a Type 3 waste. A typical barrier system is illustrated in Figure 4-3.

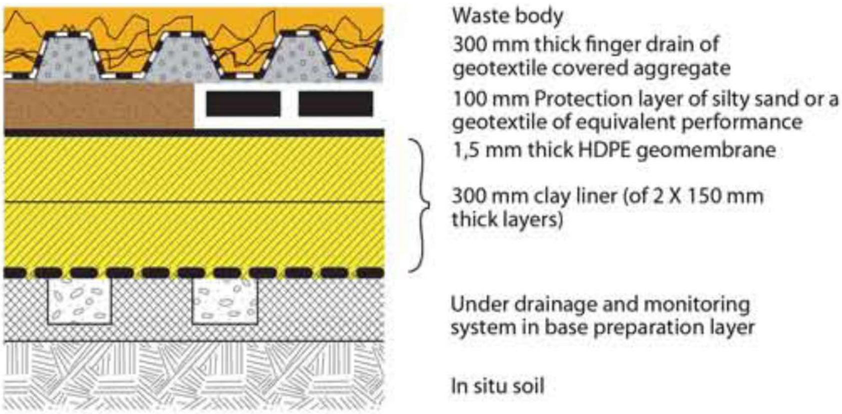

Figure 4-3 Class C Barrier System for Type 3 Waste

The regulations allow the use of alternative materials such as geosynthetics composite drainage for drainage, geotextiles for protection and geosynthetics clay liner (GCL) for compacted clay liner proven to exhibit equivalent performance to the natural materials indicated.

4.4.3 BARRIER SYSTEM DESIGN

4.4.3.1 TSF

The Class C barrier system proposed for the TSF is presented in Figure 4-4.

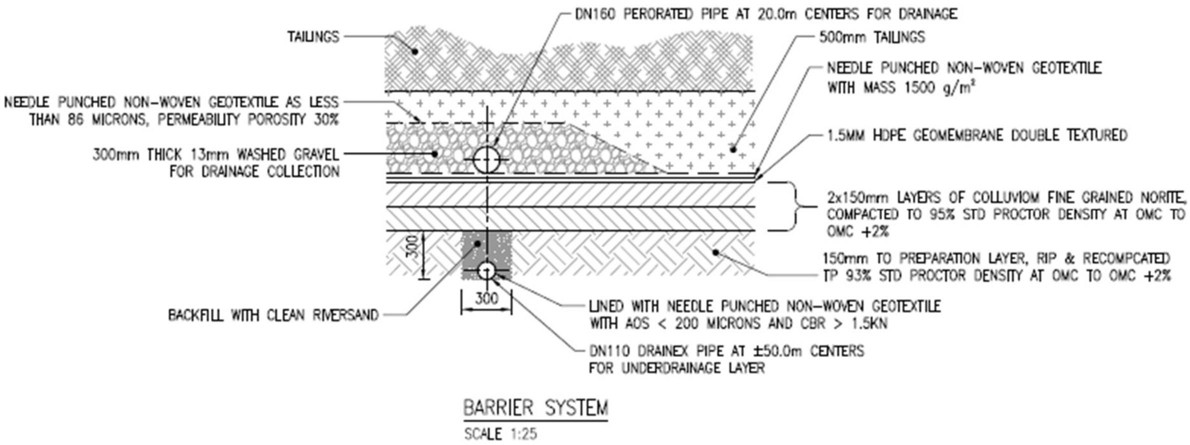

Figure 4-4 TSF – TSF Barrier System Detail

The barrier system will extend over the toe wall including the paddocks, ending in an anchor trench along the crest of the outer paddock berm.

From the excavation upwards the following notes on the barrier design are applicable:

- The base of the excavation shall be ripped and recompacted to 93% Standard Proctor density and moisture content between 0 and 2% of optimum moisture content;
- Underdrainage monitoring system shall be constructed excavating 300mm for 300mm wide, 110mm perforated pipe wrapped in a needlepunched nonwoven geotextile with an apparent opening size (AOS) of less than 200 microns (SANS 12958) and a CBR of not less than 1.5 kN (SANS 12236) encased in clean riversand;
- The compacted clay liner (CCL) shall be constructed in 2 x 150 mm thick layers compacted to 95%Standard Proctor density at a moisture content between 0 and +2% of optimum moisture content. The in-situ colluvium and fine grained norite shall be selected and used for the compacted clay liner;
- The geomembrane shall be a 1.5mm thick HDPE dual textured manufactured in accordance with GRI-GM13 (2019);
- The protection geotextile shall be a nonwoven needlepunched geotextile with a unit mass of 1500gr/m^2 in accordance with SANS 9864 and a CBR 13KN accordance with SANS 12236 made from polyester continuous filament;

- The drainage layer will comprise 160mm diameter slotted pipe surrounded by 13mm washed stone and covered by a needlepunched nonwoven geotextile with an apparent opening size of less than 86µm (SANS 12956) and Minimum Hydraulic Conductivity of $2x10^{-7}$ m/s (SANS 11058) and 30% porosity;
- 500mm cover layer of filtered tailings will be deposited throughout the entire facility as a cover protection against UV degradation of the geotextile and to allow access to the facility by operating machineries . No equipment shall be allowed to access unprotected areas. The cover layer shall be installed within 3 months from installation of the geotextile. Any access to the facility shall be build with a gradient of 1:5 minimum.

By the toe wall, the barrier system will require a separation geotextile between the compacted clay liner and the rockfill. A 150mm layer of coarse grained norite compacted to 90% Proctor will be placed over the rockfill and a nonwoven geotextile of 1500gr/m^2 will act as a separator between the coarse grained norite and the CCL to avoid migration of the CCL in the rockfill.

4.4.4 PADDOCKS

The paddocks will require to have the same barrier system as the TSF, however the 500mm protection tailings is replaced by a 150mm HDPE geocell filled with soil and 3% cement in order to provide protection to the barrier as presented in Figure 4-5, allowing maintenance operations within the paddocks and protection against weathering.

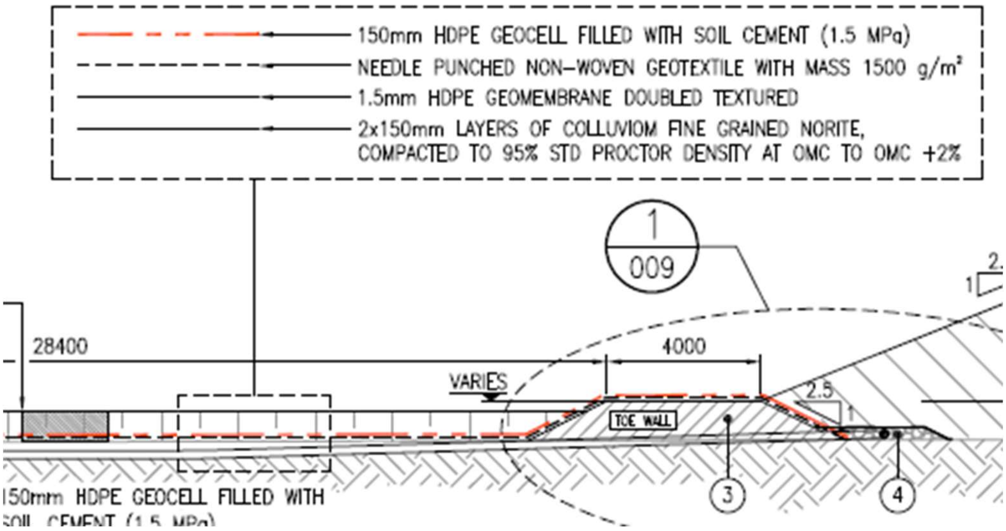

Figure 4-5 Paddocks - Typical Barrier System Detail

4.4.5 EVAPORATION POND

The Class C barrier system proposed for the evaporation pond is presented in Figure 4-6 Evaporation Pond - Typical Barrier System Detail below

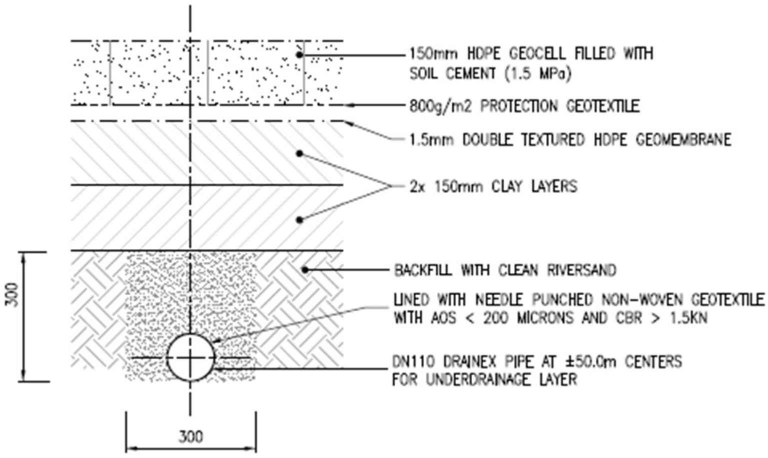

Figure 4-6 Evaporation Pond - Typical Barrier System Detail

From the excavation upwards the following notes on the barrier design are applicable:

- The base of the excavation shall be ripped and recompacted to 93% Standard Proctor density and moisture content between 0 and 2% of optimum moisture content;
- Underdrainage monitoring system shall be constructed excavating 300 mm for 300 mm wide, 110 mm perforated pipe wrapped in a needlepunched nonwoven geotextile with an apparent opening size (AOS) of less than 200 microns (SANS 12958) and a CBR of not less than 1.5 kN (SANS 12236) encased in clean riversand;
- The compacted clay liner (CCL) shall be constructed in 2 x 150 mm thick layers compacted to 95% Standard Proctor density at a moisture content between 0 and +2% of optimum moisture content. The in-situ colluvium and fine grained norite shall be selected and used for the compacted clay liner;
- The geomembrane shall be a 1.5 mm thick HDPE dual textured manufactured in accordance with GRI-GM13 (2019);
- The protection geotextile shall be a nonwoven needlepunched geotextile with a unit mass of 800 gr/m^2 in accordance with SANS 9864 and a 6 mm hole in accordance with SANS 13433 made from polyester or polypropylene fibres;
- 150mm HDPE geocell characterised by a cell wall length of 356 mm and a tensile strength of 7 KN/m in accordance with ISO 13426-1 filled with a soil cement characterised by a compressive strength of 1.5 MPa at 100% MOD AASHTO.

4.4.6 VENEER STABILITY

The barrier system will be placed over a 1:2 slopes, at the toe wall of the TSF and in the paddocks, which will be subject to shear forces due to the weight of the tailings on the TSF side and construction and operation equipment on the evaporation ponds.

Shear interface testing (**Appendix D**) reports a residual friction angle between the compacted clay liner and the double textured geomembrane of 18.2° (adhesion omitted) which generates a factor of safety higher than 1.5 with no machinery present, while for the evaporation pond a generic backhoe loader (gross operating weight 11 ton) will reduce the factor of safety to 1.18 in the condition of driving down the ramp with a 1 in 5 slope which is acceptable considering the transitory condition.

The geomembrane will not be under tension as the lowest shear interface will be between the geotextile and the tailing material, however a nominal anchor trench 0.5 m away from the crest, 0.6m wide and 0.6m deep is required to avoid wind uplift and a neater tie-in with the ground.

