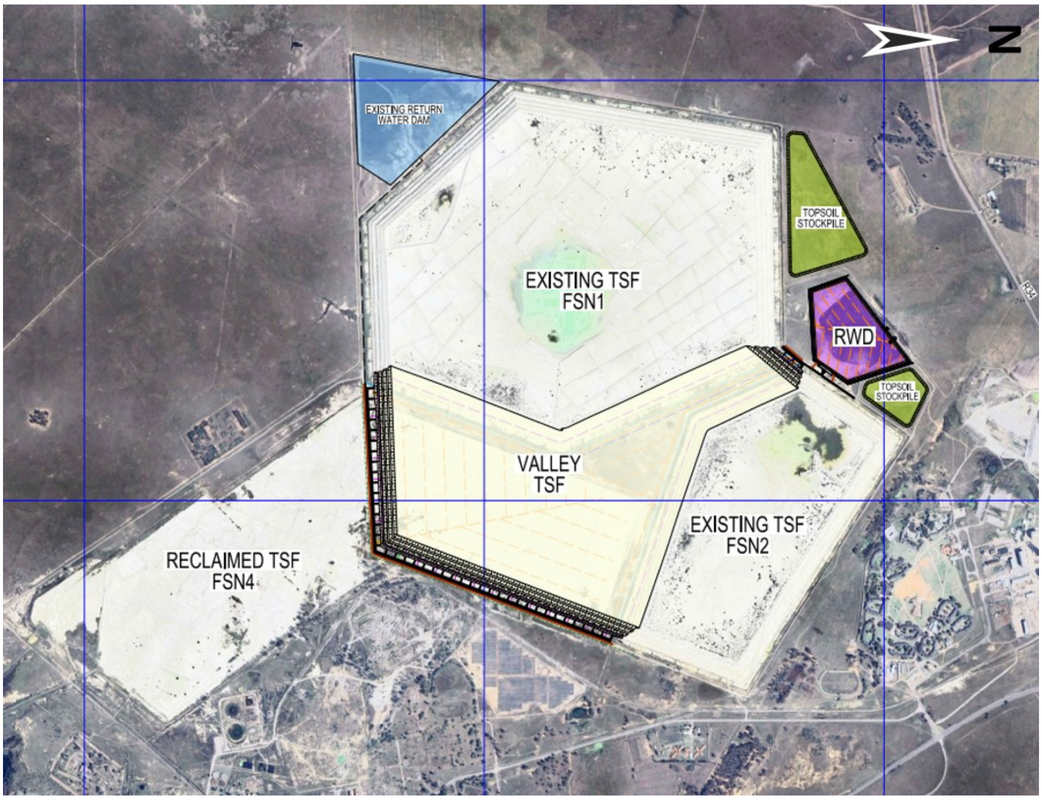
HARMONY VALLEY TAILINGS STORAGE FACILITY

DESIGN REPORT



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Cnr Main Reef Road and Ward
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1759

2210513/R03RA February 2024

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Harmony Valley TSF Design Report

Report Reference Number: 2210513/R03RA

Revision date: February 2024

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Data provided to Geotheta

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Data determined by Geotheta

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Executive Summary

Geotheta was appointed by Harmony Gold to complete the design of the proposed new Valley Tailings Storage Facility (TSF) in Welkom, South Africa.

Key Parameters of the Valley TSF design are:

- Maximum final height: 36m
- Footprint area: 163.5 Ha
- Total capacity: 56.8 million tons
- Deposition period at 600 000 tons per month: 8 years
- Maximum rate of rise (Basin): 4.12m/year
- Maximum rate of rise (Embankment): 3.99m/year
- Deposition method: Cyclone

The Valley TSF provides a storage capacity of 56.8 million tons over a deposition period of 8.0 years at the target deposition rate of 600 000tpm with a maximum rate of rise of 4.12m/year (basin) and 3.99m/year (embankment). This rate of rise will be achieved by cyclone deposition.

Valley TSF will be developed with an intermediate outer slope of 1V:3H between benches. The overall slope with benches is 1V:4H. The inter-bench height is 8.0m and the benches are 8.0m wide.

The maximum toe wall embankment height is 3m with a 3m wide crest, outer slope of 1V:1.5H and 1V:2H inner slope. The toe wall embankment will be constructed in 150mm layers to 95% Proctor density at 0% to +2% Optimum Moisture Content (OMC). The toe wall material will be obtained from the basin of the facility.

The cyclone walls will be constructed 50m away from the toe wall on the northwest, eastern and southern flanks of the Valley TSF. The other flanks butt up against the dormant FSN1 and FSN2 facilities and no cyclone deposition will occur from these flanks. Spigotting or open-end deposition will be done for pool control only when required.

These cyclone walls will provide an elevated platform to allow for overflow tailings deposition. The cyclone wall is 3m high with a 3m wide crest, outer slope of 1V:2H and 1V:2H inner slope.

According to GISTM, the Valley TSF has a **Very High Consequence Classification** rating.

Based on SANS 10286, the Valley TSF has a **High Hazard classification** rating.

The minimum Factor of Safety against failure, based on the Limit Equilibrium method of stability analysis, is 2.0 under drained conditions, 1.6 under undrained conditions, 1.2 under post seismic, post liquefaction or residual conditions and 1.3 under pseudo static conditions. These Factors of Safety comply with the local legislation and international slope stability standards.

Most dormant up-stream deposited facilities, including FSN1 and FSN2, do not meet new legislated Factor of Safety requirements. To ensure the entire complex complies at closure, remedial works for FSN1 and FSN2 may be incorporated into the Valley TSF closure plan. Conceptual-level work has been carried out to assess the required remedial work based on the limit equilibrium method for stability calculations. This work will be updated once the proposed stability assessments using finite element analyses are conducted on Harmony's dams.

The gold tailings material classified as a Type 3 waste according to the waste classification report by Jones and Wagner. This necessitates a Class C barrier system. However, as per an independent review by Legge and Associates, an 'inverted barrier' system can be used. The inverted barrier reduces seepage by changing the flow through the liner from Bernoulli flow at discontinuities to D'Arcian flow controlled by the tailings permeability at these points. The stability of the TSF is also improved by omitting lower strength compacted clay layers and the geomembrane cushion layer (replaced by tailings). The inverted barrier system is used in the design of the Valley TSF barrier system.

The Valley TSF barrier system has two different areas. Liner area 1 is within the central area of the dam basin. This liner system comprises (from top down), a 300mm thick layer of tailings, above liner drains, 1.5mm smooth HDPE liner underlain by a 300mm ripped and recompacted in-situ base layer.

Liner area 2 is present at the outer walls of the facility where high liner stresses exist and a 150T geogrid (or similar approved) is required. The geogrid (or similar approved) will be placed from the toe wall inwards for 50m. This liner system comprises (from top down), a 300mm thick layer of tailings, a 150T size geogrid (or similar approved), a 300mm thick layer of tailings, above liner drains, 1.5mm double textured HDPE liner underlain by a 300mm ripped and recompacted in-situ base layer.

The TSF underdrainage system is provided above the liner to intercept seepage through the facility. The above liner drains lower the phreatic surface, thereby improving the overall stability of the facility. The above liner drains comprise of blanket drains and herringbone drains.

The herringbone drains pipes comprise of 160mm slotted Drainex HDPE pipes surrounded in 19mm stone which is enclosed in a geofabric. These drains are spaced 100m apart. The blanket drains comprise of 160mm slotted Drainex HDPE pipes surrounded in 19mm stone overlain by a layer of 6mm stone and graded filter sand which is enclosed in a geofabric.

All above liner drains in the south-east section discharge into the solution trench located to the south of Valley TSF and water will flow to the existing Return Water Dam (RWD). The above liner drains on the north-western section discharge into the solution trench located to the north-west of Valley TSF and will flow to the new RWD.

The under-liner leakage detection drains on the Valley TSF comprise of 160mm slotted Drainex HDPE pipes surrounded in 19mm stone which is enclosed in a geofabric. Similarly to the above-liner drains, the south-eastern under liner drains flow to the existing RWD and the north-western section discharges into the new RWD.

A 150mm thick reinforced concrete lined solution trench is provided along the north-west, south and south-eastern sections of the TSF. The trapezoidal solution trench is 1m deep with side slopes of 1V:1.5H and a base width of 1m. The solution trench on the north-western section of the TSF will accommodate the maximum peak discharge from the penstock of 1.02m^3/sec and flows into the new RWD. The solution trench on the south and south-eastern sections of the TSF will accommodate drain flow only of 46.14m^3/day and flows into the existing RWD.

A hydrotechnical assessment was done to determine climatic and meteorological data. This data was used to size the new RWD situated north-west of the TSF and the associated water infrastructure. A capacity assessment was carried out on the existing RWD, situated south-west of the TSF.

The new Return Water Dam has a total storage capacity of 220 000m^3 which is sufficient to ensure that it does not spill more than once every 50 years with the inflow from the penstock and underdrains on the north-west of the TSF, when operated at a level of 0.3m.

The new Return Water Dam liner system comprises 200mm high geocells filled with 20Mpa concrete, underlain by a 1.5mm thick smooth HDPE liner and a 300mm in-situ base preparation layer. The underdrainage comprises 160mm slotted HDPE pipes encased in 19mm washed stone. The stone will be wrapped in geofabric.

A concrete lined spillway is provided at the new RWD to safely discharge excess water without overtopping of the RWD embankment walls. The RWD spillway has a freeboard of 800mm and has been designed to discharge the 1:10 000 24-hour Probable Maximum Flood volume of 9.9m^3/sec.

A silt trap is installed upstream of the new RWD. The silt trap includes infrastructure to enable cleaning. The silt trap allows solids to settle out of the water before entering the RWD, thereby minimising sedimentation in the RWD. The silt trap is a 2.0m deep reinforced concrete water retaining structure with a concrete spillway to route de-silted water to the RWD.

A capacity assessment was done on the existing RWD, which has a capacity of 300 000m^3. The inputs to this dam are low, as only drain water and rainfall will flow to the RWD. Due to evaporation and seepage, the dam is not expected to hold more than 50 000m3 and easily accommodates the expected inputs.

Concrete poles with warning signs will be installed around the TSF. A 5m wide access road is provided around the facility for operational and monitoring requirements.

The facility is to be constructed and operated to ensure that the future designed outer slope profile is achieved and to ensure the safe, efficient and environmentally responsible management of the Valley TSF and associated infrastructure.

The recommended budget allocation for the Valley TSF with a typical Class C barrier system was R750 million. The recommended budget allocation for the Valley TSF with an 'inverted barrier' is R690 million (including 20% contingency and professional fees). The budget allocation with an 'inverted barrier' has reduced by R60 million.

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List of abbreviations

DWS	:	Department of Water and Sanitation
FOS	:	Factor of Safety
Geotheta	:	Geotheta (Pty) Limited
Ha	:	Hectare
HDPE	:	High Density Polyethylene
MCE	:	Maximum Credible Earthquake
OMC	:	Optimum Moisture Content
RWD	:	Return Water Dam
PMF	:	Probable Maximum Flood
TLB	:	Tractor Loader Backhoe
TSF	:	Tailings Storage Facility
WULA	:	Water Usage Licence Application
FOS	:	Factor of Safety
ASTM	:	American Society for Testing and Materials
SANS	:	South African National Standards
GRI	:	Geosynthetic Research Institute
AASHTO	:	American Association of State Highway and Transportation Officials
CBR	:	California Bearing Ratio
SCPTu	:	Seismic Cone Penetration Test (with pore water pressure)
CQA	:	Construction Quality Assurance
GISTM	:	Global Industry Standard on Tailings Management
GN	:	Government Notice
kPa	:	kilo Pascal
LC	:	Leachable Concentration
LCT	:	Leachable Concentration Threshold
mamsl	:	Meters above mean sea level
MAP	:	Mean Annual Precipitation
Mod	:	Modified
NEMWA	:	National Environmental Management: Waste Act No 59 of 2008
NHRA	:	National Heritage Act
SABS	:	South African Bureau of Standards
SANCOLD	:	South African National Committee on Large Dams
SAWS	:	South African Weather Service
TC	:	Total Concentration
TCT	:	Total Concentration Threshold
UV	:	Ultraviolet
ZOI	:	Zone of Influence

1. Introduction

1.1 Geotheta was appointed by Harmony Gold for the design of the proposed Valley Tailings Storage Facility (TSF) in Welkom, South Africa.

1.2 The Valley TSF is to be constructed between Harmony FSN1 and FSN2 on a portion of the footprint of the Harmony FSN4 TSF.

1.3 This is the design report for the Valley TSF and associated infrastructure.

2. Tailings Storage Facility location

2.1 The Valley TSF is located approximately 8km north-west of Welkom Central in the Free State Province, South Africa.

2.2 The northern boundary of the site is demarcated by the R34 roadway. The R30 and the R710 roadways delineate the eastern and southern limits respectively. The Valley TSF location is shown in Figure 1 below.

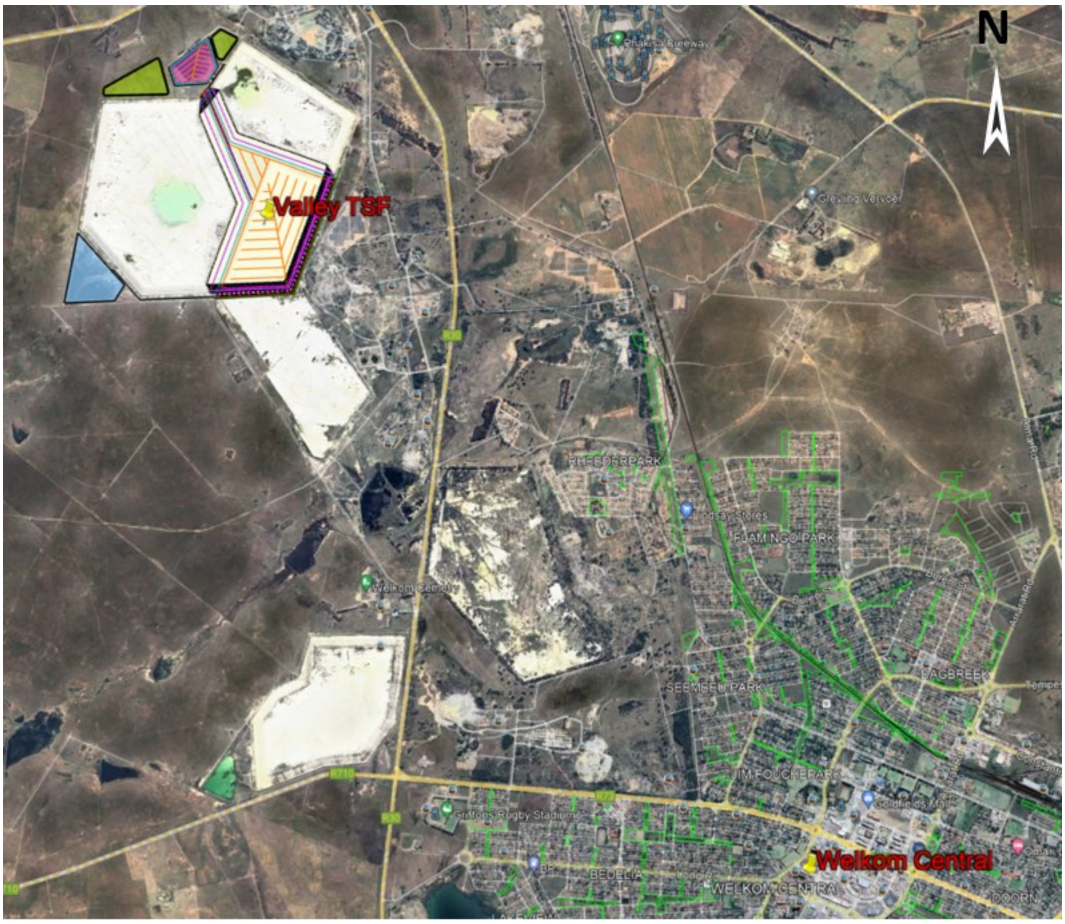

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Figure 1: Valley TSF location

3. Terms of reference

3.1 Harmony Gold submitted an invitation to tender for the design of Valley TSF on 19 October, tender number CL202209(13)DG.

3.2 Geotheta submitted proposal reference 2210513 - Harmony – Valley TSF Design - P01 on 05 November 2022.

3.3 A letter of award for the design of the Harmony Valley TSF (contract number: FG/23/01/0003) was issued to Geotheta on 31 May 2022.

3.4 Subsequent to the above submission, Geotheta were requested to review the barrier design for the Valley TSF.

3.5 Geotheta submitted proposal reference 2210513 - Harmony – Valley TSF Design - P02 on 06 December 2023 and received a confirmation of this order on 12 December 2024.

4. Legislative requirements

4.1 Construction and operation of a mine residue facility or pollution control facility requires adherences to specific legal regulations. A summary of the legal documents are:

- National Environmental Management Waste Act (Act 59 of 2008) (NEMWA).
- Environmental Conservation Act (Act 25 of 1989).
- National Water Act, 1998 (Act 36 of 1998).
- Natural Environmental Management Act (Act 107 of 1998) (NEMA).
- National Heritage Resources Act (Act 25 of 1999) (NHRA).

5. Standards and guidelines

- Harmony complies with the SANS 10286 Code of Practice for Mine Residue.
- Global Industry Standard on Tailings Management (GISTM).
- American Society for Testing and Materials (ASTM).
- South African National Standards (SANS) 1526 (2015).
- South African National Standards (SANS) 10409.
- South African National Standards (SANS) 1200.
- GN 636: National Norms and Standards for Disposal of Waste to Landfill
- ICOLD Bulletin 56 of 1986.
- Geosynthetic Research Institute (GRI) GM13.

6. Scope of work

The following was done in terms of the agreed scope of work:

6.1 Project kick-off

- The site was visited to get an understanding of the general topography.
- Prior study reports and documents were received and reviewed.
- The topographical survey was received and analysed.
- The design criteria were confirmed.

6.2 Geotechnical Work

- Review of prior Jones and Wagner geotechnical reports.
- Additional geotechnical works required were identified - Geotechnical and SCPTu report submitted separately (report reference: 2210513 – Harmony – Valley TSF SCPTu Report - R07).

6.3 Valley TSF Deposition Requirements

- Modelling and stage capacities.
- Assessment of cyclone and day wall paddock systems.
- Deposition method was recommended and selected.

6.4 Valley TSF design and drawings

- Preparatory earthworks design.
- Dam break analysis to determine the zone of influence and consequence classification rating for the facility.
- Hydrology and meteorology, including climate change increases in rainfall and evaporation.
- Catchment paddock designs.
- Construction methodology was determined and specified.
- Seepage analyses.
- Design of the inverted barrier system.
- Stability analyses.
- Underdrainage design.
- Penstock and catwalk design.
- Penstock outfall pipeline design.
- Slurry delivery piping design.
- Solution trench sizing.
- Storm diversion systems.
- Access roads.
- Closure and aftercare recommendations.

6.5 RWD design

- Stochiometric sizing to ensure the new Return Water Dam and existing Return Water Dam do not spill more than once every 50 years.
- New RWD modelling to determine the optimum construction works and costs.
- Existing RWD capacity assessment.
- Earthworks design of walls, base, and trenches. This includes the silt trap sizing and detailing.
- Barrier system design.

- Decant design.

6.6 **Drawings**

- Drawings to "Issued for Information" level.

6.7 **Bill of Quantities**

- A bill of quantities was prepared and costed.

6.8 **Reports**

- Dam break analysis report.
- Design report.
- Construction specifications.
- A Liner Construction Quality Assurance (CQA) was prepared.
- Operating, maintenance and surveillance manual.
- Geotechnical and SCPTu report.

7. Exclusions from the scope of work

The following was qualified and excluded from the scope of work:

- Ground survey work. A digital terrain model was provided by the client.
- A residue material characterisation report was provided by the client.
- Liaison or application for permissions/permits from government authorities.
- Environmental investigations or studies.
- Participation and consultation with I&AP's.

8. Consequence classification

8.1 The consequence classification of the Valley TSF was determined from the Dam Break Analysis (DBA) zone of influence and applying the consequence classification criteria indicated in the Global Industry Standard and Tailings Management.

8.2 The Valley TSF is categorised as a **Very High Consequence Classification** facility according to the Global Industry Standard on Tailings Management consequence classification criteria. This consequence has been determined by analysing the impact a failure would have on the life, environment and infrastructure in the inundation zone modelled during the dam break analysis.

8.3 The Zone of Influence indicates the overall zone of influence and the delineated background flood is shown below in Figure 2. This figure indicates the sum of all the potential failures on all flanks on the TSF under worst case conditions, which is unlikely. In the unlikely event of failure at any time, it would probably only happen on one flank.