In the evaporation pond the geocell will be anchored in an independent anchor trench, while the geomembrane and geotextile will require an anchor trench 0.5 m away from the crest as above, resulting in both being unloaded due to the lowest interface friction between the geocell and geotextile (12°).

4.4.7 EFFECT OF MACHINE LOADING ON THE BARRIER SYSTEM

The dozer and the reach stackers operating on the facility will generate ground pressure on the barrier system on surface of between 50 kPa and 350 kPa which will be spread over the 500 mm tailings to an acceptable bearing pressure of 9 kPa and 12 kPa which are acceptable for the bearing capacity of underlying soil based on the CBR value highlighted by the geotechnical investigation.

4.4.8 ALTERNATIVE ELEMENTS OF PROVEN EQUIVALENT PERFORMANCE

The use of a geomembrane requires a protection layer to avoid damaging by the drainage layer above, which is provided by a soil layer (generally sand). However, the placing of such protection layer requires stringent CQA in order to avoid damaging the geomembrane while installing the layer as well as the sourcing of the material which needs to be of a very strict particle size distribution.

Testing undertaken by Queen's University in Canada along the years have proven that the using a 2 200 gr/m^2 protection geotextile beneath a 50 mm and 25 mm stone aggregate induce strain in geomembrane of 10% and 5% at 250 kPa (Brackman, 2008), furthermore Hornsey (2013) highlighted that the raw material in the protection geotextile have an impact on the protection efficiency, highlighting that a continuous polyester nonwoven geotextile with a mass of 800 gr/m^2 limit the strain in the geomembrane to 5%, while a polypropylene staple fibre with the same mass results in 12% using a 20/75mm aggregate.

Based on the above, it is reasonable to assume that a geotextile protection with a mass of 1 500 gr/m^2 beneath the drainage stone layer of 13mm will limit the strain in the geomembrane to 3% at a design temperature of 25°. The above statement is supported by recent testing undertaken by TRI Australasia on a similar barrier system where a 540 gr/m^2 continuous filament polyester geotextile was placed between a 20 mm gravel and 1.5 mm HDPE geomembrane, resulting in strain of 2.5%.

As part of the CQA Plan, testing on the proposed alternative will be performed using actual materials to be in used in the construction.

4.4.9 BARRIER SERVICE LIFE ASSESSMENT

The service life for a tailings storage facility barrier system can be considered the end of tailings disposal due to the recharge of the phreatic surface due to excess moisture plus, the rainfall until the facility is closed by mean of a capping layer reducing rainfall infiltration. The current facility is designed to reach capacity after 7 years, however closure generally can take much longer to be implemented (100 years).

For a geomembrane the main factors affecting service life is UV exposure as well as temperature. Whilst the geomembrane will be covered during construction, therefore UV exposure can be omitted, temperature will have a degradation effect. The temperature of the flow is assumed of 25° based on monitoring undertaken in a facility nearby where monitoring on the drainage outlet has shown an average temperature of 24.76° over a period of one month in summer. Based on research conducted by the Geosynthetics Institute in USA "Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions" originally published in 2005 and latest updated in 2011, it is reported that a geomembrane in unexposed conditions at 25° will have a design life of more than 250 years. It shall be noted that such time is for the geomembrane to reach the so called "half-life", meaning the antioxidant in the geomembrane have reached 50% of their original value.

In tailings storage facility the stress of deposited tailings is near isotropic, therefore the pressure on the wrinkles will cause the wrinkle to collapse causing high stresses in the folding points, which could lead to stress-cracking of the geomembrane. The CQA Plan address the installation of the liner in favourable temperature (maximum 25°).

The protection geotextiles degradation phenomena can be due to weathering, microbiological, chemical and oxidations. In case of protection geotextiles, considering the material is stored in a dry and covered placed, once installed, covered within 24 hours, weathering degradation is a temporary condition which does not affect the long-term performance of the geotextiles, as well as microbiological as geotextiles used for protection function are manufactured from virgin fibres or are characterised by a high molecular weight (> 20 000 g/mol). Chemical degradation affects polyester based geotextile due to hydrolysis, whereby the ph of the solution and the temperature will degrade the polymer. From analysis of the tailings the ph is 7.4 and considering a temperature on the geotextile of 25°, the degradation phenomena reaching the half-life is in the order of more than 400 years, which is deemed acceptable. Oxidations phenomena occurs in polyolefin such as polypropylene geotextiles; from oxidation tests ISO 13438, the retained strength at 50% should require a test duration of more than 112 days.

4.4.10 COMPATIBILITY OF BARRIER MATERIAL WITH THE WASTE STREAM

SmecTech Research Consulting commented on the compatibility of the available clay material as a compacted clay liner (CCL) for the barrier system. The report is attached in **Appendix E.**

The following conclusions were made:

1. The clay material can be used as a CCL for the barrier system. An assessment of the mineralogy, chemistry and geotechnical properties of clay materials, as well as leachates associated with the Bakubung tailings facility, indicate only minor incompatibility with respect to cation exchange.

2. This is largely due to the fact that the bulk clay material averages about 20 wt% smectite (montmorillonite) and contains a significant proportion of exchangeable magnesium (Mg2+). As such, the low RMD1/2 value of the leachate is probably not of serious concern for the hydraulic performance of the compacted clay liner dam wall.

Calculated saturated hydraulic conductivity values, using parameters determined from the leachate chemistry, geotechnical characterisations and site plans, should be adequately low for economic operation of the tailing's facility. This of course assumes that the material is compacted to its maximum achievable density at its optimal moisture content.

With regards to the low RMD1/2 values reported for the tailings leachate, normally incompatibility is associated with the exchange of Ca^{2+} for Na^+ in sodium bentonites. However, in the case of the clay materials to be used as compacted clay liners, none of the samples have appreciable exchangeable sodium, and instead already contain considerable (~16%) exchangeable Mg^{2+}.

The potential incompatibility due to low RMD1/2 values can be expected to be somewhat counteracted because of the lack of Na^+ in both the leachate and the clay material, and the fact that leachate pH is near the pKa for calcium carbonate precipitation.

The degree to which the RMD1/2-related incompatibility may be offset is unknown. However, Ca^{2+} can be expected to replace Mg^{2+} (and other base cations) on the clay exchange complex over time, and while this would result in a limited increased hydraulic conductivity, it can be expected to also improve plasticity and activity parameters of the clay material.

Ancillary reactions of the leachate as it permeates the clay materials, including Ca^{2+} for Mg^{2+} and precipitation of calcium and magnesium carbonates may affect geotechnical characteristics of the compacted clay over time. Strict compliance to compaction of multiple lifts of the clay material to maximum dry density at optimum moisture content should assist in ensuring construction of a quality compacted clay liner.

4.4.11 ESTIMATED LEAKAGE RATES

Tailings is characterised by a low permeability, further the upstream process of filtering the tailings before delivery, reduces the saturation content, which does not suggest a build up of a phreatic surface within the tailings dam.

Research by Rowe (2016) and Joshi (2016) has highlighted that leakage rate in the order of 3L/day measured for a 10mm hole in a geomembrane with a pressure of 1 500 kPa with tailings characterised by a permeability of 1×10^{-7} m/s and different underlying material (1×10^{-7} m/s – 1×10^{-5} m/s). This is due to the migration of fine tailings through the hole which have a “sealing effect”. Considering a density of 5 holes per hectare, the leakage rate can be considered to be of 15 L/ha/day. The presence of a geotextile protection increases the leakage rate due to the transmissivity of the geotextile, however it was counteracted by clogging, corresponding to a leakage rate of 40 L/ha/day.

A further consideration is necessary as in the early stages of the operation, where the pressure on the liner is minimum, wrinkles are still present. Calculating the leakage rate based on Rowe (2012 and 2018), assuming a water head on the liner of 100mm considering a wrinkle alongside the drainage pipe, the leakage rate considering a wrinkle of 160m (length of the drainage pipe) could vary between 166 L/ha/day and 499 L/ha/day.

In the long term, considering that most of the holes will have been sealed by tailings and the pressure has reduced the height and its width to 50% of its original and length to 20 cm, the leakage rate considering 1 hole per wrinkle would be of 21 L/ha/day which adding to the leakage from a hole in the geomembrane in direct contact with CCL results in a total leakage of 61 L/ha/day.

For the paddocks and the evaporation ponds, a 20 m of interconnected wrinkles was considered with a 0.3m and 1 m phreatic surface (based on normal operating conditions) for the evaporation ponds and 0.1m and 0.5m for the paddocks. The estimated leakage rates for the liner system could vary between 21 l/ha/day and 104 l/ha/day for the paddocks while for the evaporation ponds could vary between 62 l/ha/day and 208 l/ha/day. Post closure the leakage rate in the evaporation ponds should reduce due to the closure of the TSF while for the paddocks it is estimated to remain constant as it is based on the runoff from the TSF which in the design of the paddocks was assumed to be 1. A summary is presented in Table 4-4.

The subsoil drain system will be monitored as part of the operations and maintenance plan in order to assess the amount of leakage beneath the barrier system.

Table 4-4: Leakage Rate Summary

	TSF	Paddocks	Evaporation ponds
Early stage (l/ha/day)	166-499	21-104	62-208
Half-life (l/ha/day)	61	21-104	62-208
Closure (l/ha/day)	<61	21-104	<62

Such results are well below the groundwater model presented in Section 3.0 confirming the adequate management of the risk caused by the tailing on the receiving environment.

4.5 SURFACE WATER MANAGEMENT

4.5.1 METHODOLOGY

The TSF surface water management is designed to be a closed dirty water system with a clean water diversion for the small catchment upstream of the TSF. The slope of the natural ground falls to the south-east which allows for the approach to be adopted. Figure 4-7 shows the configuration of the clean and dirty water infrastructure:

- The existing dirty water channel for the dirty catchment upstream of the TSF is indicated by the orange dashed line and directs water to the PCD (this is already constructed and falls outside the scope of this report);
- The clean water catchment (blue shaded area) is diverted around the TSF via a stone pitched clean water diversion channel (blue dashed line) that discharges to culverts (not included in this scope) under the access road and into the natural catchment;
- The TSF dry stack constitutes the dirty catchment (red shaded area) with runoff from the TSF embankments to be contained in lined paddocks and allowed to evaporate;
- A concrete lined solution trench (red dashed line) runs around the perimeter of these paddocks and directs runoff from the outer embankment of the paddocks and drainage from the TSF to lined eastern and western evaporation ponds (purple shaded areas).

The waste classification for the system dictates that a Class C barrier system is required for the containment of Type 3 waste (runoff and drainage in the evaporation ponds and the paddocks) with the arrangement depicted in Figure 4-3.

The sizing of the various infrastructure components was guided by the requirements set out in GN704. The requirements dictate that the clean and dirty water systems be sized such that each system is capable of containing/conveying the resulting flow from a 1 in 50-year storm event without spilling. The channels are thus designed to contain the peak discharge from the runoff emanating from the 1 in 50-year storm event. The paddocks and the evaporation ponds were sized on the results of a water/evaporation balance, with the system stress tested by adding a 50-year storm event, and a 50-year wet year.