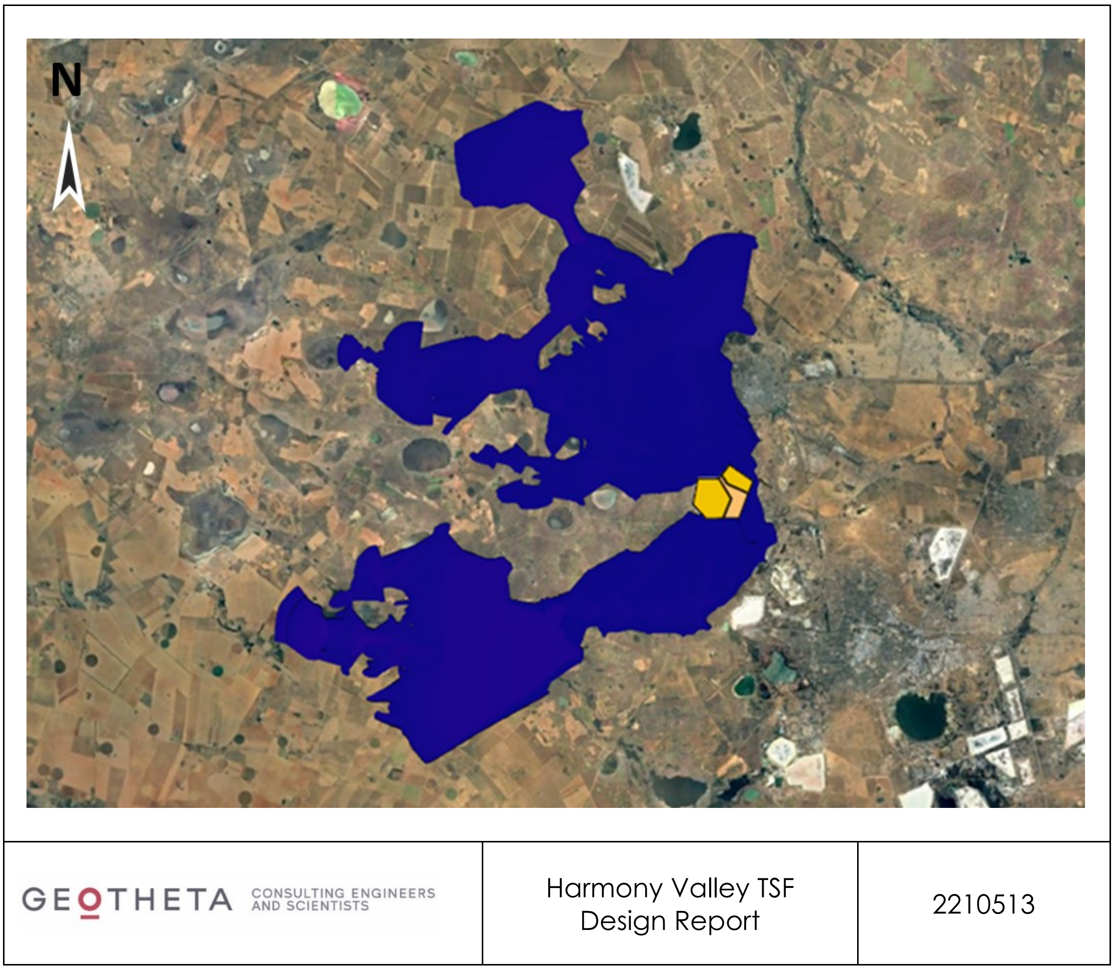

Figure 2: Overall zone of influence and the delineated background flood

8.4 In the unlikely event of a Dam Break, high economic losses affecting infrastructure are anticipated within the zone of influence of the facility. The affected infrastructure comprises the mine's own access road, solution trench, return water dam and the silt trap (all part of this design). Other infrastructure such as farmhouses and nearby mining operations may also be affected.

8.5 Major environmental losses or deterioration of habitat are expected within the zone of influence footprint area. A potential Dam Break will inundate and cause significant deterioration of the surrounding environment.

8.6 There is permanent identifiable population at risk within the zone of influence. These are the permanent operating staff and a residential area north of the facility. The potential population at risk is between 100 - 1000. The potential loss of life is considered to be ten or fewer based on a staff compliment of 8 persons.

8.7 Therefore, based on the above, and GISTM the Valley TSF has a **Very High Consequence Classification** rating.

8.8 Refer to Table 1 below for the GISTM consequence classification.

Table 1: GISTM consequence classification criteria

Dam Failure Consequence Classification	Incremental Losses					
	Potential Population at Risk	Potential Loss of Life	Environment	Health, Social & Cultural	Infrastructure & Economics	Livelihoods
Low	None	None expected	Minimal short-term loss or deterioration of habitat or rare and endangered species.	Minimal effects and disruption of business. No measurable effect on human health. No disruption of heritage, recreation, community or cultural assets.	Low economic losses; area contains limited infrastructure or services. <US$1M	Up to 10 household livelihood systems disrupted and recoverable in the short term. No long-term non-recoverable loss of livelihoods.
Significant	Temporary only	None Expected	No significant loss or deterioration of habitat. Potential contamination of livestock/fauna water supply with no health effects. Process water low potential toxicity. Tailings not potentially acid generating and have low neutral leaching potential. Restoration possible within 1 to 5 years	Significant disruption of business, service or social dislocation. Low likelihood of loss of regional heritage, recreation, community or cultural assets. Low likelihood of health effects.	Losses to recreational facilities, seasonal workplaces, and infrequently used transportation routes. <US$10M	Up to 10 household livelihood systems disrupted and recoverable in the longer-term: or Up to 100 household livelihood systems disrupted and recoverable in the short-term. No long-term non-recoverable loss of livelihoods
High	10-100	1-10	Significant loss or deterioration of critical habitat or rare and endangered species. Potential contamination of livestock/fauna water supply with no health effects. Process water moderately toxic. Low potential for acid rock drainage or metal leaching effects of released tailings. Potential area of impact 10 km2 - 20 km2. Restoration possible but difficult and could take > 5 years	500-1,000 people affected by disruption of business, services or social dislocation. Disruption of regional heritage, recreation, community or cultural assets. Potential for short term human health effects.	High economic losses affecting infrastructure, public transportation, and commercial facilities, or employment. Moderate relocation/compensation to communities. <US$100M	Up to 10 household livelihood systems lost and non-recoverable: or Up to 50 household livelihood systems disrupted and recoverable over the longer-term: or Up to 200 household livelihood systems disrupted and recoverable in the short term.
Very High	100-1000	10 to 100	Major loss or deterioration of critical habitat or rare and endangered species. Process water highly toxic. High potential for acid rock drainage or metal leaching effects from released tailings. Potential area of impact >20 km2. Restoration or compensation possible but very difficult and requires a long time (5 years to 20 years).	>1,000 people affected by disruption of business, services or social dislocation for more than one year. Significant loss of national heritage, community or cultural assets. Potential for significant longer-term human health effects.	Very high economic losses affecting important infrastructure or services (e.g., highway, industrial facility, storage facilities, for dangerous substances), or employment. High relocation/compensation to communities. <US$1B	Up to 50 household livelihood systems lost and non-recoverable: or Up to 200 household livelihood systems disrupted and recoverable over the longer-term: or Up to 500 household livelihood systems disrupted and recoverable in the short term.
Extreme	>1000	More than 100	Catastrophic loss of critical habitat or rare and endangered species. Process water highly toxic. Very high potential for acid rock drainage or metal leaching effects from released tailings. Potential area of impact > 20 km2. Restoration or compensation in kind impossible or requires a very long time (>20 years).	>5,000 people affected by disruption of business, services or social dislocation for years. Significant national heritage or community facilities or cultural asset destroyed. Potential for severe and/or longer-term human health effects.	Extreme economic losses affecting critical infrastructure or services, (e.g., hospital, major industrial complex, major storage facilities for dangerous substances) or employment. Very high relocation/compensation to communities and very high social readjustment costs. >US1B	More than 50 household livelihood systems lost and non-recoverable; or More than 200 household livelihood systems disrupted and recoverable in the longer-term; or More than 500 household livelihood systems disrupted and recoverable in the short term.

9. Waste classification

9.1 Testing was done by Waterlab (Pty) Ltd (facilitated by Jones and Wagner) to determine the geochemical properties as well as the waste classification of the tailings material. Only the conclusions of the waste classification are discussed in this report. The detailed waste classification report prepared by Jones and Wagner is included in Appendix A.

9.2 The waste classification is determined by assessing the total concentration (TC) of a material and its leachable concentration (LC) to the guidelines provided in Regulation 635 of NEMWA.

9.3 The applicable leachable or total concentration thresholds are used to classify the waste into several categories as shown in Table 2.

Table 2: Waste type classification by total and leachable concentration thresholds

Total Concentration Threshold (TCT)	Link between TCT and LCT	Leachable Concentration Threshold (LCT)	Waste Type	Barrier System
< TCT0	and	< LCT0	Type 4	Class D
< TCT1	and	< LCT1	Type 3	Class C
< TCT1	and	< LCT2	Type 2	Class B
< TCT2	or	< LCT3	Type 1	Class A
> TCT2	or	> LCT3	Type 0	Not allowed

9.4 The total concentration leachable concentration results were compared to the Total Concentration Threshold (TCT) and Leachable Concentration Threshold (LCT) values as prescribed in GN 635: National Norms and Standards for the Assessment of Waste for Landfill Disposal.

9.5 The geochemical assessment concluded that the gold tailings material is a Type 3 waste (from the classification parameters set out by the National Environmental Management Waste Act (Act 59 of 2008)).

9.6 GN 636: National Norms and Standards for Disposal of Waste to Landfill requires a Class C barrier system for a Type 3 waste. A typical Class C barrier system is illustrated in Figure 3.

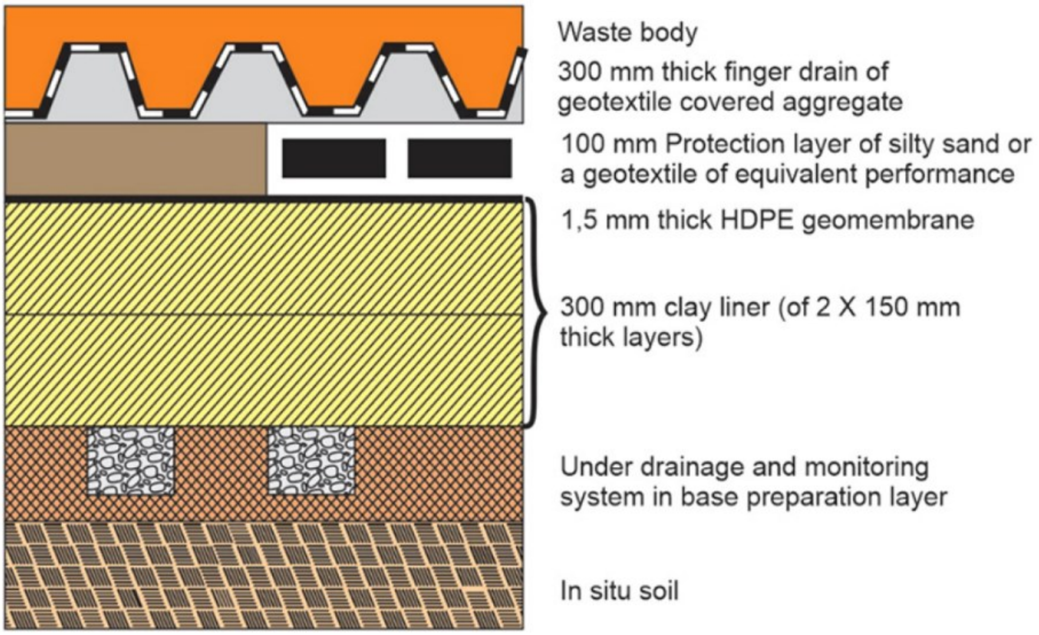

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Figure 3: Typical Class C Liner

9.7 An alternative barrier system (inverted barrier), as recommended by Legg and Associates, is used in the design of the Valley TSF.

9.8 The inverted barrier system has superior performance in terms of reducing seepage when compared to the Class C barrier system. This is due to the tailings above the liner being in direct contact with the geomembrane thus the fine tailings particles clog holes or discontinuities and change the flow through the geomembrane from orifice, flow controlled by Bernoulli's equation, to Darcian flow. The seepage flow rate for the previous design of the TSF with a Class C barrier system was estimated to be 140l/ha/day. The seepage flow rate through an 'inverted barrier' has been reduced to 18l/ha/day. The service life of both barrier systems are the same.

9.9 The report by Legge and Associates that addresses the suite of legislation (reference: Harmony Geotheta Legislation on Source Pathway Receptor Risk Modelling) is included in Appendix E.

9.10 The barrier systems used for the TSF and RWD are detailed in Sections 14 and 22 of this report.

10. Design Criteria

10.1 From the consequence classification determined above, the flood and seismic design criteria return period is 1:10 000 years. The magnitudes of each of these are provided below.

Table 3: Flood and seismic design criteria for operation

Design flood event (1:10 000) (determined by Geotheta)	Design Seismic Event (1:475) (from Seismic Hazard Map of South Africa)
1 day: 200mm	0.22g
2 days: 245mm	
3 days: 276mm	

10.2 According to Hynes-Griffin (1984), half (50%) of the peak ground acceleration value should be used as the recommended horizontal seismic coefficient in a limit equilibrium stability analysis. The expected peak horizontal acceleration with a 10% probability of being exceeded at least once in a period of 475 years at the location of Valley TSF is 0.22g (from the Council for Geosciences). The horizontal acceleration used in the pseudo static stability analyses was therefore 0.11g (=0.22g x 50%).

10.3 The agreed design criteria for slope stability requirements are:

Table 4: Slope stability design criteria

Criteria	**Minimum FOS**
Minimum required slope stability Factor of Safety – Drained conditions	1.5
Minimum required slope stability Factor of Safety – Undrained conditions (peak shear strengths)	1.3
Minimum required slope stability Factor of Safety – post seismic, post liquefaction or residual strength conditions	1.1
Minimum required slope stability Factor of Safety – Pseudo-static conditions	1.1

10.4 Drained conditions represent the strength conditions applied in which soils may either drain or, because they are dense and dilate during shearing lead to maintenance or a reduction in pore pressures. For this case, the effective strength parameters are used in the stability analyses.

10.5 Undrained conditions represent the strength conditions applied in which silty/clayey soils cannot drain during shearing. The loading initiates an increase in pore pressures and therefore undrained behaviour. For this case, the undrained strength parameters are used in the stability analyses.

10.6 Post seismic, post liquefaction or residual strength conditions represent the soil strength after liquefaction or significant shearing (deformation), which may be caused by seismic or static movements. This case would not include the horizontal driving forces of the earthquake or movement. For this case, the residual undrained strength parameters are used in the stability analyses.

10.7 Pseudo-static conditions represent the seismic loading that is modelled as a statically applied inertial force, the magnitude of which is a product of a seismic coefficient k and the weight of the potential sliding mass.

10.8 The following design criteria was established for the project:

Table 5: General design criteria

Criteria	Unit	Value
Tailings deposition period	years	8.0 years
Tailings deposition rate	tons/month	600 000
Tailings in-situ dry density	tons/m^3	1.45
Tailings slurry density	tons/m^3	1.45
Maximum allowable rate of rise (Basin)	m/year	4.12
Maximum allowable rate of rise (Embankment)	m/year	3.99
Maximum RWD spill frequency	frequency	Once every 50 years
Minimum RWD freeboard above spill level	mm	800
1:50 year 24-hour rainfall	mm	127
1:100 year 24-hour rainfall	mm	142
1:10 000 year 24-hour rainfall	mm	240
Design pseudo static earthquake Peak Ground Acceleration (1:475 year return period)	g	0.22
Design GISTM earthquake Peak Ground Acceleration (1:10 000 year return period)	g	0.30
Minimum required slope stability Factor of Safety – Drained conditions	-	1.5
Minimum required slope stability Factor of Safety – Undrained conditions (peak shear strengths)	-	1.3
Minimum required slope stability Factor of Safety – post seismic, post liquefaction or residual strength conditions	-	1.1
Minimum required slope stability Factor of Safety – Pseudo-static conditions	-	1.1
Waste type	-	Type 3
Barrier system	-	Inverted Barrier system

10.9 To achieve maximum storage capacity, the Valley TSF cyclone walls on the North-western, Southern, and South-Eastern flanks will be developed at an intermediate outer slope of 1V:3H between benches.

10.10 At closure, the Valley TSF cyclone wall outer slopes will have an overall slope of 1V:4H.

10.11 The cyclone wall outer slope will allow for sufficient vegetation growth after topsoiling and minimise erosion of the outer slopes after closure.

11. Hydrotechnical assessment

A hydrotechnical assessment was done to determine the climatic and meteorological data. This data was used to size the new Return Water Dam (RWD) situated north-west of the TSF and to do a capacity assessment on the existing RWD situated south-west of the TSF.

11.1 Climate and meteorological data

11.1.1 The average monthly rainfall for the site was obtained from the Olivine Station (SAWS station No. 0328726 W) and the average monthly lake evaporation was based on the Sand Vet Sentrum C4E009. The data for the respective stations was obtained from the Water Research Commission Report No. 298/2.1/94.

11.1.2 Monthly precipitation and evaporation data are provided in Table 6.

Table 6: Monthly rainfall and evaporation data

Month	Rainfall (mm)	Lake evaporation (mm)
January	83.9	244.8
February	71.4	189.1
March	71.9	162.3
April	43.2	104.8
May	18.9	72.5
June	7.4	47.4
July	7.5	57.2
August	8.5	88.7
September	16.9	139.2
October	47.8	183.9
November	67.5	211.7
December	69.4	247.6

11.1.3 The storm rainfall depths were obtained from the design rainfall software (Smithers and Schulze, 2002). This provides rainfall depths for various durations up to a 1:200 year return period.

11.1.4 The maximum total rainfall depths for various return periods and storm durations are provided in Table 7 below.

Table 7: Storm rainfall depths

Duration	Rainfall Depth (mm)						
(m/h/d)	1:2 year	1:5 year	1:10 year	1:20 year	1:50 year	1:100 year	1:200 year
5 m	9	12	14	17	20	22	24
10 m	13	18	21	25	29	32	36
15 m	17	23	27	31	37	41	45
30 m	21	29	34	39	46	52	57
45 m	25	33	39	45	53	59	66
1 h	27	37	43	50	59	65	73
1.5 h	31	42	50	57	67	75	83
2 h	34	46	55	63	74	83	92
4 h	40	54	63	73	86	96	107
6 h	43	59	69	80	94	105	117
8 h	46	62	74	85	100	112	124
10 h	49	66	77	89	105	117	130
12 h	51	68	81	93	109	122	135
16 h	54	73	86	99	116	130	144
20 h	56	76	90	104	122	136	151
24 h	59	79	94	108	127	142	157

11.1.5 The rainfall depths beyond a 1:200-year return period was determined using logarithmic extrapolation of the available rainfall data. This is shown in Figure 4 below.

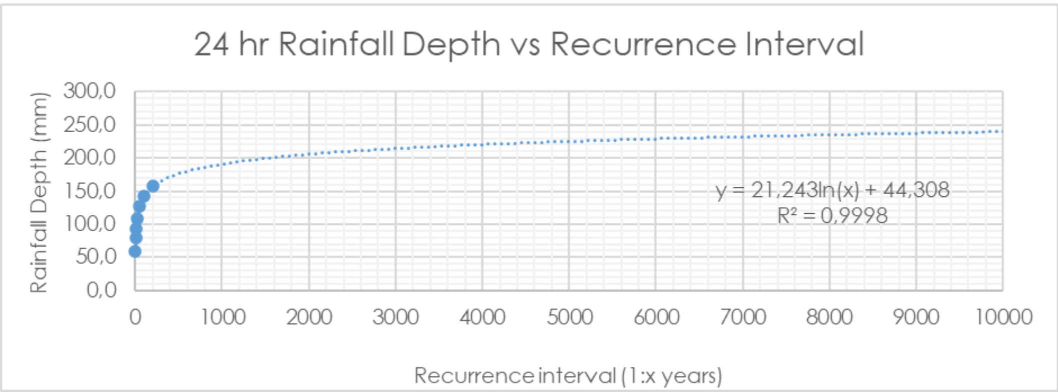

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Figure 4: Determination of the PMF

Table 8: Extrapolated rainfall data.

Input data	
Recurrence interval (years)	**24 hr rainfall depth (mm)**
2	58.6
5	79.2
10	93.5
20	107.7
50	126.9
100	141.9
200	157.3
Output data	
1 000	191.0
2 475	210.3
5 000	225.2
10 000	240.0

11.1.6 Monthly temperatures were obtained from meteoblue.com. Average monthly temperatures and ranges are provided in Table 9 below.

Table 9: Temperature ranges

Month	Minimum temperature (° Celsius)	Maximum temperature (° Celsius)	Average temperature (° Celsius)
January	16.0	32.0	24.0
February	15.0	31.0	23.0
March	13.0	29.0	21.0
April	9.0	26.0	17.5
May	5.0	22.0	13.5
June	1.0	19.0	10.0
July	0.0	19.0	9.5
August	2.0	22.0	12.0
September	7.0	27.0	17.0
October	11.0	29.0	20.0
November	13.0	30.0	21.5
December	16.0	31.0	23.5

11.1.7 The monthly climatic data summary is provided in Figure 5.

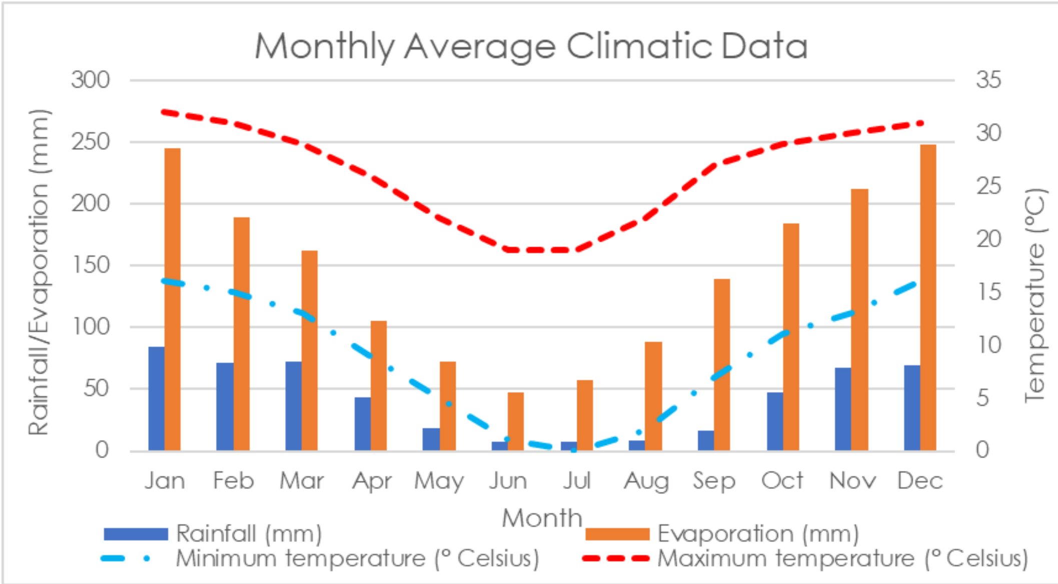

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Figure 5: Monthly Climatic Data Summary

12. Valley TSF Design

The layout of the Valley Tailings Storage Facility is shown below.

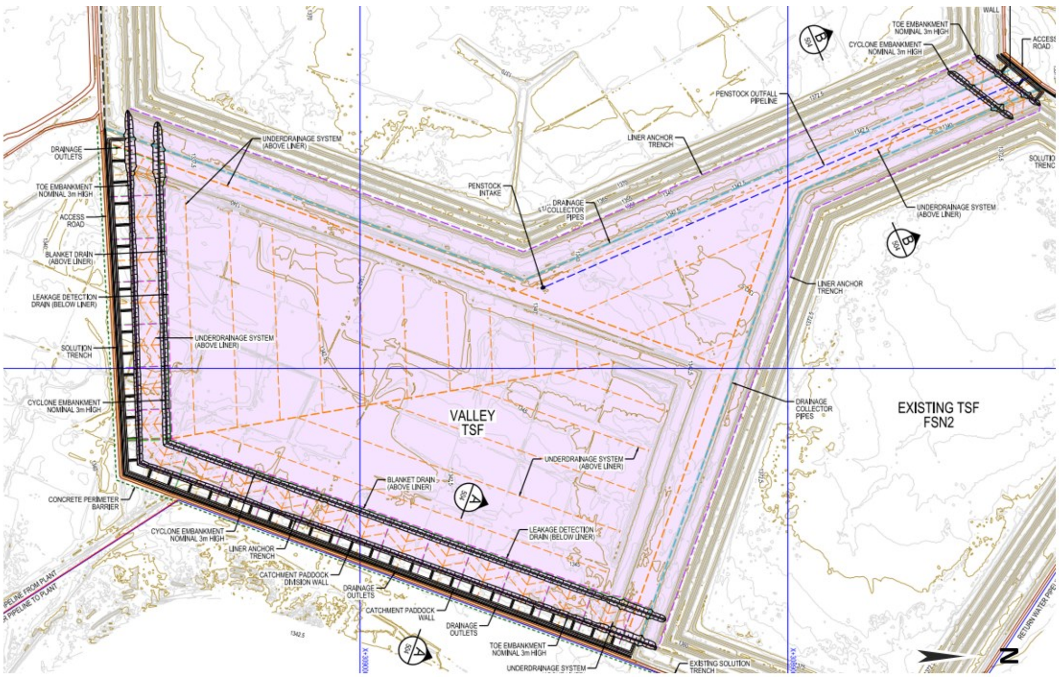

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Figure 6: Layout of Valley TSF

12.1 TSF stage capacity assessment

12.1.1 The Valley TSF will have a maximum height of 36m and a footprint area of approximately 163.5Ha.

12.1.2 The designed outer profile comprises an overall outer slope of 1V:4H with 8.0m high intermediate slopes of 1V:3H between each 8.0m wide bench. The Valley TSF outer profile configuration is shown in Figure 7 below.

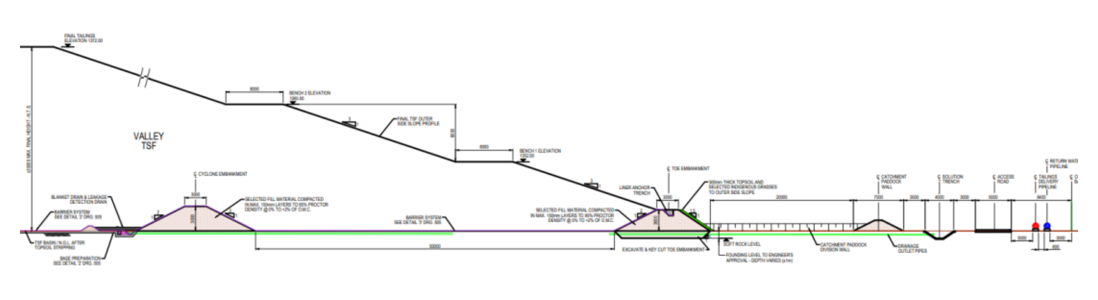

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Figure 7: TSF outer profile configuration

12.1.3 The Valley TSF is designed to be an upstream cyclone facility. From the stage capacity assessment, the maximum rate of rise is 4.12m/year (basin) and 3.99m/year (embankment).

12.1.4 No cyclone operation will occur at the FSN1 and FSN2 TSF interface: spigotting or open-end deposition will be done for pool control only.

12.1.5 Stage capacities were developed for the Valley TSF based on a tailings in-situ dry density of 1.45 tons/m^3 at the design outer profile. The facility provides storage of 56.8 million tons over 8.0 years at 600 000tpm.

Table 10: Stage capacity results

Description	Unit	Value
Deposition rate	tons/month	600 000
Storage capacity	million tons	56.8
Max rate of rise (Basin)	m/year	4.12
Max rate of rise (Embankment)	m/year	3.99
Tailings underflow mass split	%	17
Tailings overflow mass split	%	83
Deposition period	years	8.0

12.1.6 The stage capacity relationships are indicated in Figure 8 below.

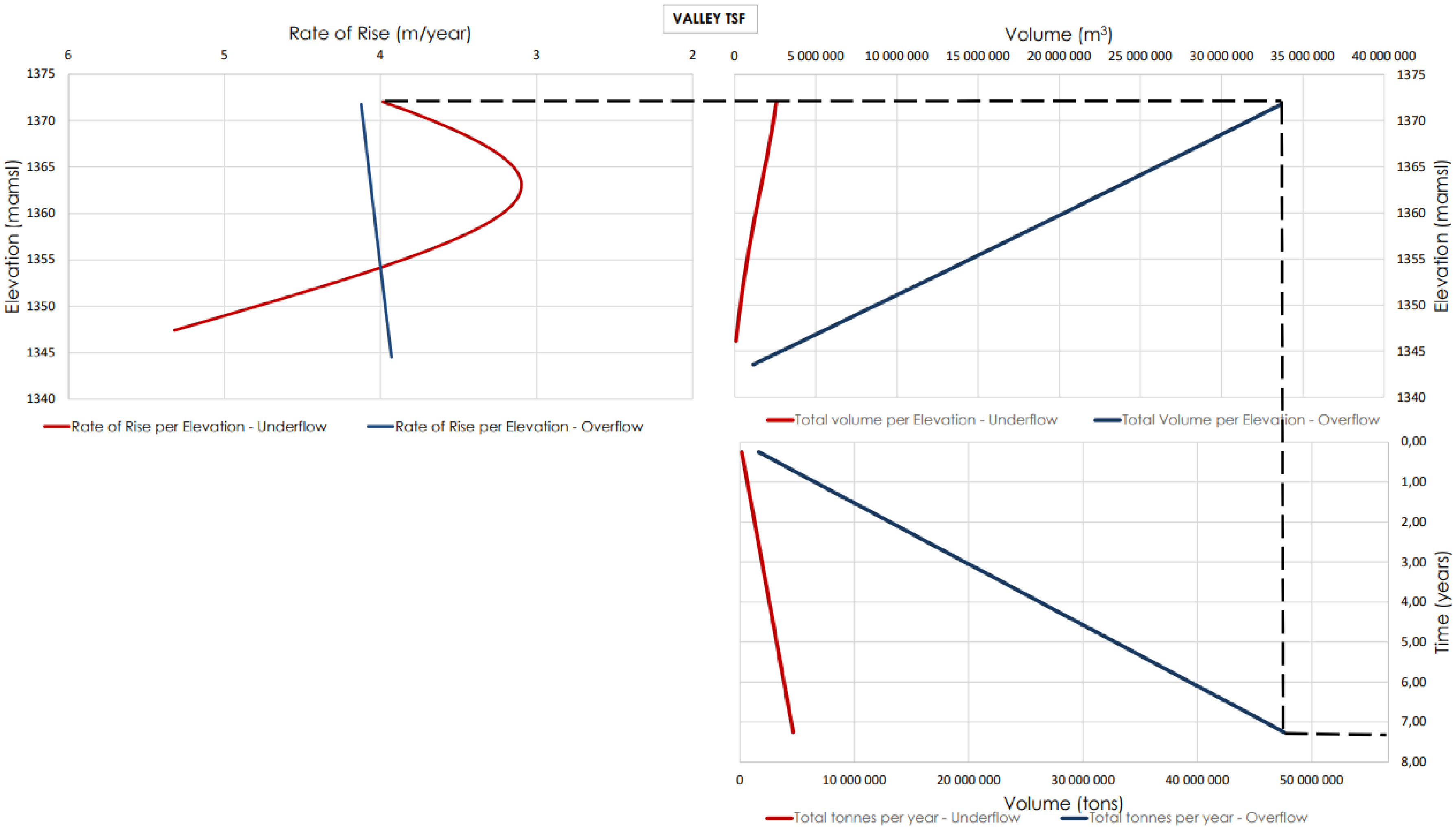

Figure 8: TSF stage capacity graphs

12.1.7 The detailed stage capacity graphs and outputs are included in Appendix B.

12.2 **Engineering geology**

12.2.1 From the 1:250 000 2726 Kroonstad geological map, the site consists of Quaternary Aeolian sands underlain by mudstone, siltstone and shale of the Volksrust Formation from the Ecca Group from the Karoo Supergroup.

12.2.2 The influence of climate on weathering is expressed by the N-value (H.H. Weinert 1980). Where N is more than 5, mechanical disintegration is dominant, and where N is less than 5, chemical decomposition is dominant.

12.2.3 The Weinert N-value is 4.7 for this region, indicating that decomposition is dominant. This decomposition means that there are finer, more impermeable layers overlying deeper permeable material and then the basement rocks.

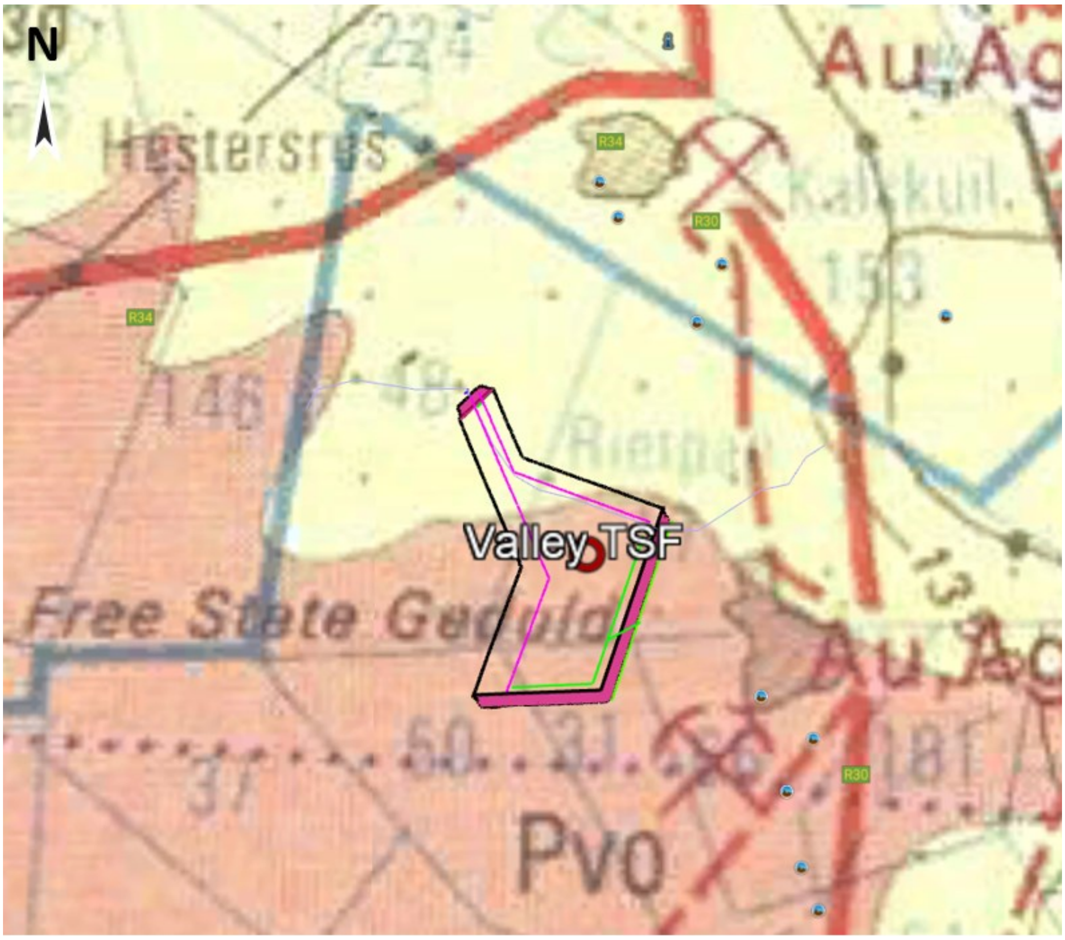

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Figure 9: Regional geology

Previous testing/fieldwork

12.2.4 Jones and Wagener investigated and evaluated the founding conditions for the Valley TSF. The geotechnical investigation was completed to characterise the soil profile, evaluate the geotechnical conditions and give founding recommendations for the subgrade preparations of the Valley TSF.

12.2.5 Sampling and geotechnical testing of the materials at the Valley TSF was undertaken by Jones & Wagner in October 2008 (JAWS report reference: JW150/08/B680 – Rev 0). This geotechnical report has been included in Appendix C. The results from this test work provided the existing gold tailings geotechnical parameters.

12.2.6 Twenty-three test pits were excavated and profiled around the site area. One of these test pits, TP 30 is located within the Valley TSF footprint. Samples were taken of representative horizons to determine the physical properties of the horizons. All test pits were excavated either to reach or refusal.

12.2.7 The laboratory tests that were undertaken are grading and indicators, Mod. AASHTO, permeability and shearbox testing.

12.2.8 The soil profile in the site area comprises between 1.1m and 1.7m of moist, brown to orange-brown, clayey fine sand overlying a residual siltstone characterised by a soft to firm silty clay. This profile represents the central and western portions of the site while more clayey transported soils are encountered along the eastern boundary.

12.2.9 In the northern to north eastern portions between TSF's FSN1, FSN2 and FSN4, a thick layer of approximately 1.7m of slightly moist, brown, pinhole voided hillwash sand is present. Although the above mentioned represents the profile of the site area, variations will be encountered within the central section where approximately 0.5m of hillwash sand is present. In other areas the topsoil is clayey.

12.3 **SCPTu testing**

12.3.1 SCPTu testing was completed in June 2023. Seven SCPTu tests were done across the site to depths between 2.4m to 5.4m. The SCPTu test locations are indicated in Figure 10.

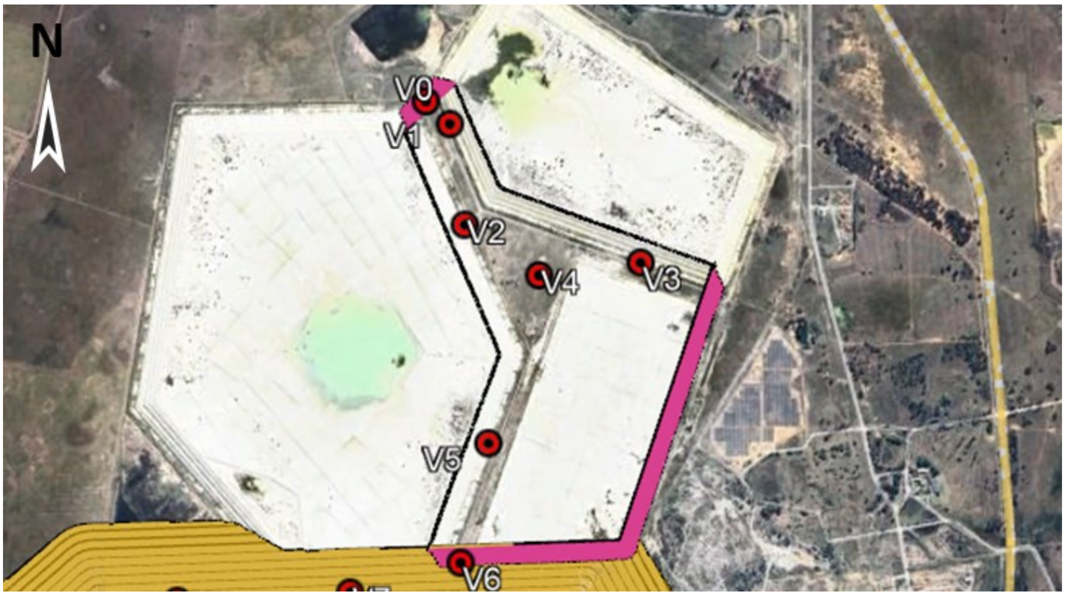

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Figure 10: SCPTu test locations

12.3.2 The SCPTu analysis was done using methods developed by P.K Robertson (2016) for SCPTu Testing. The SCPTu report is submitted as a separate report (report reference 2210513 – Harmony – Valley TSF SCPTu Report – R07).

12.3.3 A summary of the SCPTu test results is presented in Table 11.

Table 11: Summary of SCPTu test results

SCPTu No.	Probe depth (m)	No. of dissipation tests	Depth to phreatic surface (m)	Water table classification	Depth to foundation soils (m)
V0	4.8	5	0.9	Sub-hydrostatic	2.7
V1	4.6	5	1.2	Sub-hydrostatic	3.2
V2	5.0	5	0.4	Sub-hydrostatic	4.6
V3	5.4	5	0.6	Dry	5.4
V4	5.3	6	1.7	Dry	5.2
V5	4.6	5	1.2	Dry	3.4
V6	2.4	3	0.3	Dry	2.4

12.3.4 The cone resistance (qc), sleeve friction (fs), and cone induced/dynamic pore water pressure (u2) were used to derive the geotechnical parameters through accepted empirical correlations published by P.K Robertson (2016).

12.3.5 As agreed with Harmony Gold, the 20th percentile shear strength values of the SCPTu results are used.

12.3.6 The dissipation and pore water pressure data were used to interpret the phreatic surface level and the pore water pressure profiles.

12.3.7 The soil profile within the TSF footprint typically comprises the following material layers:

- Gold tailings material.
- Topsoil material
- Upper foundation material comprising predominantly sand and silty sand mixtures (Aeolian sand).
- Middle foundation material comprising clayey silty sand (Residual sandstone).
- Lower foundation material comprising sandy silt (Soft rock siltstone).

12.3.8 Dissipation results indicated that all SCPTu probes were either dry or encountered sub-hydrostatic pressures this shows that there is downward migration of ground water.

12.4 Engineering geotechnical parameters

12.4.1 Geotechnical engineering parameters were developed based on correlations between the findings from the geotechnical site investigation, similarly classified material as well as the analysis of the June 2023 SCPTu test results.

12.4.2 The summarised geotechnical engineering parameters used for slope stability analyses and the design of embankments are shown below in Table 12.