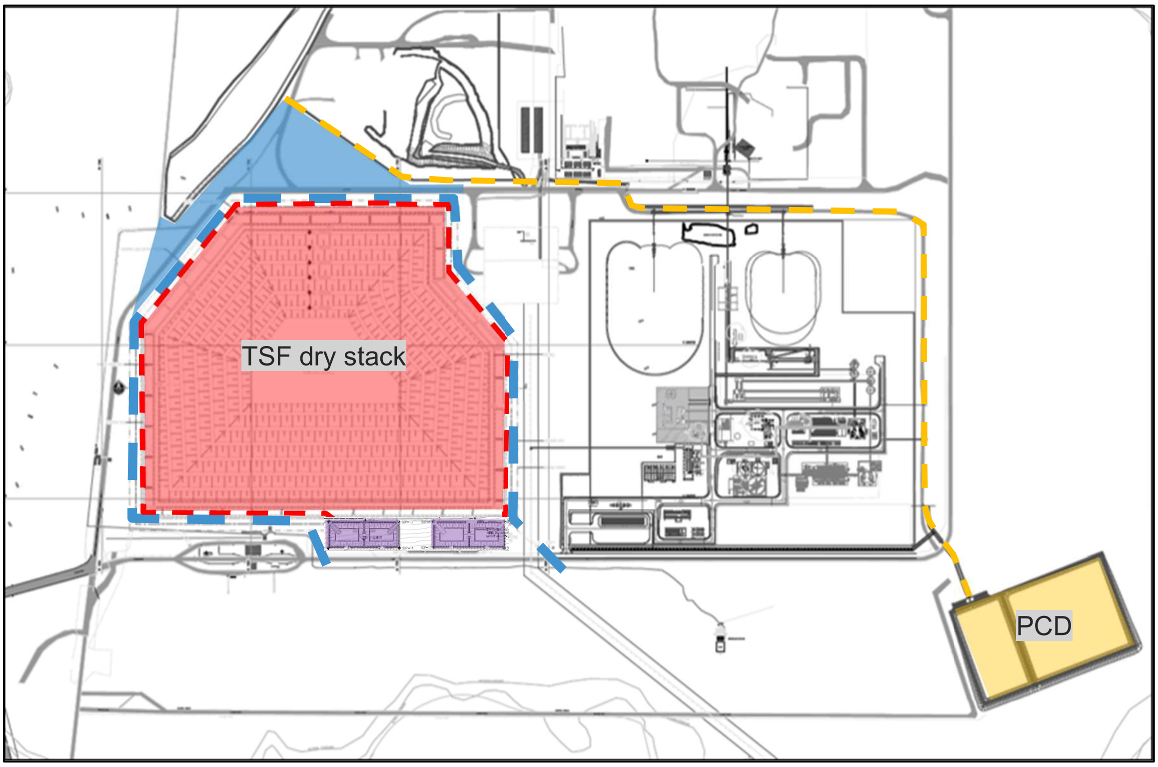

Figure 4-7 Schematic Showing Surface Water Infrastructure

4.5.2 CLIMATIC DATA

Ideally, the determination of climatic data is performed using on site climatic records on a daily time step. In the absence of daily data from site, climatic data was taken from two separate sources. Monthly precipitation data was copied from COENG (2020), which presented monthly average precipitation values taken from the monthly records of Pilanesberg weather station 0548165W (refer Table 4-5). These records also provided the 2% probability of exceedance for each month (refer Table 4-6) for use in the evaporation pond and paddock water balance. Monthly evaporation data was taken from COENG (2020) and presented in Table 4-5.

Storm data, for use in calculating peak discharges and stress testing the water balances, was taken from TR102 and presented in Table 4-7. The data is that of the Pilanesberg Station (0548165W).

Table 4-5: Average Monthly Precipitation and Evaporation (COENG, 2020)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Precipitation (mm)	117	94	89	40	15.7	6.4	4.2	7	14.1	45.3	81	109	623

Evaporation (mm)	175	153	145	114	95	77	85	111	151	183	172	174	1636

Table 4-6: 2% Probability of Exceedance Precipitation (COENG, 2020)

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Dec
Depth (mm)	138	111	108	51	22	12	8	13	20	56	94	126	126

Table 4-7 Storm Depths (mm) (TR102)

Duration	**Return Period (years)**						
	2	5	10	20	50	100	200
1 day	57	78	93	108	129	146	164
2 day	72	98	117	137	164	185	208
3 day	81	110	131	153	183	206	232
7 day	106	145	172	200	237	266	297

4.5.3 PEAK FLOW CALCULATIONS

The peak flows, for sizing the diversion channels and the solution trenches, was calculated using the Rational Method. This is widely used for catchments less than 15 km^2 in size.

4.5.3.1 RATIONAL METHOD

The peak flow (Q) is calculated by the formula:

$$Q = \frac{CIA}{3.6} \tag{5.1}$$

where I is the rainfall intensity (mm/hr), A is the upstream runoff area (km^2), and C is the runoff coefficient (unitless). The intensity is a function of the time of concentration (T_c) and the point precipitation calculated for the T_c. The point precipitation was determined using the Depth -Duration-Frequency relationships for inland precipitation, as presented in HRU Report 2/78 (Midgley and Pitman, 1978).

The determination of the peak flow is dependent on the selection of the design storm. Longer duration storms tend to provide greater storm depths but are associated with lower intensities. Conversely, short duration storms tend to have smaller storm depths but higher intensities. For the design of conveyance structures, in this case channels, the shorter duration storm is chosen. The storm depth for the 50-year recurrence storm is taken from Table 4-7 as 129 mm.

The clean water diversion is separated into water that is diverted around the east and the west of the TSF, while the dirty water catchment is sized for the catchment between the paddock berms and the solution trench. The results of the peak flow calculation are shown in Table 4-8.

Table 4-8: Peak Flows Calculated for the 50-Year Recurrence Storm

Catchment	Area (km^2)	Time of Concentration (hrs)	Intensity (mm/h)	Runoff Coefficient	Peak Flow (m^3/s)
Clean water west	0.061	0.63	101.64	0.395	0.68
Clean water east	0.029	0.43	126.24	0.335	0.34
Dirty water	0.046	0.54	112.56	0.332	0.47

The TSF itself has a much larger dirty area (21 ha) than that presented in Table 4-8, but as the runoff from the embankments is contained in the paddocks, the area for the TSF is not used to size the dirty water solution trench. The dirty area for the TSF is addressed in Section 4.5.5.

4.5.4 CHANNEL SIZING

According to Government Notice No. 704 (GN704) of 1999, effort must be made to isolate the dirty areas to prevent the runoff from the “clean” areas from entering the dirty areas, and the dirty areas from entering the clean areas. This will be achieved by constructing concrete lined dirty water solution trenches around the perimeter of the paddocks, and constructing a stone pitched clean water diversion around the solution trench.

The peaks flow calculated for the clean water diversion was calculated separately for flow that will be diverted around the east and west of the TSF. For simplicity of construction, a single channel size will be specified for both channels. The clean water channels will discharge through existing culverts under the existing access road and then into the natural drainage system. The dirty water solution trench will discharge directly into the evaporation ponds located at the south-east corner of the TSF.

The sizing of the channels is calculated using Manning’s equation:

$$Q = \frac{A}{n} R^{2/3} \sqrt{S} \tag{5.2}$$

where Q is the flow rate (m^3/s), A is the flow area (m^2), n is the Manning’s roughness coefficient, R is the hydraulic radius (m), and S is the channel slope (m/m). The flow area and hydraulic radius are a function of the flow depth, the flow rate, and the channel dimensions. The calculation is therefore an iterative process, with the channel dimensions adjusted and then the flow depth solved such that the flow rate is equal to the design flow rate. The channels will be constructed in sections to maintain a 1:200 (north and south) gradient on the flatter areas and 1:100 (west and east) gradient on the steeper perimeter slopes of the catchment. Table 4-9 summarizes the sizing of the channels, while Table 4-10 summarises the key hydraulics parameters. The flow in the clean water diversion channels is subcritical, while the flow in the solution trench is supercritical. The Froude number for the flow on the milder slope (1:200) of the solution trench is 1.02 which means that the flow will be unstable (regular surface distortions). The flow depth for this flow is only 0.20 m which allows for sufficient freeboard to contain the flow despite the flow distortions.

Table 4-9: Channel Sizing

	Clean water diversion	Dirty water channels
Bottom Width (m)	1	1
Lining Type	Stone pitching	Concrete
Manning's coefficient	0.036	0.016
Side Slopes (V:H)	1:2	1:1.5
Minimum channel depth	0.7	0.5

Table 4-10: Hydraulic Properties of Channels

	Clean water diversion	Clean water diversion	Dirty water channels	Dirty water channels
Channel Slope (V:H)	1:100	1:200	1:100	1:200
Flow Rate (m^3/s)	0.68	0.68	0.47	0.47
Flow Depth (m)	0.37	0.44	0.20	0.25
Flow Velocity (m/s)	1.07	0.83	1.78	1.40
Froude Number	0.68	0.49	1.41	1.02
Flow Regime	Sub critical	Sub critical	Super critical	Super critical

4.5.5 TOE PADDOCKS AND EVAPORATION PONDS

The toe paddocks and eastern and western evaporation ponds have been designed to receive dirty water as inflow, with evaporation and abstraction for dust suppression (paddocks only) being the outflows. A monthly water balance was developed to assess the performance of the structures. Figure 4-8, Figure 4-9, and Figure 4-10 show the typical cross section of the paddocks, the plan view of the evaporation ponds, and the typical cross section of the evaporation ponds, respectively. The evaporation ponds consist of two ponds (east and west), each containing two compartments. The primary compartments are sized such that they can receive flow from the solution trenches and will fill up and evaporate based on the seasonal precipitation and evaporation rates, while the secondary compartments provide storage capacity for containing spillage from the primary compartments.