Table 12: Geotechnical parameters

			Mohr – Coulomb		SHANSEP (kPa)		
Material	**Unit Weight (kN/m^3)**	**Permeability (m/s)**	**Cohesion (kPa)**	**Phi (deg)**	**SHANSEP A**	**SHANSEP S**	**SHANSEP m**
Tailings	14.5	8.0E-08	0	32			
Tailings – undrained	14.5	8.0E-08					0.8
Tailings – residual	14.5	8.0E-08				0.05	
Starter wall	17.5	5.0E-07	8	32			
Hillwash (topsoil)	15.0	1.3E-03	0	34			
Aeolian sand	15.0	2.88E-07	0	34			
Residual sandstone	18.0	5.45E-08	0	28			
Soft rock siltstone	20.0	1.6E-07	0	36			
Cushion sand	14.6	8.0E-08	0	32			

			Mohr – Coulomb		SHANSEP (kPa)		
Material	**Unit Weight (kN/m³)**	**Permeability (m/s)**	**Cohesion (kPa)**	**Phi (deg)**	**SHANSEP A**	**SHANSEP S**	**SHANSEP m**
HDPE Liner – double textured	9.0	5.0E-12	0	18			
HDPE Liner – smooth	9.0	5.0E-12	0	6			
Base in-situ prep	16.0	2.88E-09	0	32			
Geogrid	5.0	8.8E-10	120	35			

12.5 **Geotechnical investigation conclusions and recommendations**

12.5.1 The site is underlain by silty sand (Aeolian sand), clayey silty sand (Residual sandstone) and sandy silt (Soft rock siltstone).

12.5.2 The silty sand will provide a suitable insitu base preparation layer as required for the recommended inverted liner system.

12.6 **Slope stability cross sections**

12.6.1 Slope stabilities of the final level walls were calculated along three cross sections for the Valley TSF. The cross-section locations are shown in Figure 11 and cover all sides of the facility that are not butted up against existing facilities.

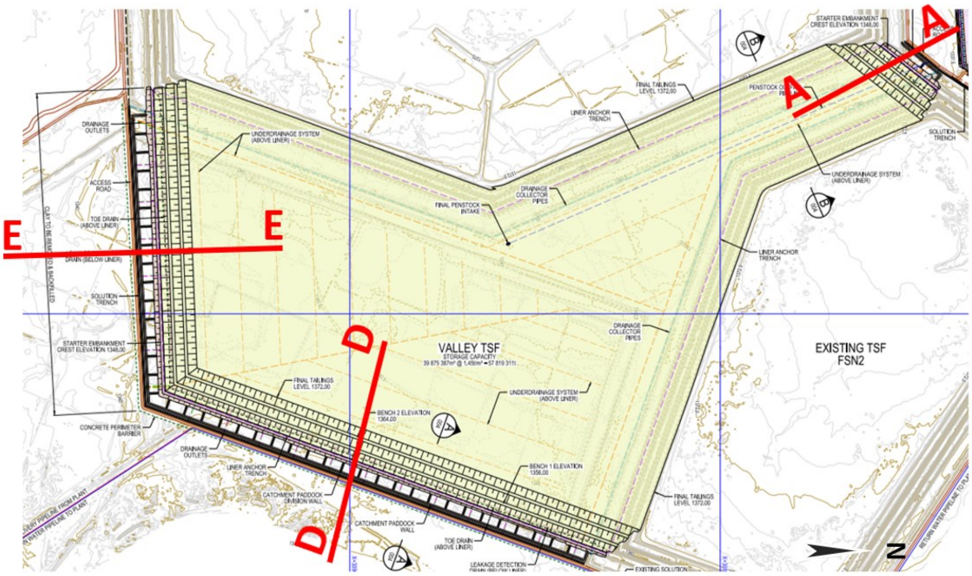

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Figure 11: Critical cross section location

12.6.2 The slope stabilities were analysed using RocScience Slide 2 slope stability software using the Cuckoo failure path search method.

12.6.3 The TSF cross sections were analysed at final height for drained, undrained, residual (post seismic) strength conditions and pseudo-static conditions.

12.6.4 Shallow failure surfaces were not considered. Shallow failures mean that the outer face of the facility fail. This would be a localised surface failure which does not affect the overall stability of the facility. These can be relatively quickly and economically repaired to prevent any long-term instabilities and are solely in the area of operational inspections and repair as and when necessary.

12.6.5 A phreatic surface was modelled through the TSF embankment with hydrostatic pore water pressures below the phreatic surface.

12.6.6 As shown in Table 11, all SCPTu probes were either dry or encountered sub-hydrostatic pressures. Therefore, no groundwater was considered below the liner system i.e. unsaturated with only the tailings behaving under undrained and post-seismic/post-liquefaction conditions.

12.6.7 The table below summarises the stability analyses results. Graphical slope stability outputs are included in Appendix D.

Table 13: TSF Slope stability Factors of Safety

Section	Drained FOS	Undrained FOS	Post seismic FOS	Pseudo – static FOS
Section A-A	2.3	2.0	1.7	1.6
Section D-D	2.0	2.0	1.8	1.3
Section E-E	2.1	1.6	1.2	1.4

12.6.8 The minimum Factor of Safety against failure is 2.0 under drained conditions, 1.6 under undrained conditions, 1.2 under post seismic, post liquefaction or residual conditions and 1.3 under pseudo static conditions. These Factors of Safety comply with the local regulation and international standards.

13. Existing facility slope stability

13.1 The Valley TSF butts up between the dormant FSN1 TSF on the west and FSN2 TSF on the east.

13.2 A geotechnical investigation (report reference: 2210540 – Harmony – FSN 1 and FSN 2 Stability Geotech – R02) and SCPTu testing was done on these two facilities. The stability of the facilities were assessed based on these results (report reference: 2210540 – Harmony – FSN 1 and FSN 2 SCPTu and Stabilities – R01).

13.3 Based on this assessment, using the limited equilibrium method of Stability Analysis, FSN1 and FSN2 do not comply with the stability requirements set out in the Valley TSF Design Criteria as indicated in table 4 of Paragraph 10.3.

13.4 To ensure the entire Valley TSF complex complies with the required Factors of Safety at closure, an alternate method of stability analysis will be carried out using 'Finite Element

Analysis' and based on the outcome of this additional stability analysis, remedial works for FSN1 and FSN2 may be incorporated into the Valley TSF operation and closure plan.

13.5 Three potential interventions were previously modelled to get the facilities to comply with the required standards, based on the Limited Equilibrium Stability Analysis. Results showed that a pushdown, with a step in and a tailings buttress, is required to improve the factors of safety of both.

14. Liner system design – TSF

14.1 Barrier system

14.1.1 An independent review of the liner system has been done by Legge and Associates. The review report recommended that an 'inverted barrier' system be used as opposed to a Class C barrier system. A comparison of these two barrier systems is shown in Figure 12 below.

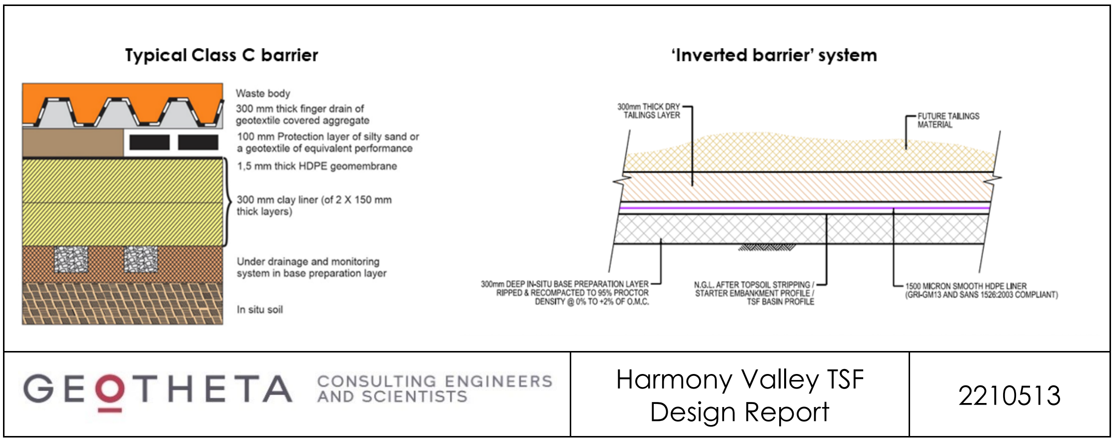

Figure 12: Comparison of typical Class C liner and inverted liner system

14.1.2 The inverted barrier system has superior performance as compared to the Class C barrier system in terms of reducing seepage, and equivalent performance in terms of service life considerations. This is a more feasible option as it removes the need for a compacted clay liner below the geomembrane. The stability of the TSF is also improved by omitting lower strength compacted clay layers and the geomembrane cushion layer (replaced by tailings).

14.1.3 The effectiveness of the proposed inverted liner system considers flow through the tailings due to the possible holes in the liner. Strict construction quality control is assumed therefore the liner system is assumed to have a maximum of 5 holes per hectare, with each hole being 10mm in diameter. When a hole forms in the liner, the fine tailings will clog it, therefore Darcy's law was applied to consider seepage through the holes. The seepage through a typical 1.5mm HDPE liner with no holes used in landfill applications is negligible (R. Kerry Rowe, 2012).

14.1.4 Refer to Appendix E (report reference: Alternative Barrier System Layout to a Class C single composite barrier_Inverted Barrier), for further details on the recommended barrier system by Legge and Associates.

14.1.5 The proposed TSF barrier system comprises of two areas as shown in Figure 13. The proposed TSF barrier system cross-sections are shown in Figures 14 and 15.

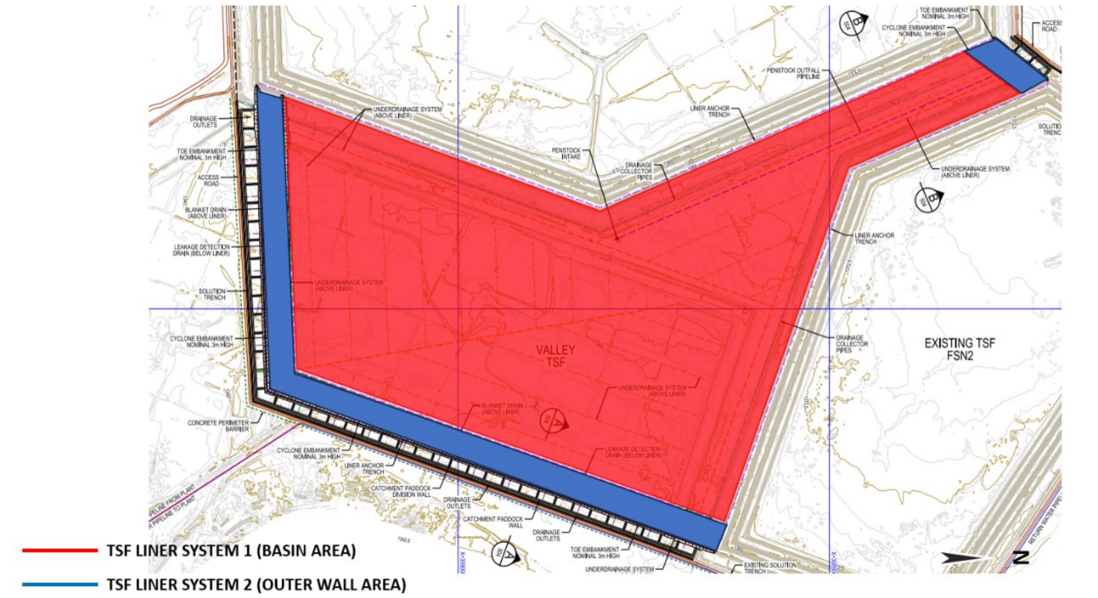

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Figure 13: Proposed liner areas on Valley TSF

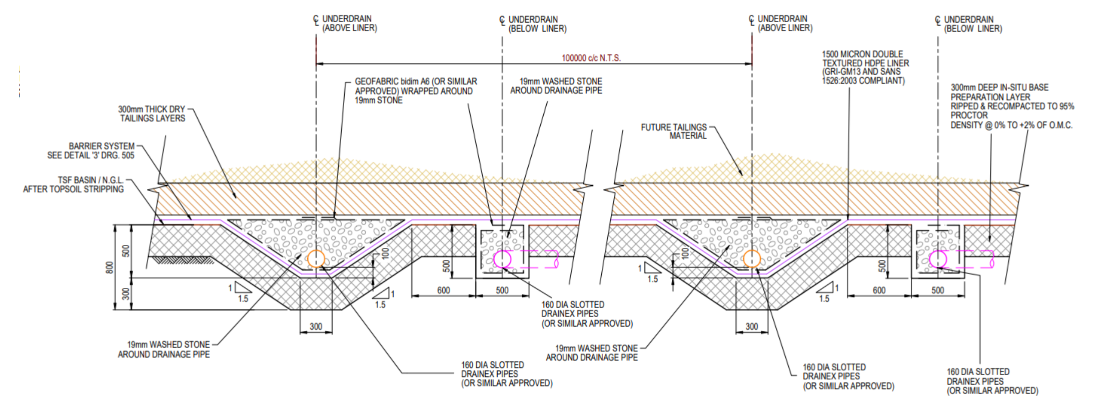

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Figure 14: TSF liner system 1 (basin area) cross section

14.1.6 The Valley TSF liner system 1 is shown in Figure 14 above and comprises the following layers (from top down):

- 300mm thick layer of tailings material. This is to be sourced from the FSN4's TSF footprint.
- Above liner drain comprising 160mm perforated HDPE pipes placed in a trapezoidal trench. The pipes will be encased in 19mm washed stone and wrapped in geofabric.
- 1.5mm thick smooth HDPE membrane (GRI-GM13 and SANS 1526:2003 compliant).

- Ripping and recompacting of the in-situ base material, 300mm to 95% Proctor density at a moisture content between 0% and +2% of optimum moisture content.
- Leakage detection system comprising 160mm perforated HDPE pipes placed in a 500mm by 500mm trench. The pipes will be encased in 19mm washed stone and wrapped in geofabric.

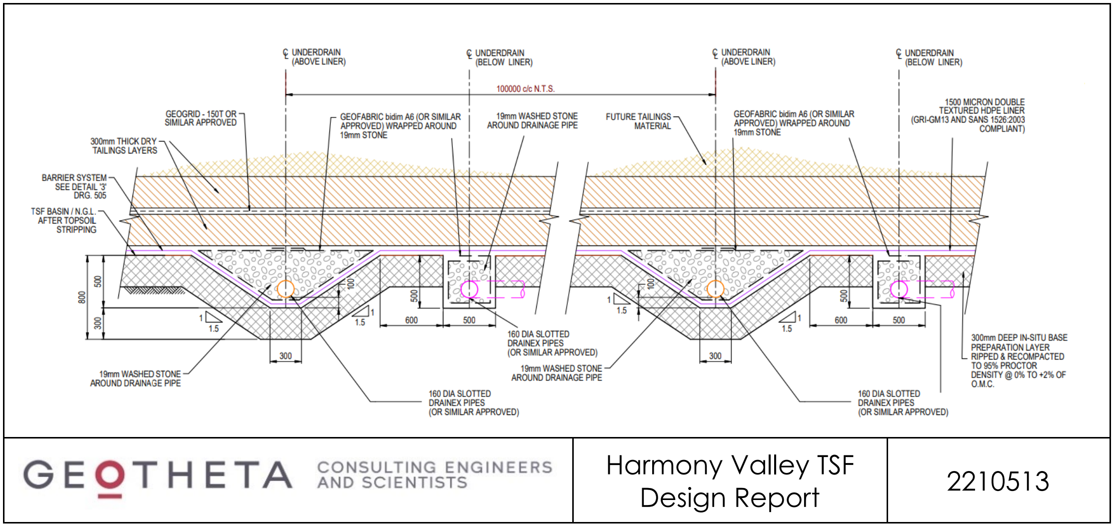

Figure 15: TSF liner system 2 (outer wall area) cross section

14.1.7 The Valley TSF liner system 2 is shown in Figure 15 above and comprises the following layers (from top down):

- 300mm thick layer of tailings material. This is to be sourced from the FSN4's TSF footprint.
- 150T polypropylene geogrid or similar approved. The 150T geogrid is to be placed 100m from the outer walls only.
- 300mm thick layer of tailings material.
- Above liner drain comprising 160mm perforated HDPE pipes placed in a trapezoidal trench. The pipes will be encased in 19mm washed stone and wrapped in geofabric.
- 1.5mm thick double textured HDPE membrane (GRI-GM13 and SANS 1526:2003 compliant).
- A 300mm in-situ base preparation layer that is ripped and recompacted to 95% Proctor density at a moisture content between 0% and +2% of optimum moisture content.
- Leakage detection system comprising 160mm perforated HDPE pipes placed in a 500mm by 500mm trench. The pipes will be encased in 19mm washed stone and wrapped in geofabric.

14.1.8 This flexible, high-strength polypropylene geogrid is used to reinforce the tailings layer over the liner. The polypropylene geogrid is made from high-modulus, low-creep synthetic materials enclosed in a protective polymer coating for protection from installation damage and short term ultraviolet exposure.

14.1.9 The maximum tensile strength of the polypropylene geogrid is up to 1600kN/m. (Refer to report 2210513 – Harmony – Valley TSF Design – CS – R04).

14.2 **Seepage assessment**

14.2.1 The seepage through the liner system was quantified to evaluate the effectiveness of the proposed liner system considering the flow through the tailings due to the possible holes in the liner.

14.2.2 The expected flow rate through the liner system was calculated using the Wissa and Fuleihan 1993 (W-F) equation. The reference document for the seepage calculation using the Wissa and Fuleihan 1993 (W-F) equation is titled Short and long-term leakage through composite liners – The 7th Arthur Casagrande Lecture – R. Kerry Rowe.

14.2.3 A geomembrane installed with good construction quality assurance (CQA) will have 2.5 – 5 holes per hectare (Giroud and Bonaparte 1989, 2001) and a typical hole diameter of approximately 10mm. The reference document for the number of holes per hectare and the typical hole diameter is titled *Leakage through Holes in Geomembranes below Saturated Tailings – R. Kerry Rowe*.

14.2.4 When a hole forms in the liner, the fine tailings will clog it, therefore the Wissa and Fuleihan 1993 (W-F) equation uses Darcy's Law to capture flow through the tailings contained within a 10mm diameter geomembrane hole. The seepage flow is calculated by the following equation.

$$Q = \frac{2khd}{1 + \frac{8}{\pi}\left(\frac{t}{d}\right)}$$

14.2.5 In the above equation, Q is the flow rate, k is the permeability of the tailings above the geomembrane, h is the head above the geomembrane, t is the geomembrane thickness and and d is the diameter of the hole in the geomembrane.

14.2.6 The calculated TSF seepage through the 'inverted barrier' is 18L/ha/day which is below the typical seepage rate for a Class C liner of 140L/ha/day. The seepage through the liner was used to size the below liner drains.

14.2.7 The seepage flow rate through the tailings layer is low. Negligible impact to the underlying soils and groundwater is therefore expected.

14.2.8 The underdrainage system will be monitored as part of the operations, maintenance, and surveillance plan to determine and quantify any leakage through the liner system.

14.3 **Underdrainage system**

14.3.1 A TSF underdrainage system is provided above the liner to intercept seepage through the facility. The underdrainage system lowers the phreatic surface, thereby improving the overall stability of the facility.

14.3.2 An above-liner drainage system is provided to intercept the seepage through the facility and reduce static water head on the liner.

14.3.3 The underdrainage system comprises of blanket drains and herringbone drainage pipes.

14.3.4 Herringbone drainage is also provided between the cyclone wall and the toe wall to ensure that the outer Cyclone underflow wall on the North-west, South and – South-Eastern sections of the TSF remains drained.

14.3.5 The blanket drains inside the basin area comprise of 160mm slotted Drainex HDPE pipes surrounded in 19mm stone overlain by a layer of 6mm stone and graded filter sand which is enclosed in a geofabric.

14.3.6 All drainage outlet pipes intersect the starter wall with an HDPE pipe boot at the point of intersection with the liner as shown in Detail 1 – Typical section HDPE pipe boot on Drawing 2210513-506. The outlet pipes are spaced 100m apart.

14.3.7 The herringbone drainage pipes comprise of 160mm slotted Drainex HDPE pipes surrounded in 19mm stone which is enclosed in a geofabric. The above-liner drains are spaced at 100m apart.

14.3.8 The south-eastern underdrainage outlet pipes discharge into the solution trench located at the south of the TSF. This solution trench conveys seepage water from the drains to the existing RWD situated to the south-west of the FSN1 TSF.

14.3.9 The western underdrainage outlet pipes discharge into the new RWD silt trap situated to the north-west of the TSF. The Valley TSF underdrainage system with flow direction is shown in Figure 16 below.