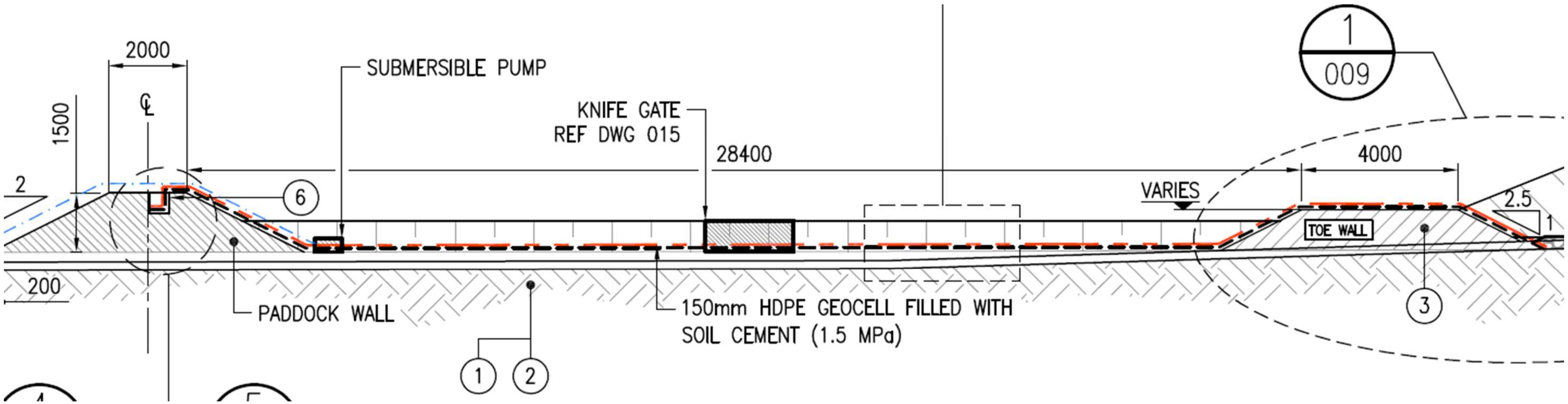

Figure 4-8: Typical detail of TSF Paddocks and Clean and Dirty Trenches

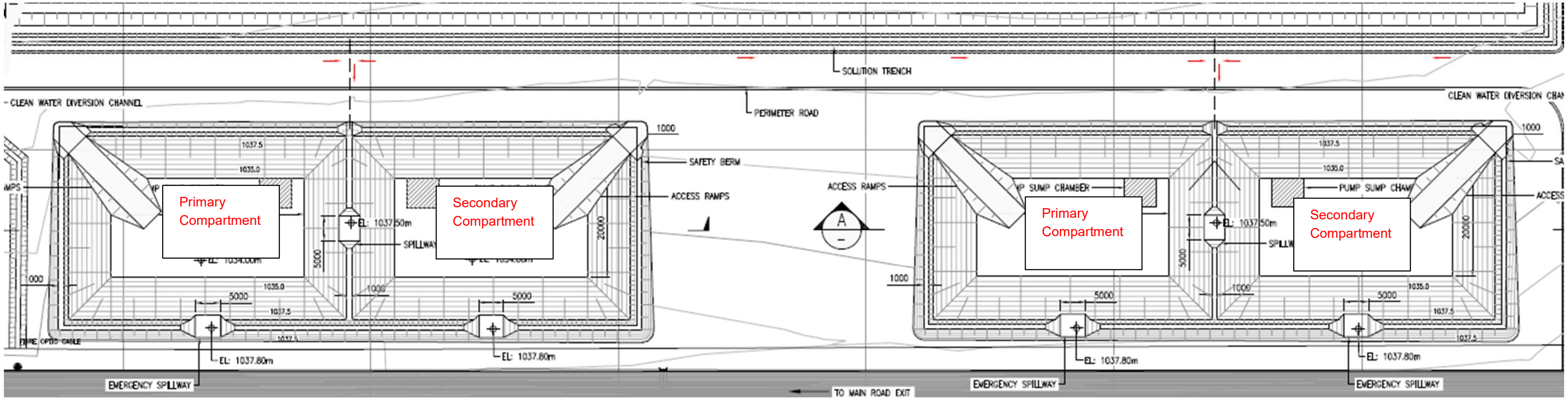

Figure 4-9: Evaporation Pond Layout

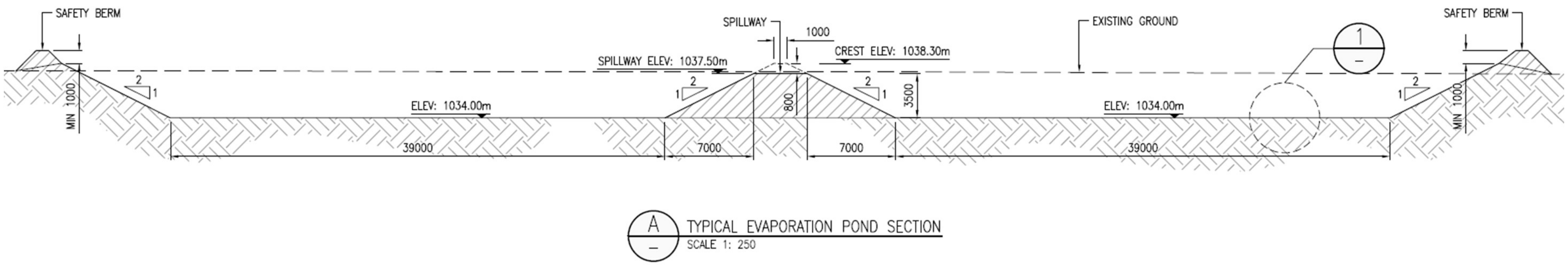

Figure 4-10: Typical Cross Section for Evaporation Pond Compartments

4.5.5.1 TOE PADDOCKS

The paddocks have been designed to accommodate the runoff generated from the TSF embankments at the full TSF height, which accounts for the maximum possible runoff from the embankments. The permeability of the TSF material is 1 x 10^{-6} m/ and the side slopes are 1V:2.5H, and as such, a maximum runoff coefficient of 1.0 was assumed on the embankments. The paddock walls are 1.5 m high, 28.4 m from the crest of the TSF toe berm to the crest of the paddock berm, and approximately 50.0 m wide (refer Figure 4-8 for paddock cross section). The size of the paddocks was determined through a monthly water balance that tested the system under three conditions:

1) Average climatic conditions
2) Containment for a 1 in 50-year wet season applied to the 2^{nd} year after fully developed TSF
3) Containment of the 1 in 50-year 7-day storm occurring in the wet season of the 2^{nd} year after fully developed TSF

Because the runoff coefficient used in determining the embankment runoff is 1.0, and the catchment area of the embankment is four time the area of the paddocks, evaporation alone will not be enough to ensure that the paddocks do not spill for even the average climatic conditions. As such, water in the paddocks needs to be abstracted ensuring that the paddocks are able to maintain a suitable freeboard. The water abstracted from the paddocks will be used for dust suppression, with a daily abstraction target that varies with the season (refer Figure 4-11). The target volume increases before the wet season to ensure that there is suitable capacity for storm events.

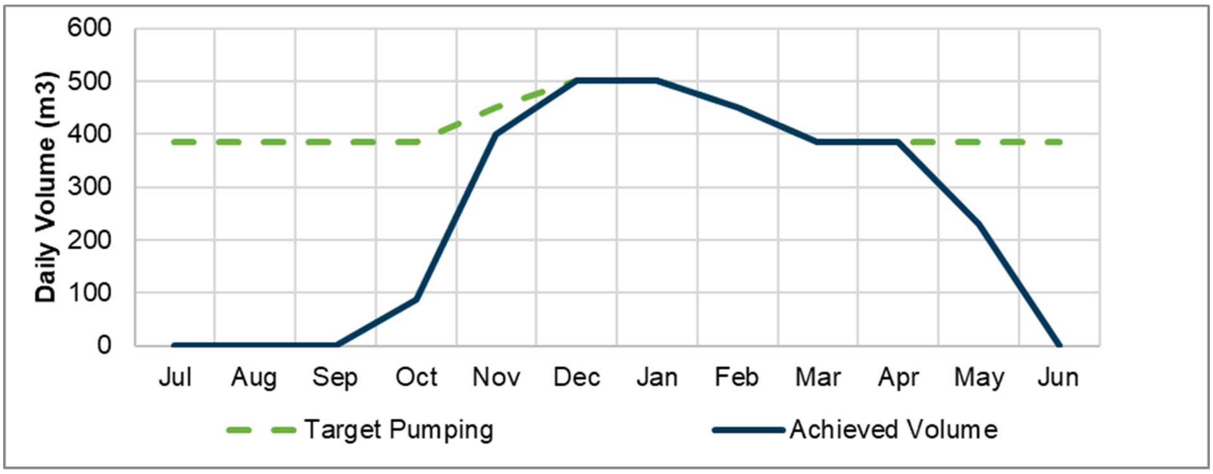

Figure 4-11: Pumping targets for dust suppression

During average climatic conditions, the maximum water level in the paddocks is expected to be 0.36 m. The occurrence of a 1 in 50-year storm event (237 mm over 7 days) will result in approximately 59 600 m^3 of runoff being contained in the paddocks, which have a combined capacity of 65 500 m^3. After accounting for pumping and evaporation, the peak water level in the paddock will be 0.88 m, which will be lowered below the freeboard depth within 2 months. The paddock system was also modelled against a 1 in 50-year wet season, with the maximum water level in the paddocks rising to 0.70 m. Figure 4-12 shows the seasonal behaviour of the water depth in a paddock, with the wet season and storm event conditions shown for comparison.

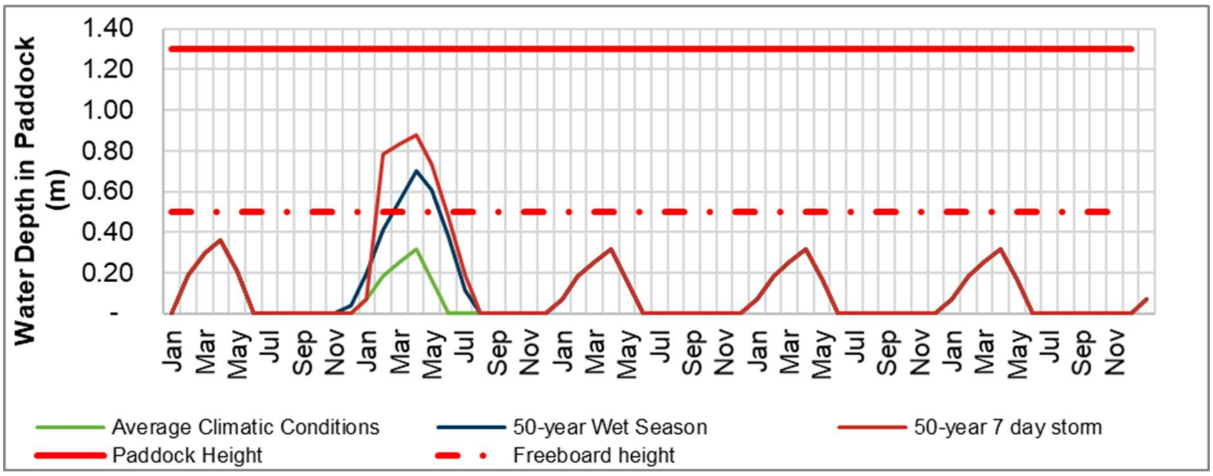

Figure 4-12: Paddock Seasonal Depth Comparison for Average Conditions vs After Storm Events

It is clear that the paddocks operate with a suitable freeboard under normal conditions, and the sizing of the paddocks is such that there should be no spilling over the paddock embankment for precipitation events that are less than or equal to the 1 in 50-year wet season or 1 in 50-year storm events.

4.5.5.2 EVAPORATION POND

The system was tested for the same conditions as set out for the evaporation paddocks. Each compartment will have a base capacity of 4 400 m^3 with a depth of 3.5 m from basin invert to the spillway invert between compartments (refer Figure 4-10), and a depth of 4.3 m from basin invert to the overflow spillway (pond capacity).

Figure 4-13 shows the comparison of the west evaporation pond depths for the average climatic conditions and the 1 in 50-year recurrence wet season and storm events.

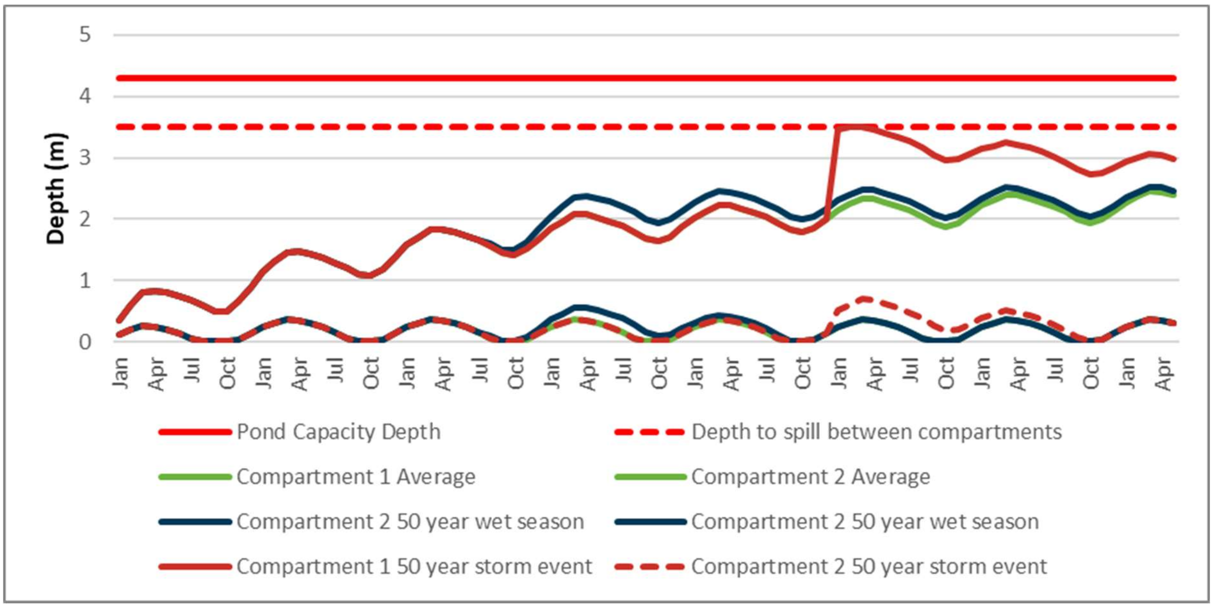

Figure 4-13 Western Pond Seasonal Depth Comparison for Average Conditions vs After Storm Events

Under average climatic conditions, the western evaporation pond takes six seasons to reach an equilibrium where the water depth in the primary compartment fluctuates between 1.88 m and 2.40 m. The 1 in 50-year wet season does not cause the compartment to spill to the secondary compartment, but the occurrence of the 1 in 50-year 7-day storm does. Figure 4-14 shows the water depth compared to the spilling event for the system with a 7-day 1 in 50-year recurrence storm event.

Under average climatic conditions, the secondary compartment will contain water in the wet months and will empty in the dry season. It will take several seasons after the occurrence of the 1 in 50-year recurrence storm, for the secondary compartment to empty.

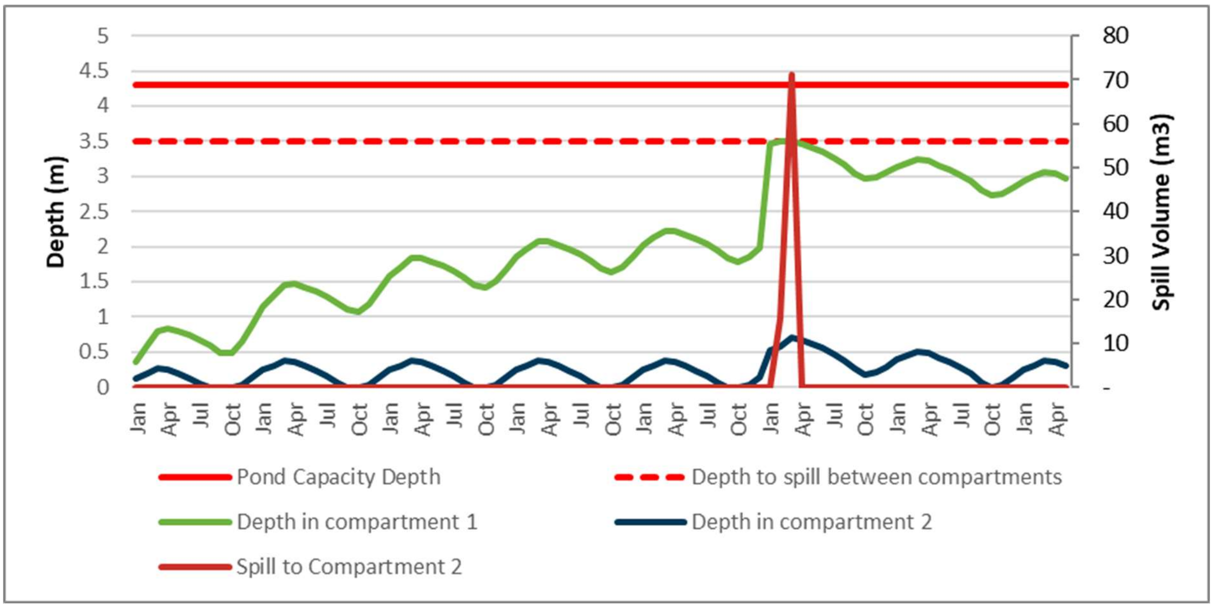

Figure 4-14 Eastern Pond vs Spill Volume for 1 in 50-Year Storm Added

Figure 4-15 shows the comparison of the east evaporation pond depths for the average climatic conditions and the 1 in 50-year recurrence wet season and storm events. Because of the natural contours around the TSF, the eastern evaporation pond receives runoff from a greater catchment than that of the western evaporation pond. The result is that the primary compartment of the eastern evaporation pond will reach an equilibrium where the pond spills every wet season.

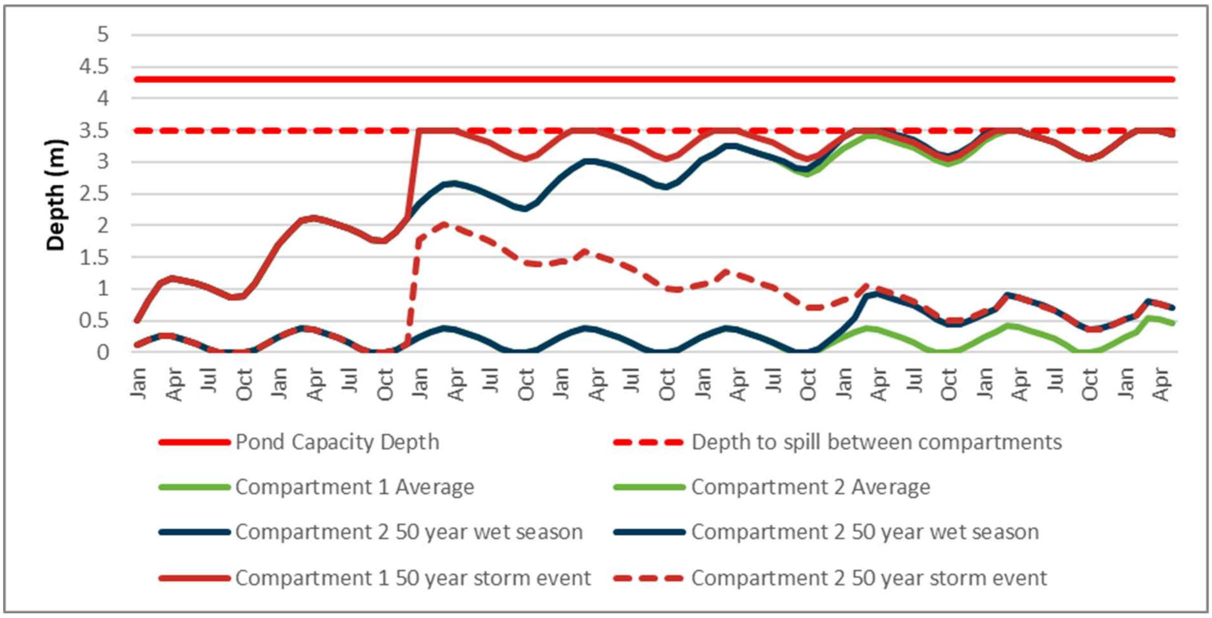

Figure 4-15 Eastern Pond Seasonal Depth Comparison for Average Conditions vs After Storm Events

Figure 4-16 shows that under average climatic conditions, the system will take 6 seasons to start spilling to the secondary compartment, with the system reaching equilibrium after the seventh season. Figure

4-17 shows that if the 1 in 50-year wet season were to occur in the fifth season, the primary compartment would spill to the secondary compartment, with the volume of spillage increasing by 70 m^3 compared to the spillage under average climatic conditions.