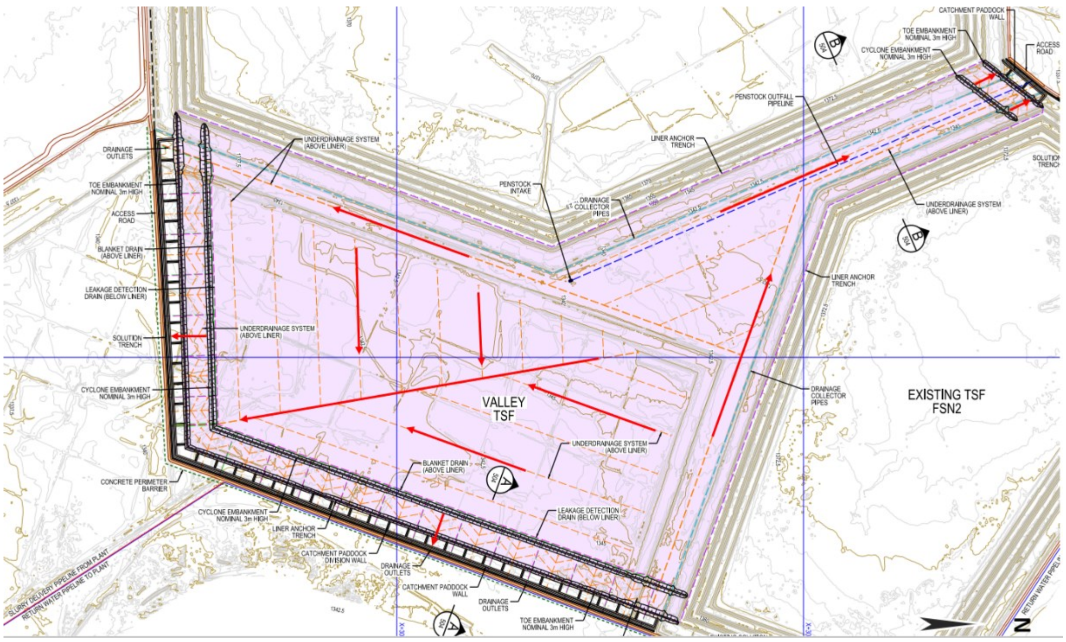

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Figure 16: TSF underdrainage system flow direction

14.3.10 Underdrainage collecting seepage from FSN1 and FSN2 along the toe of the TSF will comprise of a 160mm slotted Drainex HDPE pipe and a 160mm unslotted Drainex HDPE collector pipe, joined by a 160mm double socket Y junction (45 degree). The spacing between the Y junctions are 100mm. The 160mm pipes will be surrounded by 19mm stone overlain by a layer of 150mm washed river sand which is enclosed in a geofabric. These collector drainage pipes discharge into the new RWD via the solution trench on the North.

14.4 Leakage detection system

14.4.1 An underdrainage leak detection system will be monitored as part of the operations, maintenance, and surveillance plan to determine and quantify any leakage through the liner system.

14.4.2 The leakage detection system also alleviates any possible water pressure build-up beneath the liner from a potential rise of the groundwater table. The leakage detection system was designed to cater for the flow through holes in the HDPE liner.

14.4.3 In the event of a leak occurring, the drains serve to locate the area of the leak. Once the area of the leak is located, monitoring of the area and maintenance of the phreatic level is required or further action will need to be taken.

14.4.4 The leakage detection drain comprises a trapezoidal shaped trench with a 160mm slotted HDPE pipe surrounded in 19mm stone which is enclosed in a geofabric.

14.4.5 The leakage detection outlet pipes discharge into the solution trench. All drain outlets will be clearly marked to distinguish between the underdrains, blanket drains and leakage detection drains.

14.4.6 The TSF underdrainage details are shown in Drawing No. 2210513 – 506.

15. Liner tension forces – TSF

15.1 The slope stability analyses were used to determine the maximum shear stresses in the polypropylene geogrid.

15.2 The maximum shear stresses (and forces) have been analysed against the yield strength of the polypropylene geogrid to determine the Factor of Safety against yield (failure) of the polypropylene geogrid under drained, undrained, post seismic conditions and pseudo-static conditions.

15.3 A 150T geogrid will be installed to reduce the stresses in the liner to a Factor of Safety of 1.5. The results from the analysis are shown in Table 14.

Table 14: Forces in HDPE liner

Description	Drained conditions	Undrained conditions	Residual conditions	Pseudo-static conditions
Max shear stress along liner surface (kPa)	28	81	96	72
Max shear force per m width (kN/m)	28	81	96	72
Yield strength of Geogrid (kN/m)	150	150	150	150
FOS against Geogrid yield strength	5.4	1.9	1.6	2.1

15.4 The minimum FOS against yield (failure) of the polypropylene geogrid is 5.4 for drained conditions, 1.9 for undrained conditions, 1.6 for residual conditions and 2.1 for pseudo-static conditions. These Factor of Safety are satisfactory.

15.5 The liner forces were also analysed at the liner anchor trenches. Shear stresses develop at the interface between the anchor trench surface and the liner. The detailed calculations are included in Appendix F.

15.6 The results of the calculations show that the Factor of Safety at the TSF anchor trench is 1.5, which is adequate.

15.7 The total tensile strain in the geomembrane is less than 1%. This is due to minimal movement expected because of the engineered base, reinforcing Geogrid, and cushion tailings protection layer in the inverted barrier system.

16. Liner service life assessment

16.1 The service life of a geomembrane is affected by various factors including UV exposure, temperature conditions and applied loading.

16.2 The deposition life of the Valley TSF is estimated at 8 years, after which, pending future reclamation, it may exist as a dormant TSF for a very long time.

16.3 The main factors affecting the service life of a geomembrane is UV exposure and temperature. The geomembrane on the TSF will be covered during construction by graded filter sand, therefore UV exposure will not have a detrimental effect on the service life of the geomembrane.

16.4 The average minimum and maximum ambient temperature of the site is 10°C and 25°C respectively. Based on research conducted by the Geosynthetics Institute in USA "Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions" originally published in 2005 and later updated in 2011, it is reported that an unexposed geomembrane at 25°C will have a design life of more than 250 years.

16.5 It is noted that this 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.

16.6 Rowe (2005) discussed the effects of temperature on a geomembrane's service life. In the manufacturing process of geomembranes, antioxidants are added to the material to act as the sacrificial component in terms of oxidation. This means that for a certain time period the antioxidants prevent the geomembrane from being oxidised, which results in increasing the material's durability and service life.

16.7 The time required to deplete the antioxidants in the geomembrane depended on its exposure rate. The table below details the three stages of degradation and the service life of 1.5mm thick double textured HDPE geomembrane that meets the GRI GM13 specification.

16.8 The projected service life of a geomembrane is 2 775 years at 10°C and decreases to 608 years at a temperature of 25°C.

Table 15: Estimated times for degradation and service life Rowe, 2005)

(1) Temp: °C	(2) Stage 1: years Simulated	(3) Stage 2: years Base	(4) Stage 2: years Adjusted	(5) Stage 3: years Base	(6) Stage 3: years Adjusted	(7) Service life: years Unadjusted	(8) Service life: years Adjusted
10	280	50	30	2445	1380	2775	1690
20	115	15	10	765	440	900	565
30	50	6	4	260	150	315	205
35	35	4	2	155	90	190	130
40	25	2	1	95	55	120	80
50	10	1	0.6	35	20	50	35
60	6	0.4	0.3	15	9	20	15

17. Dam Break Analysis

17.1 Geotheta completed a feasibility dam break analysis for the Harmony Gold Valley TSF using FLO-2D Overland Flood Modelling (report reference: 2210513 - Harmony – Valley TSF Design - DBA - R02).

17.2 The following guidelines were used for the feasibility modelling and analysis to determine the GISTM Consequence Classification:

- Canadian Dam Association: Tailings Dam Breach Analysis (2021).
- ICOLD. Committee L Tailings Dams and Waste Lagoons. Technical Guidelines for Tailings Dam Safety Assessment and Design (2019).

17.3 Contours of the inundation area were used to create a digital terrain model. These are accurate at 5m intervals which is considered sufficiently accurate for the purposes of establishing an indicative inundation zone. Given the range of possible failure scenarios, failure volumes and surface flow resistances that can occur, the contour intervals are therefore adequate for the purpose of this feasibility level study.

17.4 The proposed Valley TSF is a **Very High Consequence Classification** facility according to the Global Industry Standard on Tailings Management (GISTM) criteria. This is determined by analysing the impact a failure would have on life, the environment and infrastructure in the modelled inundation zone. The corresponding SANS 10286 hazard classification is **High**. The Zone of Influence is shown below.

17.5 The image below indicates the overall zone of influence and the delineated background flood.

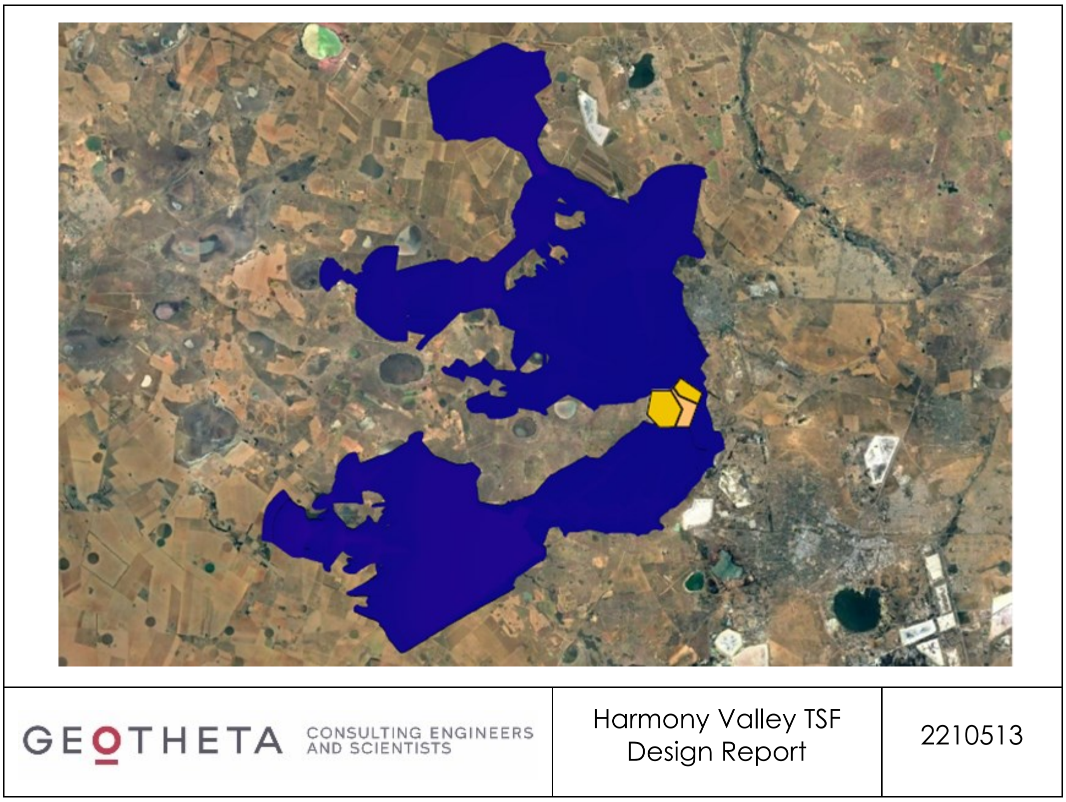

Figure 17: Overall zone of influence and the delineated background flood

17.6 The analyses concluded that there would be extensive damage to both the natural environment and infrastructure within the inundation area.

17.7 Tailings flowing into the river South of the facility will result in the loss of aquatic wildlife and decrease in water quality. It is likely that the pollution of the river and loss of aquatic wildlife would have adverse impacts on the ecosystem of the area and adversely affect users of the water.

17.8 The flood event would inundate households and associated infrastructure located near the facility and the populated area to the northmeast of the Valley TSF. The potential population at risk falls between 100 – 1 000, with the potential loss of life not exceeding 10. The inundated area must be environmentally surveyed to identify the affected population, environment and infrastructure within the Zone of Influence.

17.9 A rainy day tailings flow can be diverted away from the nearby residential area by constructing a dump rock bund approximately 1m high at the edge of the residential area. This can be designed to decrease the probability of loss of life to ranges of 1:10 000 or better. The bund must be designed to withstand flow erosion.

17.10 This is the dam break analysis of the Valley TSF in isolation. An extensive dam break analysis can be done, as necessary, on the concurrent failure of FSN1, FSN2, Valley and the future Nooitgedacht TSFs to fully determine the extent of the impacted zone.

17.11 Note that this is the facility's "Consequence Classification", and it is not at all linked to the likelihood of failure. The consequence classification leads to the design criteria to be used to ensure that the facility is adequately designed, operated, managed and closed so that the risk of failure is reduced to as low as reasonable practicable. The dam break analyses merely address the consequence should the facility fail.

17.12 The likelihood of failure would be addressed from further Failure Mode Effects Analyses, specifications, QA/QC aspects, etc. which does not fall part of this scope and is addressed separately as part of the design.

18. SANS 10286 TSF Classifications

18.1 TSF safety classification

18.1.1 The SANS 10286 Code of Practice for Mine Residue, requires that all mine residue deposits be classified into one or a combination of the following safety categories:

- High hazard
- Medium hazard
- Low hazard

18.1.2 The safety classification of the Valley TSF was determined by analysing the zone of influence and applying the safety classification criteria provided in the SANS 10286 Code of Practice for Mine Residue. The safety classification criteria are indicated in Table 16.

Table 16: Safety classification criteria

No of residents in zone of influence	No of workers in zone of influence[1]	Value of third party property in zone of influence[2]	Depth to underground mined workings[3]	Classification
0	<10	0-R2 m	>200 m	Low hazard
1-10	11-100	R2 m-R20 m	50 m-200 m	Medium hazard
>10	>100	>R20 m	<50 m	High hazard

1) Not including workers employed solely for the purposes of operating the deposit
2) The value of third party property should be the replacement value in 1996 terms
3) The potential for collapse of the deposit into the underground workings effectively extends the zone of influence to below ground level.

18.1.3 Based on SANS 10286, the Valley TSF has a High hazard classification rating.

18.2 TSF environmental classification

18.2.1 Table 17 below, outlines how the environmental classification is determined using SANS 10286.

18.2.2 The environmental classification of the TSF is a residue deposit with a significant impact on any environmental component.

Table 17: Environmental classification criteria

Aspect under consideration	Environmental classification		
	Significant	**Possibly significant**	**Not significant**
Surface and groundwater	Deposit has potential to contaminate water that may be consumed by humans.	Deposit has potential to contaminate water that may be consumed by flora or fauna.	No contamination of water supplies likely.
Land	Deposit has potential to permanently render surrounding land unsuitable for its pre-existing potential.	Release of residue from the deposit could have a long-term detrimental effect on land.	Release of residue from the deposit can be completely remediated.
Air	Deposit has potential to degrade air quality to a level that is detrimental to human health.	Deposit has potential to elevate dust nuisance (only) to an unacceptable level.	Deposit has negligible potential to adversely affect air quality.
Physical security	Residue has potential to cause injury on release as a result of structural failure. [1]	Residue has potential to cause injury as a result of structural failure [2]	Residue has negligible potential to cause harm through structural failure.
Business environment	Failure of Deposit has potential to result in business failure of operation.	Failure of Deposit has potential to result in significant economic loss.	Low potential for failure of Deposit to result in economic loss.
Social environment	Failure of Deposit could lead to severe adverse publicity, resulting in business failure and impairment of credibility.	Failure of Deposit could lead to adverse publicity, leading to regulatory intervention and/or financial loss.	Failure of Deposit is unlikely to lead to adverse publicity or indirect losses.
Government	Failure of deposits can lead to Harmony receiving directives/penalties.	Possibility of notice	None

19. GISTM TSF Classification

Please note that GISTM is not a local regulation requirement but is reported here for Harmony Gold's internal requirements only.

19.1 The GISTM, requires that all TSFs be classified into one of the following consequence classifications:

- Low
- Significant
- High
- Very High
- Extreme

19.2 The consequence classification of the Valley TSF was determined by analysing the zone of influence and applying the consequence classification criteria provided in Table 1.

19.3 The Valley TSF is categorised as a **Very High Consequence Classification** facility due to the impact a failure of this facility would have on the life, environment and infrastructure in the inundation zone modelled during the dam break analysis. Refer to Section 8 for the consequence classification table.

20. TSF Dam legal classification

20.1 Regulation 139 of the of the National Water Act and South African National Committee on Large Dams (SANCOLD) regulations stipulates that a dam storing more than 50 000m^3 **and** having an outer wall height of more than 5m should be registered as a dam with a safety risk.

20.2 A dam with a safety risk requires more stringent monitoring, inspection and controls by appropriately qualified persons registered in terms of the Act.

20.3 The Dam Safety Office originally only considered the amount of water stored on the facility when classifying dams with a safety risk. In this case the expected pool volume is less than the required 50 000m^3.

20.4 However, since the volume of flowable tailings in the event of a dam break is 847 215m^3, it is recommended to request an assessment by the Dam Safety Office of whether the facility should be categorised as a dam with a safety risk.

21. New Return Water Dam design and existing RWD Capacity assessment

The New RWD will be constructed within the existing RWD situated north-west of the Valley TSF. This existing RWD needs to be drained before construction of the new RWD. Other preparatory works, to be assessed on site, may be required.

An assessment was done to determine if the capacity of 304 000m^3 of the existing Return Water Dam situated south-west of Valley TSF is sufficient to contain drain flow from Valley TSF.

21.1 Layout

21.1.1 The layout of the new Return Water Dam (RWD) is shown in Figure 18 below.

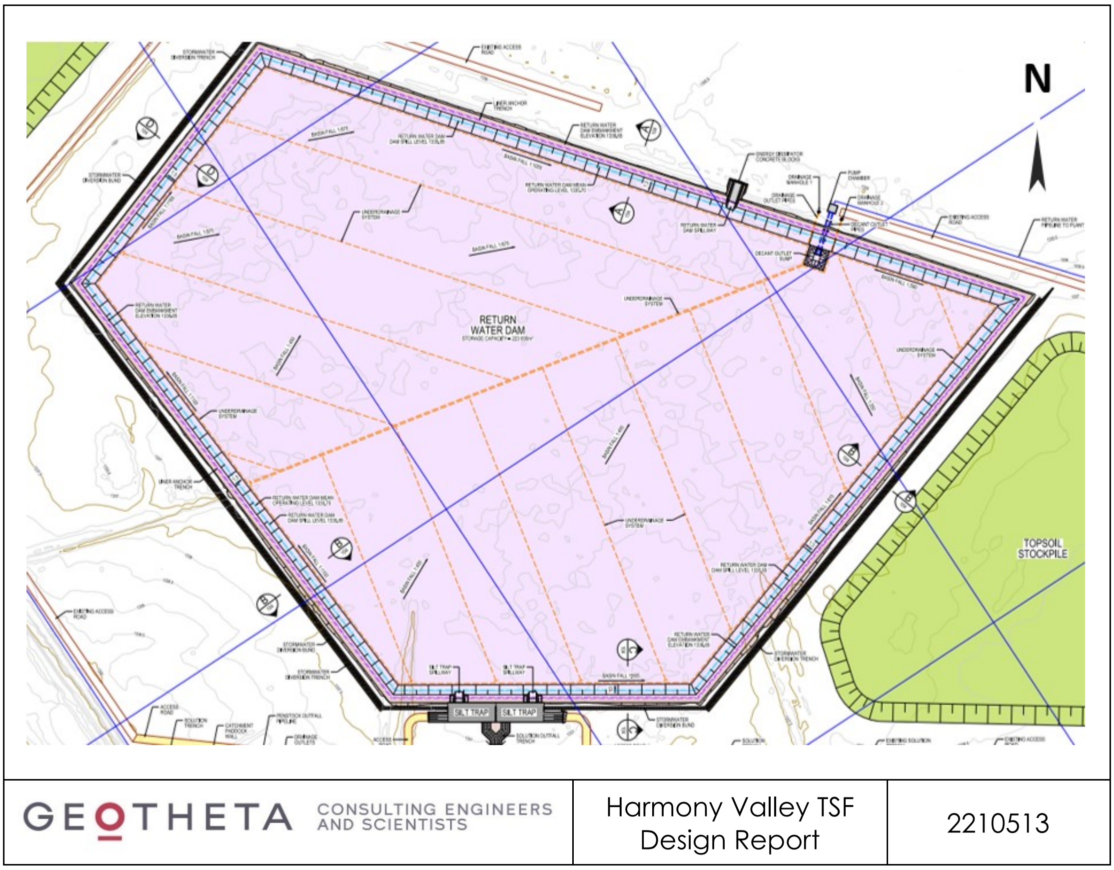

Figure 18: New RWD layout

21.1.2 The lined RWD is situated north-west of the TSF.

21.1.3 The basin of the RWD is formed by excavation. The new RWD has a total storage capacity of 220 000m^3. This provides adequate capacity to contain runoff from the TSF catchment area.