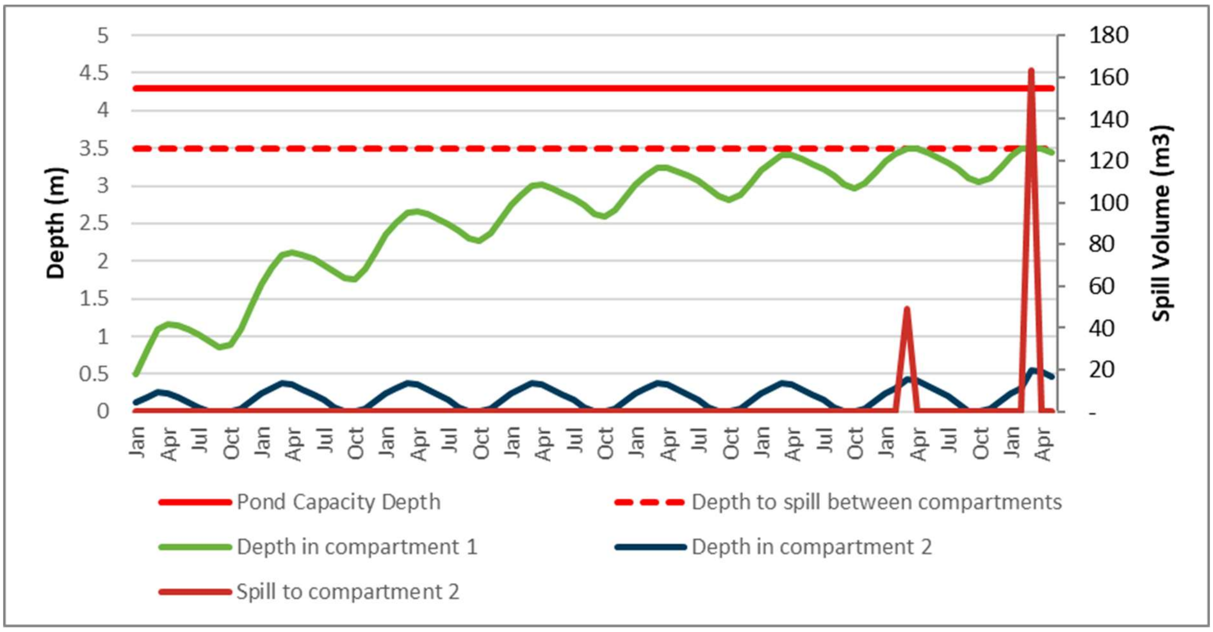

Figure 4-16 Eastern Pond Depth vs Spill Volume for Average Precipitation

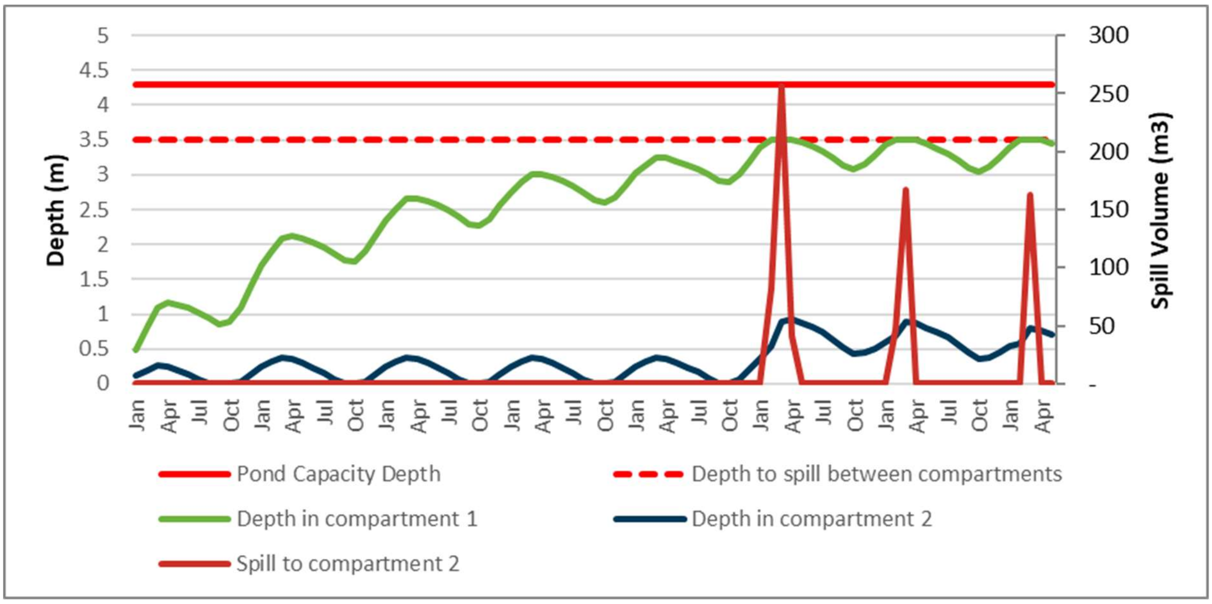

Figure 4-17 Eastern Pond vs Spill Volume for 1 in 50-Year Wet Season

Figure 4-18 shows that adding the 7-day 1 in 50-year storm to the system results in the spill volume increasing to greater than 1 400 m^3. It would take several seasons for the secondary compartment to return to average season operating water depths.

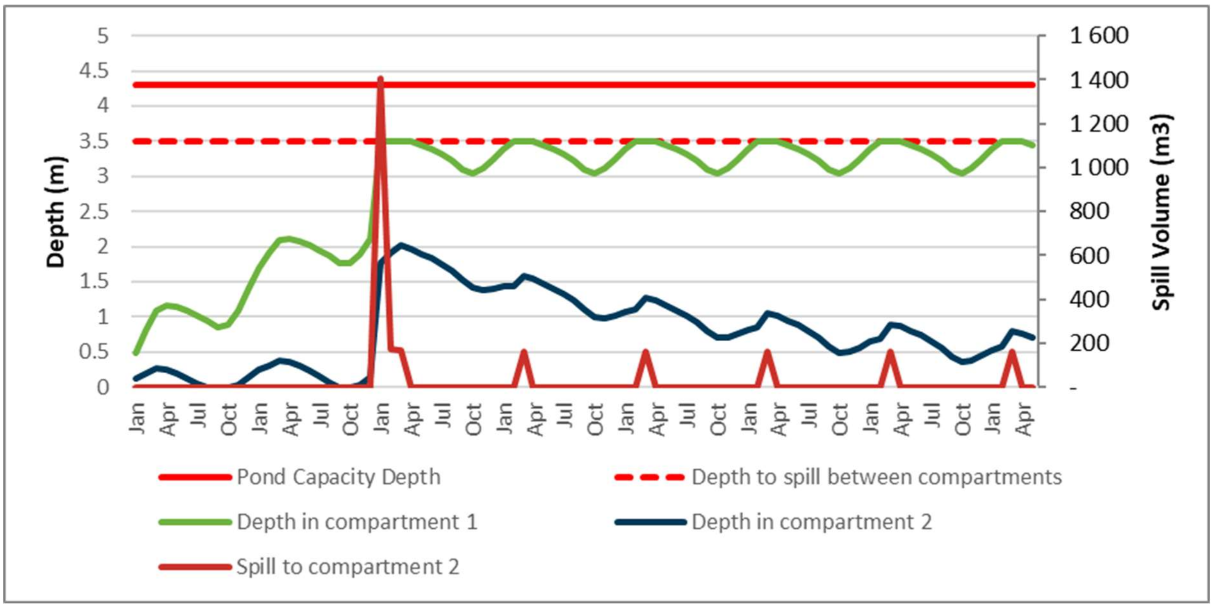

Figure 4-18 Eastern Pond vs Spill Volume for 1 in 50-Year Storm Added

4.6 DRAINAGE COLLECTION LAYER

A drainage collection layer will be placed on top of the geomembrane on the barrier system as finger drains at a 20 m distance centre to centre. The drainage layer will comprise 160 mm diameter slotted pipe surrounded by 13 mm washed stone and covered by a nonwoven geotextile. The finger drain will have a trapezoidal shape a top width of 1 m, an overall height of 300 mm and side slopes of 1 (vertical) in 3 (horizontal) as per Figure 4-19 . The TSF base will have a slope of 1.5% from the higher side of the TSF to the lower side and the drainage pipes will have a minimum longitudinal slope of not less than 1%.

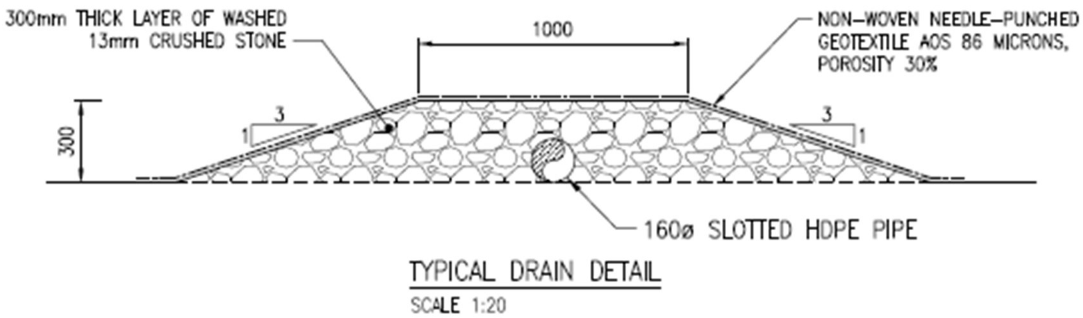

Figure 4-19: Drainage Collection System

The Geotextile Filter Design Guide (Luettich, Giroud and Bachus, 1992) was used for selection of the geotextile. The geotextile will serve as filter to retain tailings whilst allowing water to flow through. The voids in the 6mm stone are considered large such the permeability criteria have been favoured whilst the boundary condition for flow is steady-state. Table 4-11 summarises the properties of the recommended geotextile.

Table 4-11: Geotextile Properties

Property	Value	Test Method
Manufacturing	Non woven	-
Apparent Opening size, O_{95}	Less than 86µm	SANS 12956
Minimum Hydraulic Conductivity	$2 * 10^{-7}$ m/s	SANS 11058
Porosity	≥30%	
CBR	≥3 600 N	SANS 12236

All pipes shall be manufacture 160m diameter PN10, providing long-term strength and creep resistance, stress crack resistance and rapid crack propagation resistance. the pipe shall be manufactured from twin wall Class 34 according to SANS 1601 which under the pressure of the tailings at full height, the FoS against buckling is higher than 1.8, which lifetime is deemed to exceed the lifetime of the facility.

4.7 TAILINGS DELIVERY

The tailings will be delivered to the TSF via a conveyor system from the mine as shown on Drawing No. 301-00509/10-008. The design of the conveyor system and spreader will be conducted during the detailed design phase of the project by others.

4.8 ROADS

There will be a 7 meter wide perimeter road between the clean water diversion channel and the solution trench as shown on drawing 301-509/10-005. The roads may act as a fire break as well in the event of veld fires. The typical road section is presented in Figure 4-20.

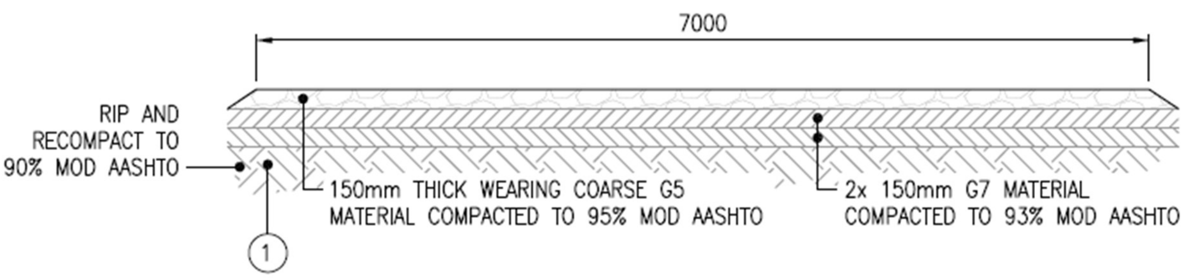

Figure 4-20 Typical Perimeter Access Road Detail

The layers of the road are as follows:

- Topsoil and vegetation is stripped to a depth of 250 mm
- The base of the excavation is to be ripped and recompacted to a depth of 200 mm at 90% MOD AASHTO.
- A minimum depth of 300 mm G7 material to be placed in layers not exceeding 150 mm and compacted to 93% MOD AASHTO
- A 150 mm thick layer of G5 wearing course, compacted to 95% MOD AASHTO

4.9 SILT TRAP

A silt trap will not be required for the evaporation ponds, the water that will report to the pond will flow through the filter drains of the TSF and the silt will be filtered out. An access ramp into the basin is included if desilting the basin is required. Dirty stormwater from the side slopes will be stored in the paddocks around the TSF. These paddocks will be cleaned of silt when required.

4.10 CLOSURE CONSIDERATION

The TSF will be constructed in 7 m lifts (7 lifts in total to its final height). It is assumed for rehabilitation that while a lift is being constructed, rehabilitation will be taking place on the shift below. The reasons for the rehabilitation are as follows:

- Slope stability – if excessive infiltration is allowed, this could lead to a build-up of water in the TSF and the additional pore pressure would decrease the shear strength of the material reducing the factor of safety.
- Erosion control – the vegetation anchors the topsoil layer and reduces the flow velocity of the run-off which reduces erosion taking place.
- Leachate control – the topsoil layer reduces the amount of infiltration into the TSF thus reducing the amount of water contaminated.

The proposed rehab will include covering the surface of the facility with a 300 mm layer of material excavated from the basin of the TSF and reshaping the plateau area with a slope of 3% to reduce infiltration. The material will be fertilized and grassed.

4.11 STABILITY AND SEEPAGE ANALYSIS

Slope stability and seepage analyses are modelling tools used to evaluate designs of man-made embankments or assess the safety of a natural slope (Stability Modelling with SLOPE/W, 2013). These analyses are used to determine the possible failure mechanisms, the likelihood of failure occurring and the design of the optimal slope.