21.1.4 The unlined existing RWD situated south-west of the Valley TSF is shown in Figure 19 below.

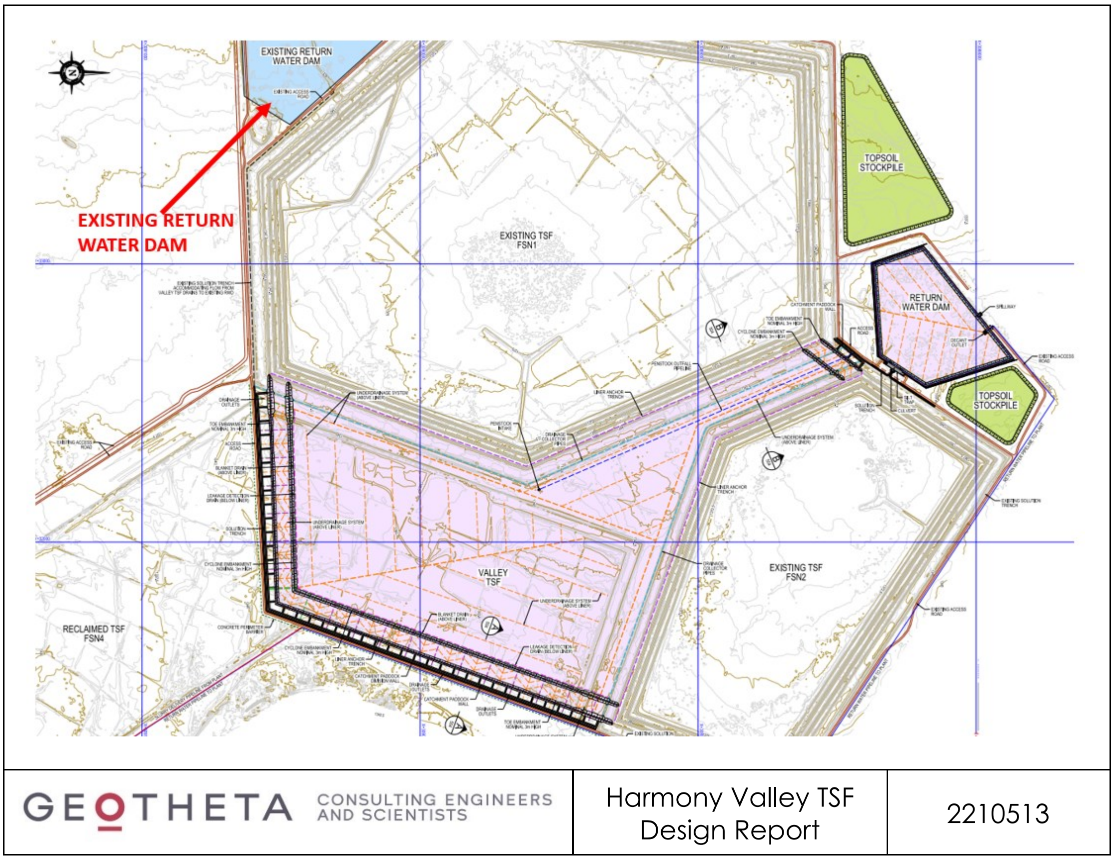

Figure 19: Existing RWD location

21.2 **New RWD sizing**

21.2.1 A daily stochastic water balance of the TSF and associated infrastructure was modelled to determine the required capacity of the New RWD.

21.2.2 The average monthly water balance model of the New RWD is shown in Figure 20.

21.2.3 Storage and spillage have been considered at each daily time step to determine the frequency of spillage as required by Government Notice 704 of the National Water Act (Act No 36 of 1998).

21.2.4 The water balance model comprises three main components (the processing plant, the RWD and the TSF). Each component has various inputs and outputs.

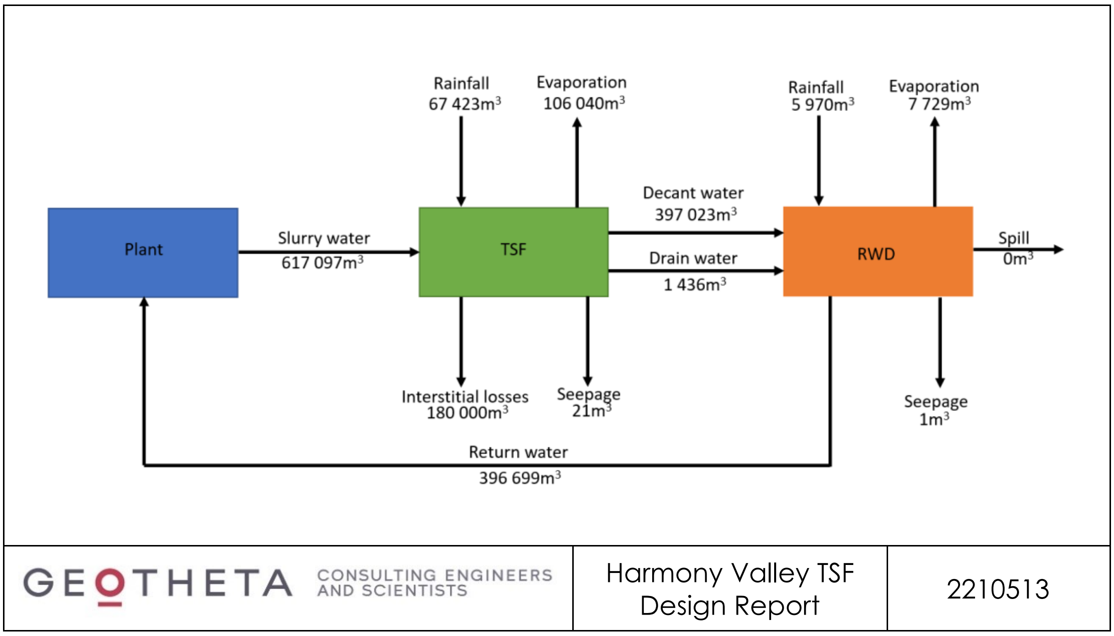

Figure 20: Water balance model (New RWD) – monthly averages

21.2.5 The daily water balance model has been developed by utilising a continuity equation (inflow – outflow = Δ storage) for each component. Storage capacity in the plant is not considered.

21.2.6 TSF inputs are as follows:

- Direct rainfall onto the TSF.
- Slurry water from the plant.

21.2.7 TSF outputs are as follows:

- Evaporation from the TSF after a rainfall event.
- Seepage from the leak detection drains through the liner.
- Under drain flow to the new RWD and the existing RWD.
- Interstitial losses within the gold tailings material.
- Decant water to the new RWD.

21.2.8 RWD inputs are as follows:

- Direct rainfall into the RWD basin.
- Decant water from the TSF.
- Under drain flow from the TSF.

21.2.9 RWD outputs are as follows:

- Evaporation from the RWD basins
- Seepage into the underlying in-situ soils.
- Spill into the natural environment when the RWDs capacities are exceeded.
- Return water which is pumped from the RWDs to the plant for re-use.

21.2.10 The daily rainfall data needed for the RWD sizing was extracted from the Olivine Station rain gauge (SAWS station No. 0328726 W). This station was selected due to its long record length, completeness of the data set, mean annual precipitation (MAP) and location of the rainfall station with respect to the site.

21.2.11 The Olivine Station rain gauge has the longest, most reliable record for the stations in the vicinity of the site area. Rainfall data collected from Olivine Station started on 1 January 1903 and ended on 30 December 2018 (115 years).

21.2.12 The daily rainfall data is indicated in Figure 21 below. The daily rainfall data has been extrapolated to increase the water balance simulation period.

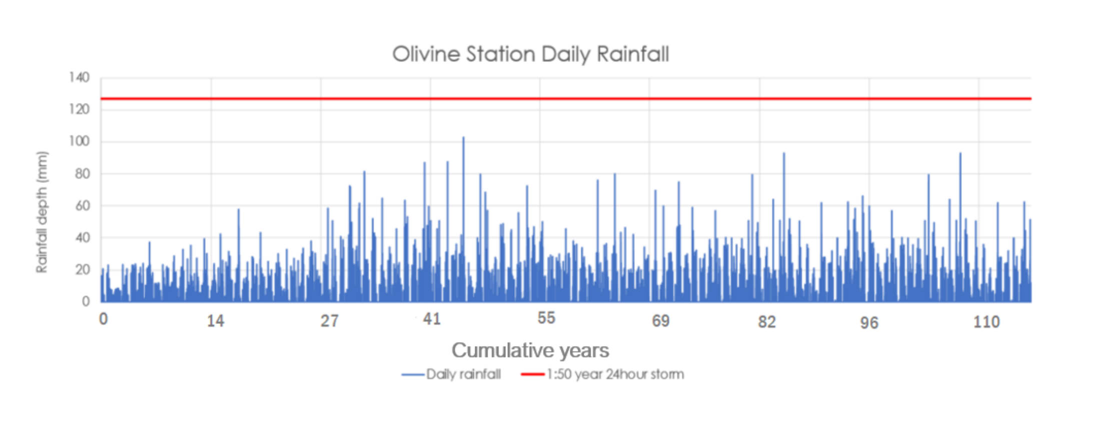

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Figure 21: Daily rainfall record for Olivine (0328726 W) rainfall station

21.2.13 The graphical plot showing the new RWD volume with time is indicted in Figure 22. The graph shows no spill events over a total simulation period of 110 years. This complies with the requirements to not spill more than once every 50 years.

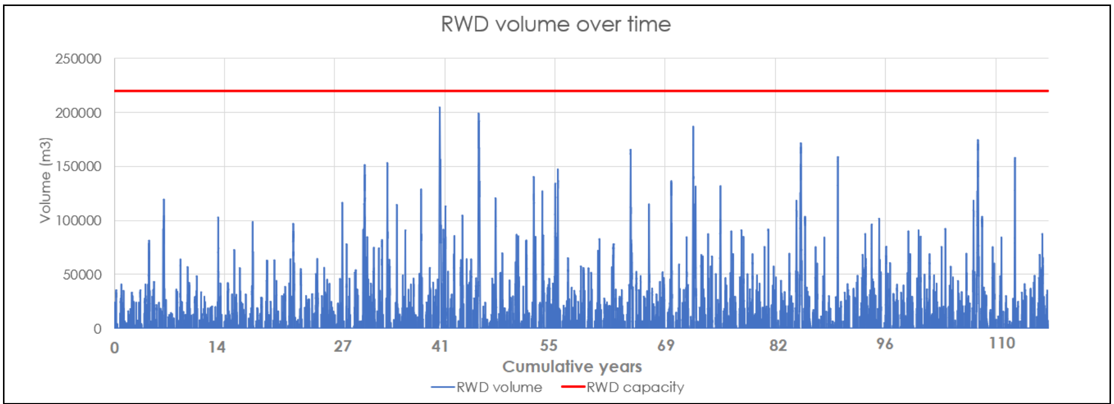

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Figure 22: RWD volume over time

21.2.14 The stochastic water balance analysis indicates that the required RWD storage capacity is 220 000 m^3. This ensures that the RWD does not spill more than once every 50 years as required by Government Notice 704 of the National Water Act (Act No 36 of 1998).

21.2.15 The RWD mean operating volume is 39 880m³. The operating volume was calculated by taking the volume of the third highest spill that is most likely to happen (180 120m³), subtracted from the total storage capacity of the RWD (220 000m³). The operating depth is 0.276m. This is calculated by dividing the operating volume by the total area of the RWD. The depth of 0.276m is recommended during the heavy rainfall seasons. This can be raised during the drier months.

21.3 Existing RWD capacity assessment

21.3.1 A stochastic water balance analysis was done using the storage capacity of the existing RWD (304 000m³) to determine if this is adequate to accommodate drain flow from Valley TSF. It was made certain that the RWD does not spill more than once every 50 years as required by Government Notice 704 of the National Water Act (Act No 36 of 1998).

21.3.2 The graphical plot showing the existing RWD volume with time is indicted in Figure 23. The graph shows no spill events over a total simulation period of 110 years. This complies with the requirements to not spill more than once every 50 years.

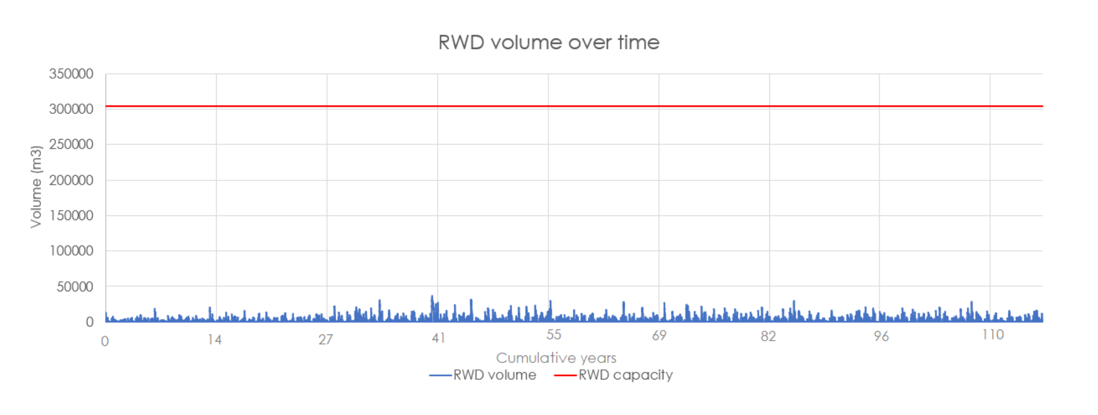

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Figure 23: RWD volume over time

21.3.3 The existing RWD operating volume is 25 000m³. The operating depth is approximately 0.10m.

21.3.4 The results of the capacity assessment indicate that the capacity of 304 000m³ of the existing RWD is sufficient to contain drain flow from Valley TSF.

21.4 New RWD water balance volumes

21.4.1 The required capacity to pump water from the new RWD back to the operations is 16 440m³/day.

21.4.2 The average monthly water balance volumes are shown in Table 19.

Table 19: Monthly water balance volumes

Facility Name	Inflow		Outflow		Comment
TSF	Rainfall	67 423m³	Evaporation	106 040m³	
	Slurry water	617 097m³	Drain water	1 436m³	
			Seepage	21m³	
			Interstitial losses	180 000m³	
			Decant water	397 023m³	
	Total	**684 520m³**	**Total**	**684 520m³**	**Adequate**
RWD	Rainfall	5 970m³	Return water	396 699m³	
	Decant water	397 023m³	Evaporation	7 729m3	
	Drain water	1 436m³	Seepage	1m3	
			Spill	0m³	
	Total	**404 429m³**	**Total**	**404 429m³**	**Adequate**

21.5 **RWD dam safety risk classification**

21.5.1 The total design capacity of the new RWD is 220 000m³ with a maximum above-ground wall height of 2.0m.

21.5.2 In terms of Regulation 139 of the of the National Water Act, for a dam to classify as a dam with a safety risk, the storage capacity must be greater than 50 000m³ **and** the maximum wall height must be greater than 5m.

21.5.3 Although the storage capacity is greater than 50 000m³, the maximum above-ground wall height (2.0m) is less than what is required for a dam to be classified as a dam with a safety risk (5.0m).

21.5.4 The new RWD is not classified as a dam with a safety risk in terms of Regulation 139 of the National Water Act. The requirements for a dam with a safety risk as indicated in Regulation 139 of the National Water Act do not apply.

21.6 **Return Water Dam spillway design**

21.6.1 The new RWD spillway is designed to accommodate the expected probable maximum flood (PMF), i.e. the 1:10 000 year 24-hour storm event, without overtopping of the RWD embankment.

21.6.2 Determination of rainfall depths beyond a 1:200 year return period was done by logarithmic extrapolation of the available rainfall data.

21.6.3 The RWD spillway was sized to have adequate capacity to safely discharge the PMF. The 1:10 000 year 24-hour storm event was calculated at 240mm. Determination of the PMF rainfall depth is indicated in Figure 4.

21.6.4 A concrete lined spillway is provided to safely discharge excess water without overtopping of the RWD embankment walls. The RWD spillway has a freeboard of 800mm and has been designed to safely discharge the 1:10 000 24-hour PMF volume of 9.9m^3/sec over a 12-hour period. The RWD spillway details are shown in Drawing No. 2210513 – 605.

21.7 **Monitoring requirements**

21.7.1 The stormwater runoff water quality is to be monitored by the Mine's environmental consultants as and when required.

21.7.2 Drain water discharging from the Valley TSF and RWD underdrainage outlet pipes are to be monitored by the Mine's environmental staff/consultants, at most annually.

21.8 **Silt trap**

21.8.1 A silt trap is provided upstream of the new RWD. The silt trap will include infrastructure for cleaning. The silt trap ensures that solids are captured before entering the RWD, thereby minimising sedimentation in the RWD. The silt trap details are shown in Drawing No. 2210513 -601.

21.8.2 The silt trap comprises a 2.0m deep reinforced concrete water retaining structure. An access ramp is provided to allow for a TLB (or similar) to clean out the silt trap when required. Used rail sections will be cast into the floor of the silt trap to prevent damage to the concrete surface during cleaning.

21.9 **RWD slope stability assessment**

One cross section was analysed for empty and full scenarios of the new RWD. The cross-section location is shown in Figure 24. This cross section of the new RWD represents the critical scenario in terms of stability, which is the cross section with the maximum height on the RWD.

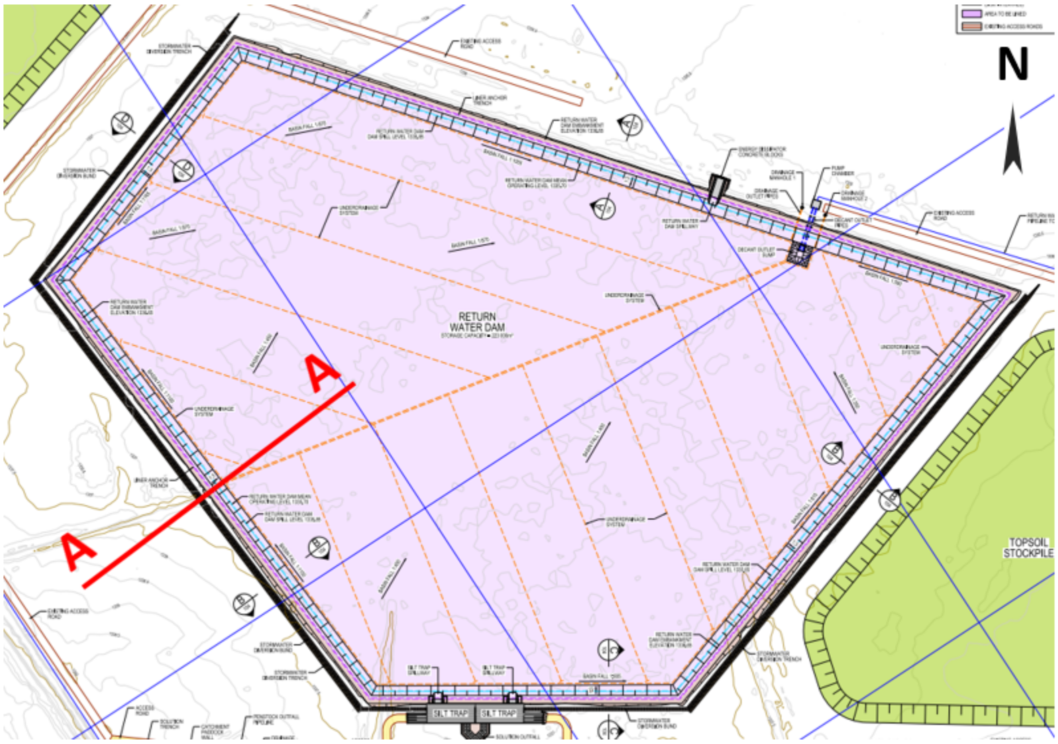

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Figure 24: Critical cross section location

21.9.1 The slope stabilities were analysed using RocScience Slide 2 slope stability software using the Cuckoo search method.

21.9.2 Table 20 summarises the stability analyses results. Graphical slope stability outputs are included in Appendix D.

Table 20: Slope stability Factors of Safety

	Drained conditions	**Pseudo – static conditions**
RWD outer slope - Empty	2.7	1.9
RWD outer slope – Full	3.5	2.3
RWD inner slope - Empty	2.0	1.4
RWD inner slope – Full	2.9	1.5

21.9.3 The minimum Factor of Safety against failure is 2.0 for static conditions and 1.4 for pseudo static conditions. These Factors of Safety comply with the local regulation and international slope stability standards.

22. Liner system design - RWD

22.1 Barrier system

22.1.1 The liner system (from top down) comprises:

- 200mm high perforated HDPE Geocells filed with 20Mpa concrete.

- 1.5mm thick smooth HDPE membrane (GRI-GM13 and SANS 1526:2003 compliant).
- 300mm of base in-situ preparation layer.
- Ripping and recompacting of the in-situ base material to 95% Proctor density at a moisture content between 0% and +2% of optimum moisture content.
- Underdrainage system comprising 160mm perforated HDPE pipes placed in a 300mm by 300mm trench. The pipes will be encased in 19mm washed stone and wrapped in geofabric.

22.1.2 The RWD liner system is shown in Figure 25. The liner system comprises the following layers (from top down):

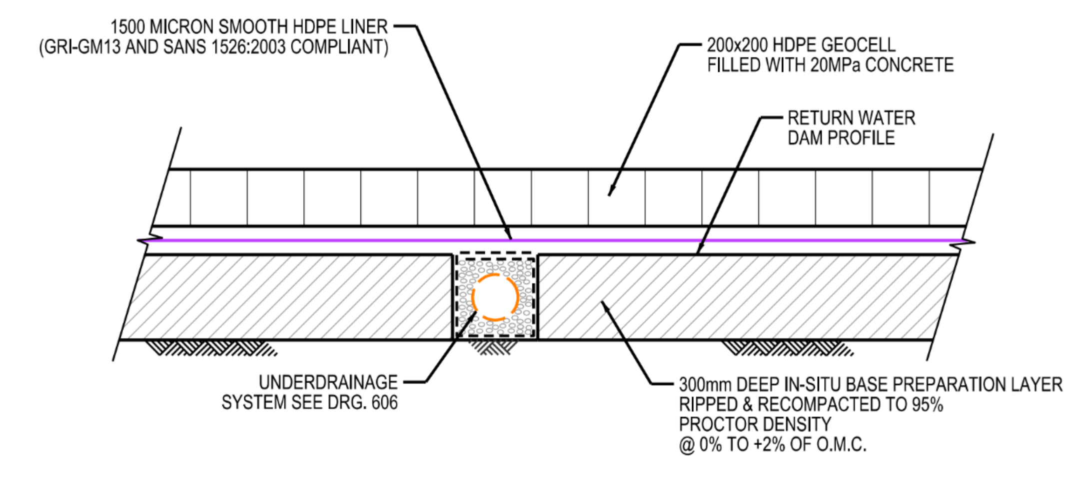

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Figure 25: RWD liner system

22.1.3 The use of a geomembrane requires a protection layer to achieve intimate contact between the liner and the underlying layer to ensure overall liner functionality.

22.1.4 The protection layer also provides durable protection to the liner against UV degradation and (possible) equipment and machine damage.

22.1.5 The protection layer will be 200mm high concrete filled perforated geocells. These will provide superior and long life protection compared to other options evaluated.

22.1.6 The geocells are perforated to prevent the blocks from acting independently and, if trafficked by maintenance or cleaning equipment, puncturing through the HDPE geomembrane. A typical image of the HDPE geocell is shown in Figure 26 below.

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Figure 26: Typical HDPE geocell

22.2 **Seepage assessment**

22.2.1 The seepage through the liner system was quantified to evaluate the effectiveness of the proposed liner system.

22.2.2 The expected flow rate through the liner system was calculated using the Wissa and Fuleihan 1993 (W-F) equation. This equation uses Darcy's Law to capture flow through the concrete filled geocells taking into consideration the discontinuity of the geocell. The seepage flow is calculated by the following equation.

$$Q = \frac{2khd}{1 + \frac{8}{\pi}\left(\frac{t}{d}\right)}$$

22.2.3 In the above equation, *Q* is the flow rate, *k* is the permeability of the concrete filled geocells above the geomembrane, *h* is the head above the geomembrane, *t* is the geomembrane thickness and and *d* is the diameter of the hole in the geomembrane.

22.2.4 The calculated RWD seepage through an 'inverted barrier' is 9.3 x 10-5 L/ha/day. The seepage through the liner was used to size the below liner drains.

22.3 **Underdrainage system**

22.3.1 An underdrainage system is provided beneath the new RWD basin area. The underdrainage system acts as a leakage detection system to detect any leaks through the liner system. The underdrainage system also alleviates any possible water pressure build-up beneath the liner caused by a potential rise of the groundwater table. The new RWD underdrainage details are shown in Drawing No. 2210513 – 606. The new RWD underdrainage layout is shown in Figure 27.

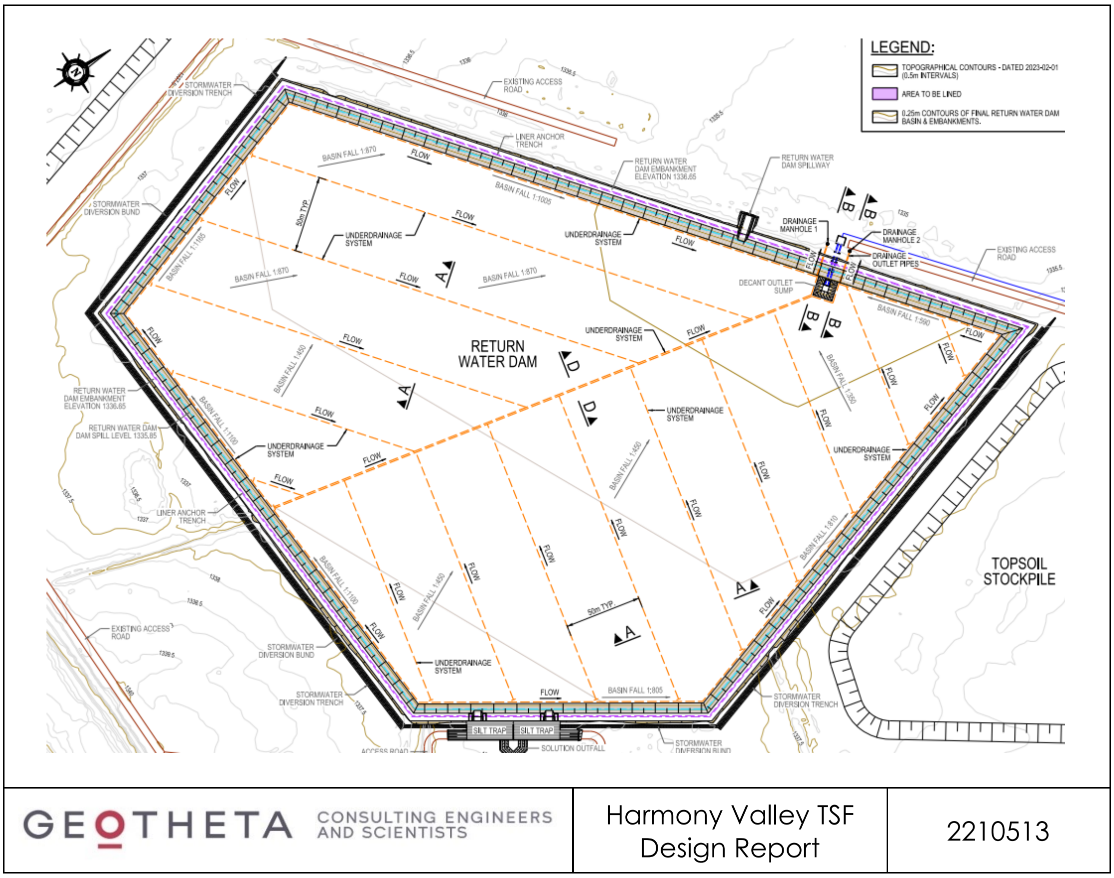


Figure 27: New RWD underdrainage layout

22.3.2 The underdrains comprise 160mm slotted HDPE pipes encased in 19mm washed stone. The stone will be wrapped in geofabric to prevent fines from entering the drains.

22.3.3 The underdrains from the RWD basin lead to collection manholes located on the perimeter of the RWD. The manholes provide access to monitor under-liner seepage.

22.3.4 The underdrainage system will be monitored as part of the operations, maintenance, and surveillance plan to determine and quantify any leakage through the liner system.

22.3.5 Water extraction from the RWD will be by means of a decant outlet sump and pump chamber. The RWD decant structure and details are shown on Drawing No. 2210513 – 607.

23. Liner tension forces – New Return Water Dam

23.1 The slope stability models were used to determine the maximum shear stresses in the 1.5mm thick smooth HDPE liner.

23.2 The maximum shear stresses (and forces) have been analysed against the yield strength of the 1.5mm thick smooth HDPE liner to determine the Factor of Safety against yield (failure) of the HDPE liner.

23.3 The results from the analysis are shown in Table 21.

Table 21: Forces in HDPE liner at the new RWD

Description	**RWD full – Static**	**RWD full – Seismic**	**RWD empty – Static**	**RWD empty – Seismic**
Max shear stress along liner surface (kPa)	6.7	14.2	6.3	7.8
Max shear force per m width (kN/m)	6.7	14.2	6.3	7.8
Yield strength of 1.5mm HDPE Liner (kN/m)	22	22	22	22
FOS against yield	3.3	1.5	3.5	2.8

23.4 The minimum FOS against yield (failure) of the 1.5mm thick smooth HDPE liner is 1.5. See Table 21 above.

23.5 The results show that the liner stresses were not exceeded.

23.6 The liner forces were also analysed at the liner anchor trenches. Shear stresses develop at the interface between the anchor trench surface and the liner. The detailed calculations are included in Appendix F.

23.7 The results of the calculations show that the Factor of Safety at the new RWD anchor trench is 1.5 which is adequate. This indicates that the liner can withstand the shear stresses developed at the anchor trenches.

23.8 The total tensile strain in the geomembrane is less than 1%. This is due to minimal movement expected because of the engineered base and concrete filled geocell protection layer in the inverted barrier system.

24. Liner service life assessment

24.1 Refer to Section 16 for Liner service life assessment.

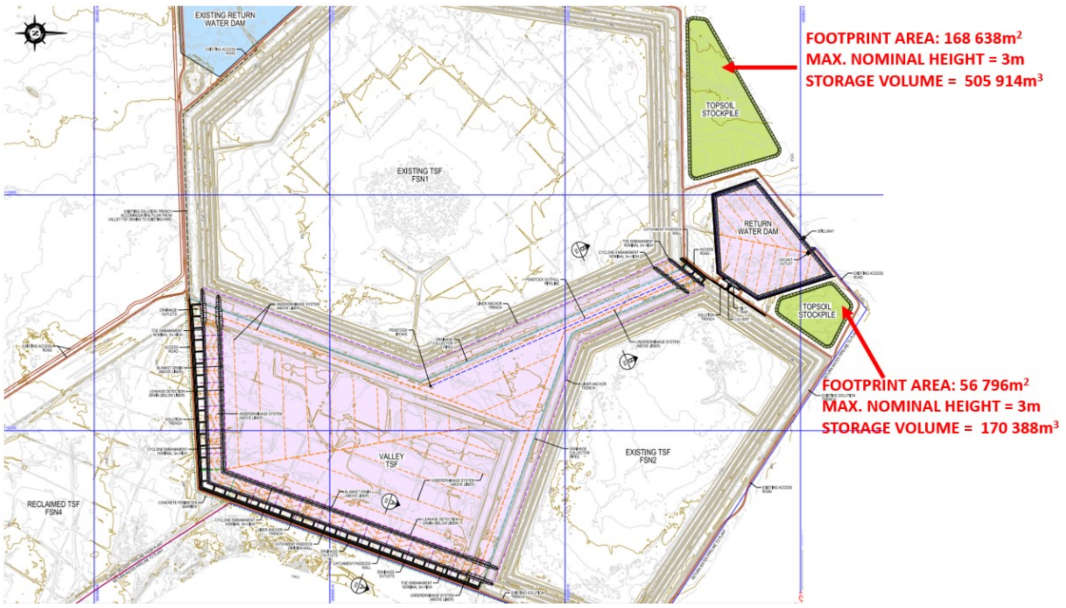
25. TSF engineering features

This section outlines the pertinent features of the TSF.

25.1 Topsoil stripping and stockpiling

25.1.1 Topsoil stripping and stockpiling will take place on the new RWD site, Valley TSF site and a portion of the FSN4 site. The volumes of topsoil to be stripped and stockpiled are 21 685m^3, 245 229m^3, and 136 739m^3 on the new RWD site, Valley TSF site and on a portion of the FSN4 site respectively.

25.1.2 The total topsoil to be stripped is 403 653m^3 and the available storage volume is 676 302m^3 so there is excess capacity available. The available area and volume for stockpiling is indicated in Figure 28 below.



GEOTHETA CONSULTING ENGINEERS AND SCIENTISTS	Harmony Valley TSF Design Report	2210513

Figure 28: Valley TSF stockpile area and volume

25.1.3 The proposed future Nooitgedacht TSF has a shortfall of topsoil storage space because of changes to the available space, due to the discovery of graves and a property being sold.

25.1.4 The Nooitgedacht TSF requires 1.2 million m^3 of topsoil storage space and the available stockpile area reduced from 1.4 million m^3 to 1.18 million m^3.

25.1.5 Once the available stockpile area at Nooitgedacht is full, the remaining material can be transported to the Valley TSF stockpile area.

25.2 **Water management structures**

25.2.1 A 150mm thick reinforced concrete lined solution trench is provided at the north-west, south and south-eastern sections of the Valley TSF. The trapezoidal solution trench is 1m deep with side slopes of 1V:1.5H and a base width of 1m.

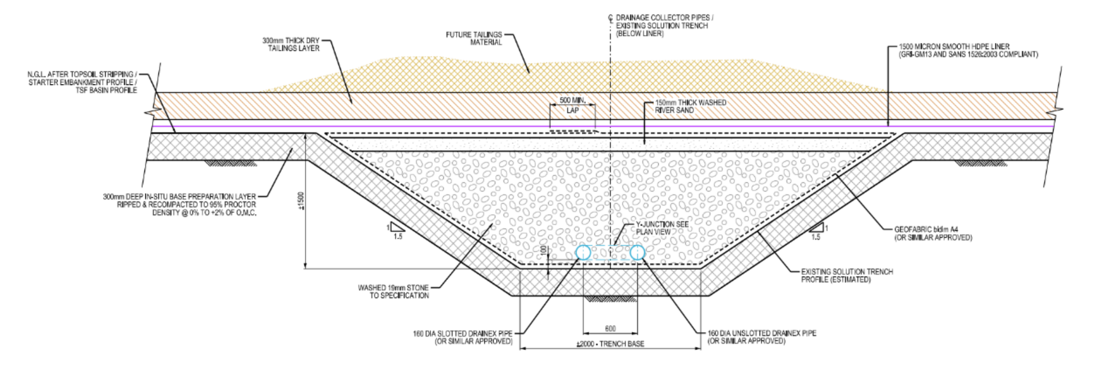
25.2.2 The solution trench conveys effluent from the drain outlets as well as other contaminated water from the facility to the silt trap of the New RWD. The solution trench on the north-western section of the TSF will accommodate the maximum peak discharge from the penstock of 1.02m^3/sec. The solution trench on the south and south-eastern sections of the TSF will accommodate drain flows only.

25.2.3 A concrete lined solution trench will be installed since the effluent is contaminated dirty water. This will prevent seepage of the drain effluent into the underlying soils. It also provides a durable surface for cleaning and maintenance. An HDPE liner can be considered; however the liner is exposed and therefore deteriorates over time. Cleaning and maintenance will need to be done by hand and any damage caused to the liner will need to be repaired immediately.

25.2.4 The existing solution trenches along the east of the facility will need to be assessed and rehabilitated.

25.2.5 The underdrainage towards the south-eastern portion of Valley TSF will flow through the drainage outlet pipes and into the concrete lined solution trench located south of the TSF. This will then flow through the existing solution trench towards the existing RWD located south-west of the Valley TSF. The underdrainage towards the north-western portion of Valley TSF will flow through the drainage outlet pipes and into the concrete lined solution trench located north-west of the TSF. This will then flow through the existing solution trench towards the existing RWD located north-west of the Valley TSF.

25.2.6 The existing earth trenches alongside existing FSN1 and FSN2 will be converted to drainage collector pipes comprising of 160mm pipes surrounded by 19mm stone overlain by a layer of 150mm washed river sand which is enclosed in a geofabric. These drainage collector pipes will retain seepage from FSN1 and FSN2 facilities. The pipe configuration is shown in Figure 30 below. The drainage collector pipe details are shown in Drawing No. 2210513 – 507.



GEOTHETA CONSULTING ENGINEERS AND SCIENTISTS	Harmony Valley TSF Design Report	2210513

Figure 30: Drainage collector pipe section

25.3 Access control

25.3.1 A perimeter fence will not be installed around the TSF complex as the fence is prone to theft. Perimeter barrier warning signs will be installed around the perimeter of the TSF complex as an alternative. The signs will be installed during construction. All signs are to comply with the Harmony Gold Mine standards.

25.3.2 A 5m wide access road is provided around the facility to all key infrastructure for operational and monitoring requirements.

25.4 Tailings slurry delivery system

25.4.1 Slurry will be delivered from One Plant to the TSF site via an overland Cement Mortar Lined (CML) flanged steel pipe up to the perimeter of the TSF.

25.4.2 Slurry will be distributed to cyclones via a 560mm OD PN16 PE100 HDPE ring main provided around the TSF perimeter.

25.4.3 Tailings delivery stations are provided every 30m along the cyclone wall crests to convey tailings slurry from the ring main pipeline to the cyclones. As the facility is raised with tailings, the cyclones will be raised to the new crest elevation. The ring main will be lifted onto new berms as required.

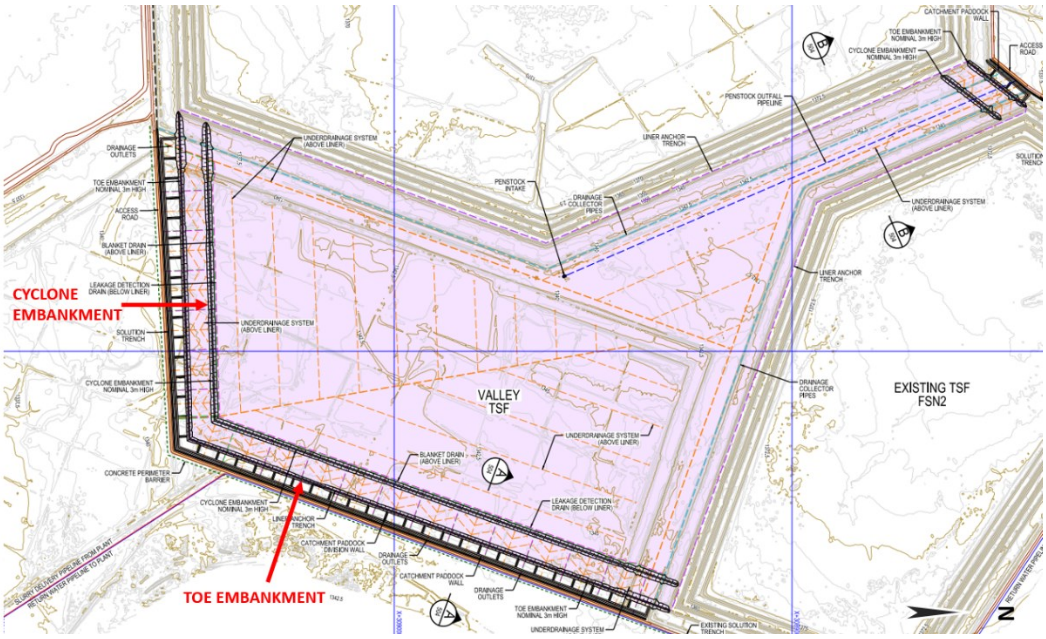
25.5 **Toe and cyclone walls**

25.5.1 The toe walls are constructed at the north-western, southern and south-eastern edges of the Valley TSF and demarcate the extent of tailings underflow deposition.

25.5.2 An engineered toe wall embankment is specified to an elevation of 1348 mamsl to accommodate tailings during initial deposition. The maximum toe wall embankment height is 3m with a 3m wide crest, outer slope of 1V:1.5H and 1V:2H inner slope. The toe wall embankment will be constructed in 150mm layers to 95% Proctor density at 0% to +2% O.M.C. The toe wall material will be won from borrow pits in the basin of the facility. The toe wall embankment is key cut 1.0m into the foundation.

25.5.3 The cyclone walls will be constructed 50m away from the toe wall. These cyclone walls will provide an elevated platform level to allow for overflow tailings deposition. The cyclone wall height is 3m with a 3m wide crest, outer slope of 1V:2H and 1V:2H inner slope.

25.5.4 Figure 31 below shows the positions of the toe and cyclone embankments respectively The layout of the toe and cyclone walls are indicated in Drawing No. 2210513 - 504.



GEOTHETA CONSULTING ENGINEERS AND SCIENTISTS	Harmony Valley TSF Design Report	2210513

Figure 31: Underdrainage layout with cyclone and toe embankment (refer to Drawing No. 2210513-502)

25.6 **Decant system**

25.6.1 The decant system comprises a gravity decant and outfall pipe.

25.6.2 The final penstock intake structures comprise a reinforced concrete base with four 510mm precast concrete penstock rings cast into the base.

25.6.3 As the facility rises the penstock intake will be raised by stacking standard precast concrete penstock rings. The final penstock intake structure is located near the eastern wall of the FSN1 TSF, within the TSF basin to allow for decanting of supernatant water at the final height.

25.6.4 The penstock outfall pipe comprises a 900mm OD PN10 PE100 HDPE pipe.

25.6.5 The penstock outfall pipe discharges into the solution trench situated north-west of the Valley TSF.

25.6.6 The layout of the penstock pipes are shown on drawing No. 2210513 – 509. Figure 32 below show the plan view of the final penstock.

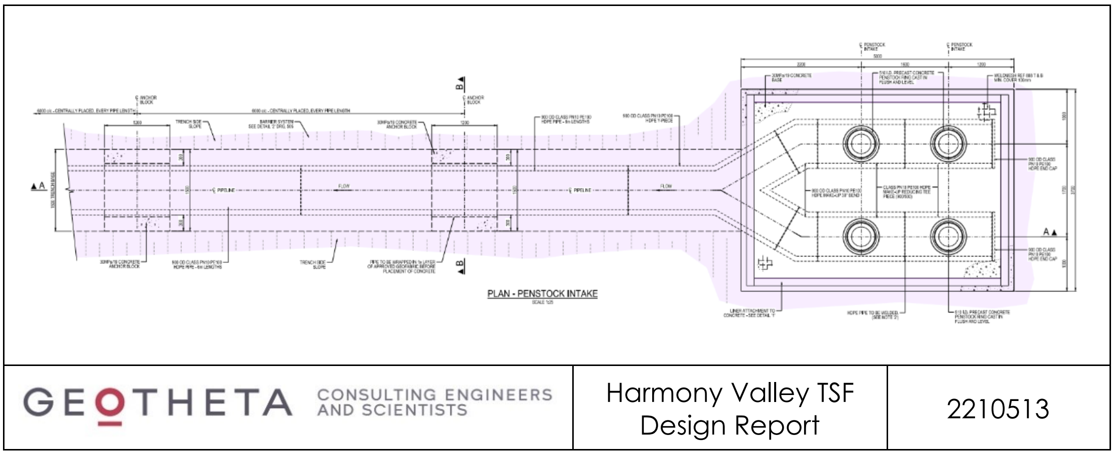


Figure 32: Final penstock

26. Maintenance Plan

26.1 An operating, maintenance and surveillance manual has been prepared for the Valley TSF.

26.2 The objective of the manual is to provide a methodology for the safe, efficient and environmentally responsible management of the TSF and associated infrastructure.