The stability analysis was completed using Slide2 (Version 9.011) limit equilibrium software from RocScience. The location of the phreatic surface used in the stability analysis was obtained from an integrated seepage analysis completed within the groundwater module of Slide2 (steady state finite element analysis).

The analysis was completed using auto-refine circular search with the Morgenstern and Price and Spencer methods. The analysis was also completed with the inclusion of a weak layer to simulate the HDPE liner, this allows the failure surface to form along the weak surface. The Morgenstern and Price and Spencer methods were selected because both are complete equilibrium procedures of slices, with Spencer being the simplest of the procedures to satisfy all conditions of equilibrium and Morgenstern and Price being the most flexible of the procedures that satisfy all conditions of equilibrium (Duncan et al, 2014). According to Duncan et al, the procedures that satisfy complete static equilibrium are the most accurate and preferred in general.

The Peak Ground Acceleration (PGA) value for the Bakubung TSF footprint was estimated at 0.14g (Kijko, 2003). This peak ground acceleration represents a 10% probability being exceeded in a 50-year period (Esterhuyse et al. 2014).

The seismic coefficient is a lateral force coefficient used in a pseudo static analysis to determine the effect of seismic loading on slopes using a limit equilibrium analysis. Hynes-Griffin and Franklin (1984) proposed a factor of 0.5 be applied to the PGA to get the seismic coefficient for use in stability analysis. However, a conservative approach was taken, and the PGA value was reduced by a factor of 0.7 which results in a horizontal seismic coefficient of 0.10. This is considered the worst-case horizontal seismic loading.

4.12 MODELLING DETAILS

The stability of a section on the south side of the facility was investigated. This section was selected as representative of the worst case for stability because the embankment will be highest in this area and test pits in this area indicate the deepest horizon of residual fine grained norite. Figure 4-21 illustrates the slide model used to analyse the section.

These material design properties were sourced from characteristic properties of platinum tailings and neighbouring mines in the same geological region and will need to be refined upon completion of the geotechnical investigation report.

The slope stability and seepage analysis were carried out simultaneously. Hydraulic conductivity properties were assigned to the seepage model and material strength properties were assigned to the slope stability model to determine the FoS. A summary of the material properties for the seepage and slope stability analysis are presented in Table 4-12.

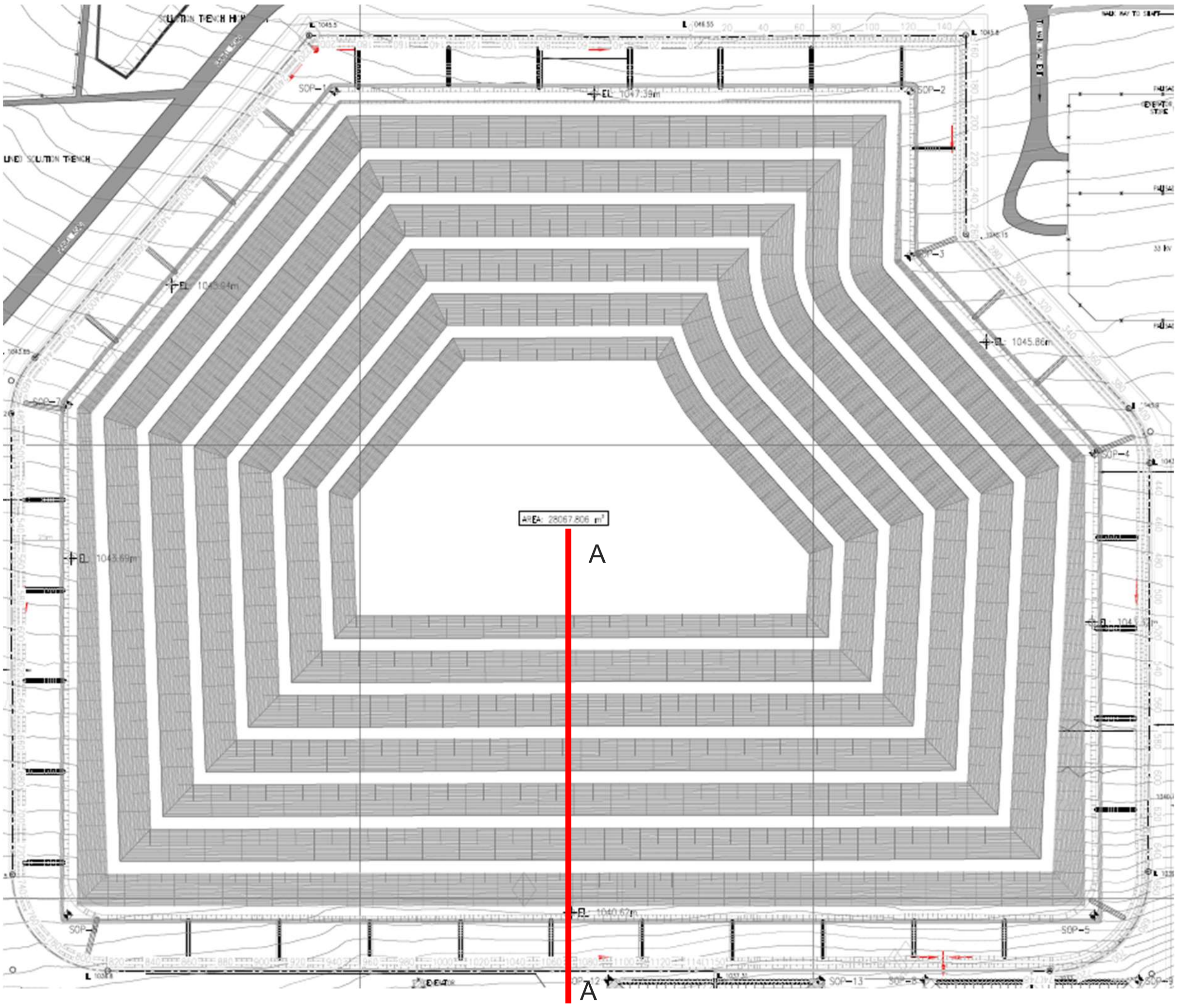

Figure 4-21: Slope Stability Analysis Sections

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Phib (deg)	Air Entry (kPa)
Tailings		19	Mohr-Coulomb	0	32	0	0
Alluvium		18	Mohr-Coulomb	0	22	0	0
Residual norite fine grained		15	Mohr-Coulomb	10	15	0	0
Residual norite coarse grained		18	Mohr-Coulomb	0	32	0	0
Rockfill toe		20	Mohr-Coulomb	0	42	0	0
HDPE liner		14	Mohr-Coulomb	0	18	0	0

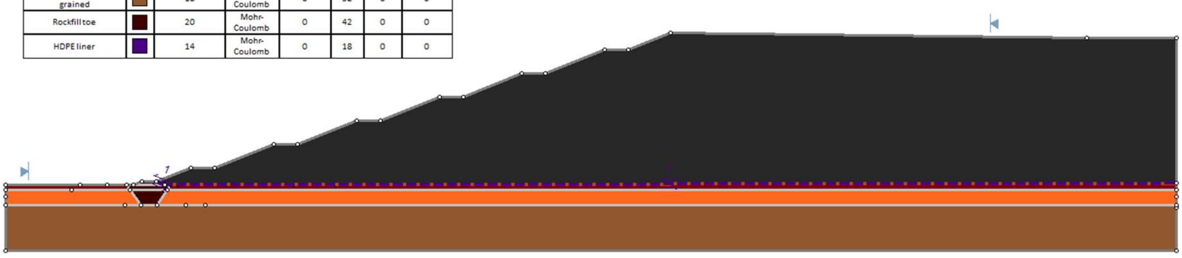

Figure 4-22: Typical Section Model

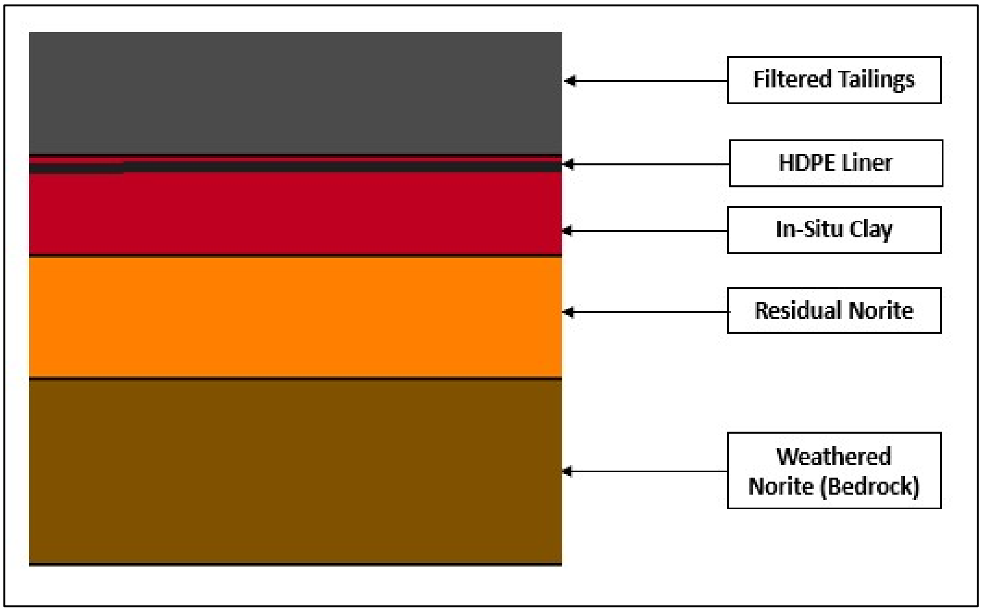

Figure 4-23: Typical Foundation Details

Table 4-12: Material Design Properties

Material	Unit Weight (kN/m^3)	Cohesion (kPa)	Friction Angle (°)	Permeability (m/s)	Reference
Tailings	19	0	32	1×10^{-6}	Appendix A[1]
Alluvium	18	0	22	2×10^{-7}	Appendix A
Residual Norite fine grained	15	10	15	1×10^{-7}	Appendix A
Residual Norite coarse grained	18	0	32	1×10^{-6}	Appendix A
Rockfill toe	20	0	42	1×10^{-5}	Appendix A[1]
HDPE Geomembrane	14	0	18	5×10^{-10}	Appendix C

NOTES:

(1) friction angle and cohesion has been assumed from previous projects in the area by KP

4.13 SLOPE STABILITY RESULTS

Stability analyses were carried out with the dam at maximum elevation (1 088 mamsl). As the TSF is designed for filtered tailings, the build-up of a phreatic surface is highly unlikely as it will require a drastic change in the plant operations upstream. A pseudo static analysis of the section and a post construction scenario where the fine grained residual norite could still be undrained was also completed.

A global failure was perceived as a failure which would cause potential damage to the equipment on top of the tailings dump. Whereas a local failure is seen as a less critical failure. A minimum FoS of 1.5 is required for static conditions and 1.1 is required for pseudo static conditions (ANCOLD, 2012). The results presented indicate the lowest factors of safety achieved. Table 4-13 presents a summary of the results. Figure 4-24 presents the analysis images for static conditions, Figure 4-25 presents the analysis images for pseudo static conditions and Figure 4-26 presents the analysis with a modelled phreatic surface.