26.3 Adherence to the guidelines provided in the operating, maintenance and surveillance manual will result in continued safe operations of the TSF for the design life.

26.4 The TSF operating, maintenance and surveillance manual is submitted as a separate report (report reference 2210513 – Harmony – Valley TSF OMS Manual - R06).

27. Emergency management plan

27.1 Emergency management is addressed in the TSF operating, maintenance and surveillance manual (report reference 2210513 – Harmony – Valley TSF OMS Manual – R06).

27.2 Emergency management encompasses the following aspects:

27.3 Prevention of emergencies

27.4 Awareness and training

27.5 Continuous monitoring

27.6 On-going assessment of risk

27.7 Emergency response procedures

27.8 A Trigger Action Response Plan (TARP) and Emergency Response Plan (ERP) will be developed by Harmony.

28. Operation and development

28.1 Method of tailings deposition

28.1.1 Tailings will be deposited using cyclones on the northwest, eastern and southern flanks of the Valley TSF.

28.1.2 No cyclone deposition will take place on the outer wall of FSN1 and FSN2 which butt up against the Valley TSF. Spigot and socket deposition will take place from the flanks of the existing facility for pool control only when required. Delivery piping will be placed on the dormant facilities as required.

28.1.3 During cyclone tailings deposition, the total tailings stream is split into a coarse fraction (underflow) and fine fraction (overflow) by centrifugal separation.

28.1.4 The coarse underflow is usually discharged as a flare or spray in the shape of an inverted cone (spray discharge). A continuous discharge with the appearance of a rope (roping discharge) must be avoided. The optimum split of underflow is usually achieved when the underflow is spraying, but just at the point between spraying and roping. An underflow : overflow mass split of 17 : 83 was used in the stage capacity calculations.

28.1.5 The cyclones are supported on customised steel stands placed in such a manner that an underflow cone of about 1.2m high will be deposited. The cyclone and stand are then moved to an adjacent position to deposit another underflow cone. The cyclone should also be moved to fill in low spots between underflow cones to ensure an even horizontal surface along the top of the outer wall.

28.1.6 The fine overflow will be discharged into the basin through an overflow pipe connected to the cyclone. The end of the overflow pipe discharging into the basin should always be at a lower elevation than the cyclone vortex finder. During commissioning the overflow pipes must be long enough to discharge overflow directly into the basin area beyond the blanket drains.

28.1.7 Overflow must be discharged well beyond the coarse underflow zone and must not be discharged directly over the exposed toe or blanket drains during commissioning.

28.1.8 Deposition of the tailings material must be done according to the deposition plan. The deposition plan must ensure that the rate of rise of the cyclone underflow is greater than the rate of rise of the basin.

28.1.9 The deposition position into the basin is to be selected based upon managing the height of solids around the TSF perimeter and the shape of the pool. The deposition locations are to be rotated around the facility to ensure adequate beach formation and favourable pool location and size.

28.2 Method of embankment construction

28.2.1 Valley TSF will be built in two stages. The first stage will consist of coarse underflow tailings being filled between the toe wall and cyclone wall, by downstreaming from the cyclone wall.

28.2.2 The toe wall was sized for freeboard requirements and the cyclone wall was sized for slope stability requirements.

28.2.3 Upstream deposition will commence once the area between the toe wall and the cyclone wall has been filled with cyclone underflow material.

28.2.4 When the upstream method of embankment construction is employed, the facility is raised using the underflow material. Tailings material is then placed on previously deposited tailings which has consolidated sufficiently and is safe to access. This is shown in Figure 33.

28.2.5 The upstream deposition cycle must allow for the previous layer of deposition to dry before the next layer is deposited.

28.2.6 The two stage deposition methods as described above has been implemented and is operational on the Elikhulu Upper and Lower compartments.

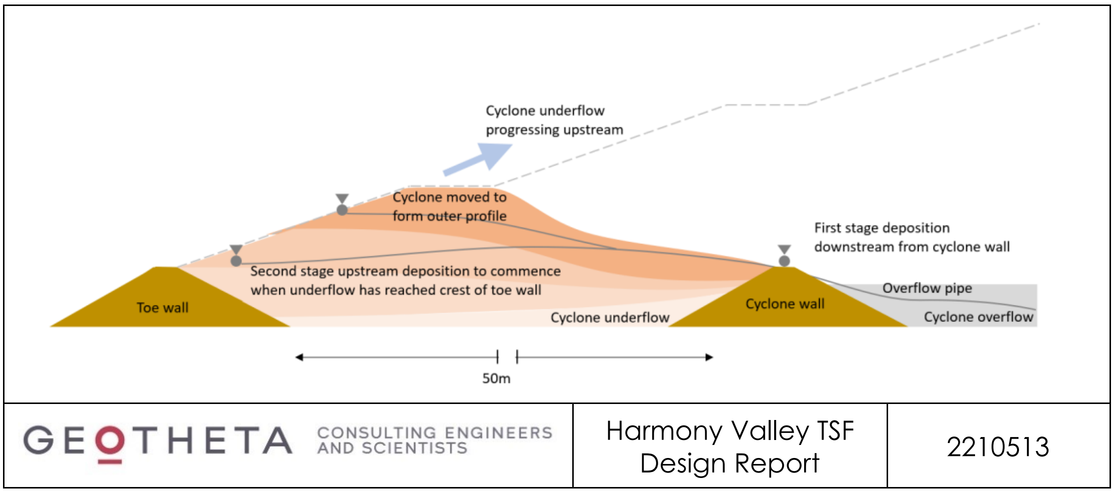


Figure 33: Typical two-stage cyclone embankment construction

28.3 Freeboard and pool control

28.3.1 Freeboard is defined as the vertical distance between the operating pool level and the lowest point on the top of the outer wall. The minimum freeboard must be equal to the water level rise that is caused by a 1:50 year 24-hour storm event plus 800 mm [Regulation 704 of the Water Act - Act 36 of 1998], or the 1:100 year 24 hour storm event plus 500 mm [GN R527(73) of the Mineral and Petroleum Resources Development Act 28 of 2002]. The minimum freeboard requirement is illustrated in Figure 34 below.

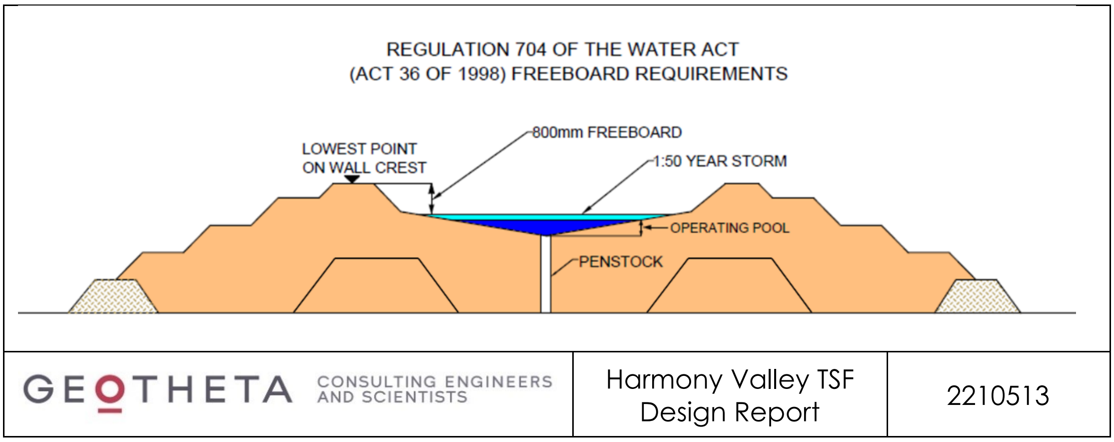


Figure 34: Freeboard requirement

28.3.2 The facility must be operated so that it can always contain the 1:50 year 24-hour storm event on top of the normal operating pool level whilst maintaining 800mm freeboard, or the 1:100 year 24 hour storm event on the normal operating pool level whilst maintaining 500mm freeboard.

29. Closure plan

29.1 Closure objectives

29.1.1 The objectives of the closure and rehabilitation procedures will be:

- To establish a self-sustaining eco-system solution that minimises the need of continuous maintenance.
- To minimise the potential impact to the surrounding environment.
- To create safe and stable landforms.

29.1.2 In achieving these objectives, the closure and rehabilitation procedures must comply with the relevant legislation.

29.2 Closure considerations

29.2.1 The TSF is to be constructed and operated towards final closure.

29.2.2 A detailed closure plan will be developed during the life of the TSF.

29.2.3 The objectives for the closure and rehabilitation of the TSF is to prevent pollution to the surrounding environment and ensure a stable facility is maintained.

29.2.4 The final surface of the facility will be the same configuration as the operating dam with inter-bench slopes of 1V:4H.

29.2.5 The outer surface, the benches and the shape top surface will be covered with a layer of stripped and stockpiled topsoil (retained from earlier construction removal). The topsoil will be grassed and vegetated to form a self-sufficient eco-system. (The vegetation design will be done by others).

29.3 Final geometry

29.3.1 It is intended that the upper surface of the facility will be shaped to divert rainfall off the facility.

29.3.2 The outer slopes of the facility ensure structural stability with limited erosion damage.

29.4 Water control

29.4.1 Stormwater around the facility will be gravity-drained away from the Valley TSF. The stormwater will be directed to the nearest water course.

29.4.2 The run-off from the side slopes of the TSF wall will be attenuated by the vegetation cover established at closure.

29.4.3 Dirty runoff water within the Valley TSF catchment area is routed to one of the two RWDs via the concrete lined solution trenches.

29.5 Vegetation

29.5.1 Vegetation on the surface and outer slopes of the facility will reduce erosion and dust generation.

29.5.2 Vegetation on all the outer side slopes is to be established at closure. The growth medium, vegetation establishment and maintenance will be designed by others.

30. Drawings

30.1 Signed drawings are included in Appendix G.

31. Quality assurance and quality control

31.1 The construction of the Valley TSF and associated infrastructure will be done according to strict quality assurance and quality control requirements.

31.2 The Liner CQA report is submitted separately under reference 2210513 - Harmony – Valley TSF CQA Report - R05.

31.3 The civil works will be done according to SANS 1200, subject to Particular Specifications for certain aspects.

32. Bill of Quantities

32.1 A Bill of Quantities has been prepared and is included in Appendix H. The Bill of Quantities is issued separately for use by Harmony for tender purposes.

33. Price estimates and construction timelines

33.1 The following contractors have provided rates for the construction of Valley TSF:

- Stefanutti Stocks Inland (SSI)
- Intasol
- WBHO

33.2 A priced Bill of Quantities from each contractor is included in Appendix I.

33.3 A summary of the estimated capital costs from each contractor is provided in Table 22 below.

Table 22: Estimated capital costs

SECTION	DESCRIPTION		INTASOL	WBHO	SSI
			AMOUNT Excl VAT	**AMOUNT Excl VAT**	**AMOUNT Excl VAT**
	PART 1: GENERAL				
A	PART A: PRELIMINARY AND GENERAL	R	106 257 815	108 711 362	99 949 187
B	PART B: SITE CLEARANCE	R	54 651 987	34 769 152	48 220 275
C	PART C: EARTHWORKS	R	106 214 220	142 313 193	114 287 067
D	PART D: SMALL EARTH DAMS	R	233 290	586 358	219 050
E	PART E: EARTHWORKS (ROADS, SUBGRADE)	R	786 068	838 898	429 476
F	PART F: CONCRETE (STRUCTURAL)	R	97 595 970	76 825 861	93 423 176
G	PART G: MEDIUM PRESSURE PIPELINES	R	19 640 994	14 052 672	16 085 242
H	PART H: SEWERS	R	3 118 256	23 564 324	3 391 631
I	PART I: STORMWATER DRAINAGE	R	179 479	142 790	76 570
J	PART J: GEOSYNTHETICS	R	169 797 364	155 357 438	149 543 826
K	PART K: TIMBER (STRUCTURAL)	R	1 879 691	2 293 763	2 150 908
L	PART L: CYCLONES	R	7 893 178	21 913 645	6 734 461
TOTAL CONSTRUCTION EXCLUDING VAT		R	**568 248 313**	**581 369 457**	**534 510 867**
ESTIMATED PROFESSIONAL FEES		R	17 047 449	17 441 084	16 035 326
CONSTRUCTION CONTINGENCY		R	113 649 663	116 273 891	106 902 173
BUDGET PRICE		R	**698 945 425**	**715 084 432**	**657 448 367**

33.4 The recommended budget allocation for the Valley TSF construction is R690 million (including 3% professional fees and 20% construction contingency).

34. Conclusions and recommendations

Based on the detailed design of the Valley TSF, the following conclusions and recommendations can be drawn:

34.1 Safe operating systems and procedures are to be implemented during operation of the facility.

34.2 Key parameters of the Valley TSF design are:

- Maximum final height: 36m
- Footprint area of facility: 163.5 Ha
- Total capacity: 56.8 million tons
- Total deposition period at 600 000 tons per month: 8.0 years
- Maximum rate of rise (Basin): 4.12m/year

- Maximum rate of rise (Embankment): 3.99m/year
- Deposition method: Cyclone

34.3 The Valley TSF will be developed with an intermediate outer slope of 1V:3H between benches. The inter-bench height is 8.0m and the benches are 8.0m wide. The overall slope with benches is 1V:4H.

34.4 The maximum toe wall embankment height is 3.0m with a 3.0m wide crest, outer slope of 1V:1.5H and 1V:2H inner slope. The toe wall embankment will be constructed in 150mm layers to 95% Proctor density at 0% to +2% O.M.C. The toe wall material will be obtained from borrow pits in the basin of the facility.

34.5 The cyclone walls will be constructed 50m away from the toe wall. These cyclone walls will provide an elevated platform level to allow for overflow tailings deposition. The cyclone wall height is 3m with a 3m wide crest, outer slope of 1V:2H and 1V:2H inner slope.

34.6 The Valley TSF provides a storage capacity of 56.8 million tons over a deposition period of 8.0 years at the target deposition rate of 600 000tpm with a maximum rate of rise of 4.12m/year (basin) and 3.99m/year (embankment). This rate of rise will be achieved by cyclone deposition.

34.7 According to GISTM, the Valley TSF has a **Very High Consequence Classification** rating.

34.8 Based on SANS 10286, the Valley TSF has a **High Hazard Classification** rating.

34.9 The Valley TSF will butt up against the dormant FSN1 TSF on the West and FSN2 TSF on the East. Tailings will be deposited using cyclones on the northwest, eastern and southern flanks of the Valley TSF. The Valley TSF is designed to be an upstream cyclone facility. No cyclone operation will occur at the FSN1 and FSN2 TSF interface, spigotting or open-end deposition will be done for pool control only.

34.10 Few of the dormant up-stream deposited facilities meet the legislated requirements based on the current Limited Equilibrium method of stability analysis. The FSN1 and FSN2 facilities do not presently comply with legislated requirements.

34.11 To ensure the entire complex complies to the required Factors of Safety at closure, a finite element stability analysis will be conducted, and remedial works for FSN1 and FSN2 may be incorporated into the Valley TSF operation and closure plan if required.

34.12 The minimum Factor of Safety based on the Limit Equilibrium method of stability analysis, against failure is 2.0 under drained conditions, 1.6 under undrained conditions, 1.2 under post seismic, post liquefaction or residual conditions and 1.3 under pseudo static conditions. These Factors of Safety comply with the local regulation and international slope stability standards.

34.13 The gold tailings material classified as a Type 3 waste as provided by Jones and Wagner. This necessitates a Class C barrier system however as per an independent review by Legge and Associates, an 'inverted barrier' system is a more practical and feasible option. This inverted barrier system is used in the design of the Valley TSF barrier system.

34.14 The Valley TSF 'inverted barrier' liner system has two different areas. The liner in area 1 is within the central basin of the TSF. This liner system comprises (from top down), a 300mm thick layer of tailings, above liner drain comprising 160mm perforated HDPE pipes, 1.5mm thick smooth HDPE liner underlain by a 300mm ripped and recompacted layer of in-situ base preparation.

34.15 The Valley TSF design with a Class C liner system required a 200T size geogrid (or similar approved). However, with the 'inverted barrier' system, a 150T size geogrid (or similar approved) is required. The Liner in area 2 is located at the outer walls of the facility. A 150T geogrid (or similar approved) will be placed from the toe wall inwards for 100m. The liner system at the outer walls comprises (from top down), a 300mm thick layer of tailings, a 150T size geogrid (or similar approved), a 300mm thick layer of tailings, above liner drain comprising 160mm perforated HDPE pipes, 1.5mm thick double textured HDPE liner underlain by a 300mm ripped and recompacted layer of in-situ base preparation.

34.16 The under-liner leakage detection drains on the Valley TSF comprises 160mm slotted Drainex HDPE pipes surrounded in 19mm stone which is enclosed in a geofabric. The leakage detection drain outlet pipes on the south-eastern section discharge into the solution trench located to the south of the Valley TSF. The leakage detection drain outlet pipes on the north-western section discharge into the solution trench located to the north of the Valley TSF.

34.17 The above-liner blanket drain system on the TSF comprises 160mm slotted HDPE pipe surrounded in 19mm stone which is covered by layers of 6mm stone and graded filter sand, enclosed in a geofabric. The blanket drain outlet pipes on the south-east section discharge into the solution trench located to the south of Valley TSF. The blanket drain outlet pipes on the north-western section discharge into the solution trench located to the north of Valley TSF.

34.18 A 150mm thick reinforced concrete lined solution trench is provided along the north-west, south and south-eastern sections of the TSF. The trapezoidal solution trench is 1m deep with side slopes of 1V:1.5H and a base width of 1m. The designed maximum flow depth in the channel is 800mm so that the channel capacity of 3.87m^3/sec can accommodate the maximum peak discharge from the penstock of 1.02m^3/sec when the pool depth is 200mm. The solution trench on the north-western section of the TSF will accommodate the maximum peak discharge from the penstock of 1.02m^3/sec. The solution trench on the south and south-eastern sections of the TSF will accommodate drain flows only.

34.19 A hydrotechnical assessment was done to determine the climatic and meteorological data. This data was used to size the new Return Water Dam (RWD) situated north-west of the TSF and associated water infrastructure. The climatic and meteorological data was used to do a capacity assessment on the existing RWD situated south-west of the TSF.

34.20 A concrete lined spillway is provided at the new RWD to safely discharge excess water without overtopping of the RWD embankment walls. The RWD spillway has a freeboard of 800mm and has been designed to discharge the 1:10 000 24-hour Probable Maximum Flood volume of 9.9m^3/sec.

34.21 The new RWD has a total storage capacity of 220 000m^3 which is sufficient to ensure that it does not spill more than once every 50 years with the inflow from the penstock and underdrains on the north-west of the TSF, when operated at a level of 0.276m.

34.22 A silt trap is provided upstream of the new RWD. The silt trap includes infrastructure to enable cleaning. The silt trap allows solids to settle out of the water before entering the

RWD, thereby minimising sedimentation in the RWD. The silt trap is a 2.0m deep reinforced concrete water retaining structure with a concrete spillway to route de-silted water to the RWD.

34.23 The new RWD liner system comprises a 200mm high geocell filed with 20Mpa concrete, underlain by a 1.5mm thick smooth HDPE liner and a 300mm layer of base in-situ preparation.

34.24 The new RWD underdrainage comprise 160mm slotted HDPE pipes encased in 19mm washed stone. The stone will be wrapped in geofabric.

34.25 A perimeter barrier with warning signs will be installed around the TSF. A 5m wide access road is provided around the facility for operational and monitoring requirements.

34.26 Preliminary work has been carried out to assess the required remedial work at the FSN1 and FSN2 facilities based on the limited equilibrium method of stability analysis. These recommendations will be updated once the Finite Element Stability analysis has been conducted and if required any resulting requirements for remediation will be designed and implemented to occur simultaneously with the Valley TSF operation so that the overall factors of safety comply at the time of Valley TSF closure.

34.27 The facility is to be constructed and operated to ensure that the designed outer slope profile is achieved, and that operations are safe and environmentally responsible.

34.28 Monitoring of the facility is to be undertaken as outlined in the Operating, Maintenance and Surveillance Manual.

34.29 The preliminary BOQ pricing returns are:

SSI	
Construction cost:	R 535 Million
Professional fees at 3%:	R 16 Million
Add contingency at 20%:	R 107 Million
Total excluding VAT	**R 657 Million**
INTASOL	
Construction cost:	R568 Million
Professional fees at 3%:	R 17 Million
Add contingency at 20%:	R 113 Million
Total excluding VAT	**R