Table 4-13: Stability Analysis Results

No	Scenario Description	FOS
1	Section A – Static conditions – drained	1.5
2	Section A - Pseudo static conditions – drained	1.1
3	Section A –Undrained fine grained residual norite	1.3

The results of the stability analysis indicate that the proposed design meets the stability requirements for static long-term and pseudo static conditions if there is not a raised phreatic surface. The post construction scenario where undrained fine grained tailings was modelled achieved a FoS greater than 1.3, which is acceptable as a short term condition.

A further simulation was carried to simulate a possible blockage of the drainage collection layer and the phreatic surface was varied aiming to drop the FoS below 1.0. The phreatic surface leading to a failure of the TSF is deemed to be unrealistic, given the filtered tailings and standard operating procedures of a tailing's facility (ie. a pond will be required to be present on top).

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Tailings		19	Mohr-Coulomb	0	32
Alluvium		18	Mohr-Coulomb	0	22
Residual norite fine grained		15	Mohr-Coulomb	10	15
Residual norite coarse grained		18	Mohr-Coulomb	0	32
Rockfill toe		20	Mohr-Coulomb	0	42
HDPE liner		14	Mohr-Coulomb	0	18

Method Name	Min FS
Spencer	1.510
GLE / Morgenstern-Price	1.503

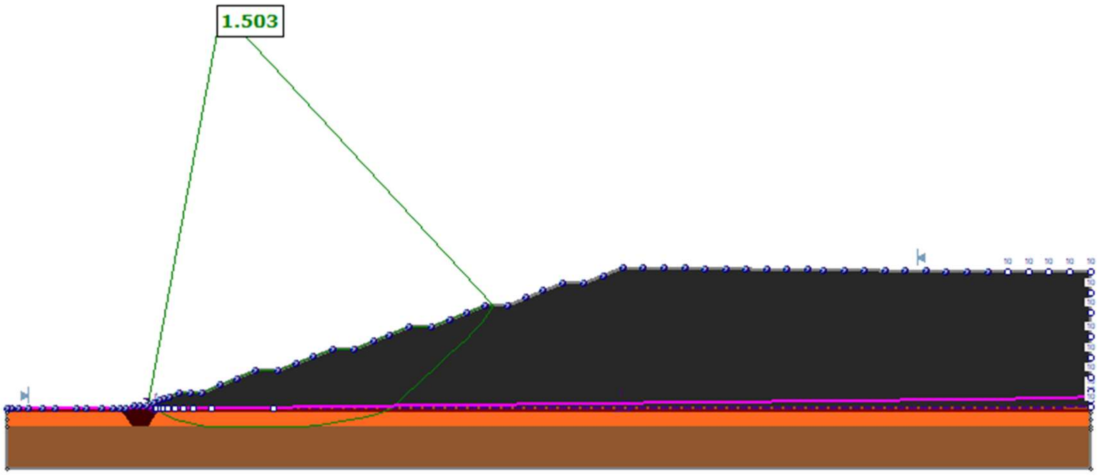

Figure 4-24: Section A Static Conditions

Method Name	Min FS
Spencer	1.076
GLE / Morgenstern-Price	1.067

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)
Tailings		19	Mohr-Coulomb	0	32
Alluvium		18	Mohr-Coulomb	0	22
Residual norite fine grained		15	Mohr-Coulomb	10	15
Residual norite coarse grained		18	Mohr-Coulomb	0	32
Rockfill toe		20	Mohr-Coulomb	0	42
HDPE liner		14	Mohr-Coulomb	0	18

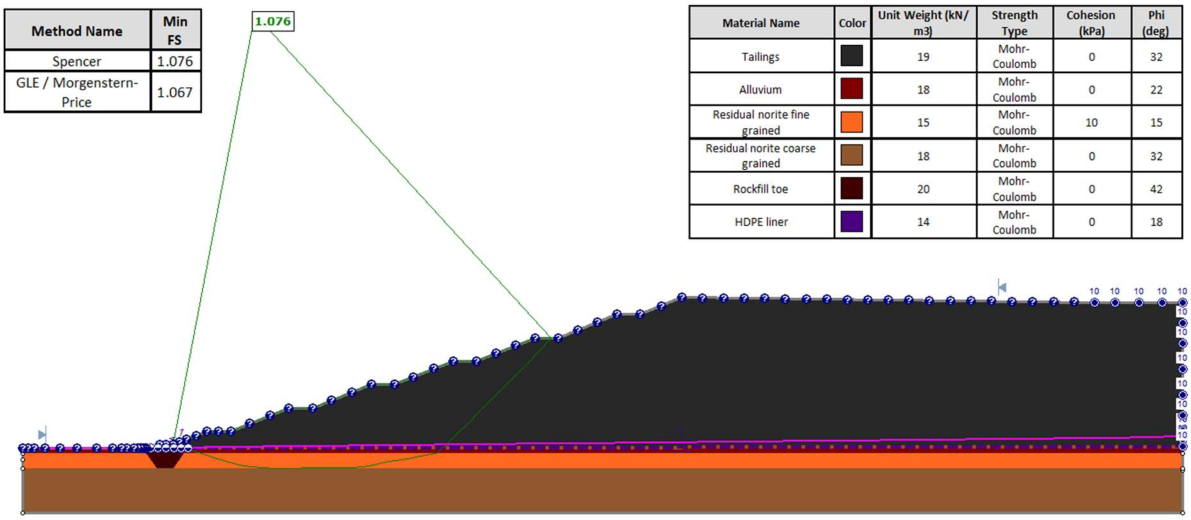

Figure 4-25: Section A Pseudo Static Conditions

Material Name	Color	Unit Weight (kN/m3)	Strength Type	Cohesion (kPa)	Phi (deg)	Vertical Strength Ratio	Minimum Shear Strength (kPa)
Tailings		19	Mohr-Coulomb	0	32		
Alluvium		18	Mohr-Coulomb	0	22		
Residual norite coarse grained		18	Mohr-Coulomb	0	32		
Rockfill toe		20	Mohr-Coulomb	0	42		
HDPE liner		14	Mohr-Coulomb	0	18		
Residual norite fine grained undrained		15	Vertical Stress Ratio			0.25	0

Method Name	Min FS
Spencer	1.340
GLE / Morgenstern-Price	1.325

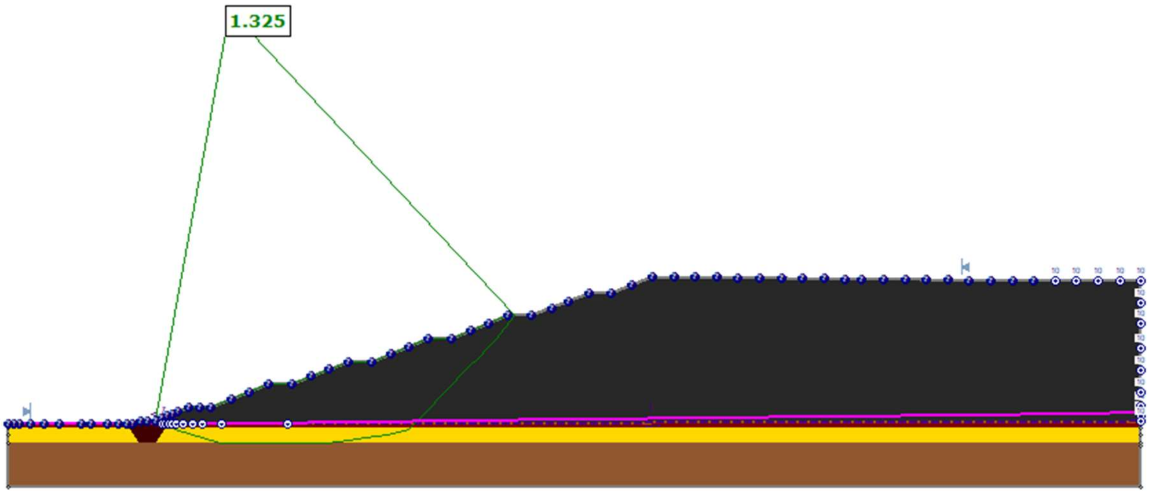

Figure 4-26: Section A Undrained Fine Grained Residual Norite

CONSTRUCTION QUALITY ASSURANCE

The main risk for underperforming of a barrier system is due to mechanical and physical damage of the barrier system during the installation. It is therefore paramount for a reputable contractor to supply and install the liner with proven track record in similar work compounded with a construction quality assurance programme. The CQA is a detailed programme for checking all part of the design, particularly the barrier system, such as technical specifications, test methods and frequency and validation requirements. The CQA would include

- General information
- Definitions
- Responsibilities of parties
- Manufacturer's Quality Control
- Specifications
- Conformance Testing
- Defects and repairs
- Reporting and
- Drawings

A CQA Programme for the earthworks and barrier installation is reported in **Appendix F** and it will be further developed to suit construction specific requirements.

5.0 DRAWINGS

List of Project Drawings

Drawing Number	Revision	Title
301-00509/10-000	E	List of Drawings
301-00509/10-001	D	Existing Conditions - Layout Plan
301-00509/10-002	E	TSF Final Landform - Layout Plan
301-00509/10-003	D	Geotechnical Test Pits - Layout Plan
301-00509/10-004	C	Drainage - Layout Plan
301-00509/10-005	D	Toe Wall - Typical Sections and Details
301-00509/10-006	C	Solution Trench - Layout Plan
301-00509/10-007	C	Solution Trench - Longitudinal Sections
301-00509/10-008	DC	Conveyor - Layout and Typical Sections
301-00509/10-009	DC	Seepage Interception Filter Drain - Typical Sections and Details
301-00509/10-010	B	Evaporation Ponds - Layout, Typical Sections and Details
301-00509/10-011	BA	Underdrainage layer – Layout Plan
301-00509/10-015	A	Paddock wall typical details

The drawings above are presented in **Appendix G**.

6.0 SCHEDULE OF QUANTITIES

A schedule of quantities and a cost estimate was prepared for the construction of the TSF. A summary of the cost estimate is presented in Table 6-1.

Table 6-1 Cost Estimate for the Construction of the TSF

	SUMMARY	
Section 1	PRELIMINARY & GENERAL	R 27 560 058.49
Section 2	BULK EARTHWORKS	R 20 131 962.20
Section 3	TSF LINER	R 59 271 344.35
Section 4	EVAPORATION DAM	R 9 188 426.34
Section 5	CANALS	R 2 586 946.48
Section 6	PERIMETER ROAD	R 688 182.26
Section 7	**CONTENGENCY (10%)**	R11 942 692.01
Section 8	**ENGINEERING (3.5%)**	R4 179 942.20
	Total (Excl. VAT)	R 135 549 554.33
	VAT (15%)	R20 332 433.15
	TOTAL	R155 881 987.48

Assumptions:

- The Preliminary & General is assumed to be 30% of the construction and material cost for the TSF
- A contingency of 10% has been included for
- The engineering cost during the project is taken as 3.5% of the total Capex of the project

The complete schedule of quantities is included in **Appendix H**.

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8.0 CERTIFICATION

This report was prepared and reviewed by the undersigned.

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Approval that this document adheres to Knight Piésold Quality Systems: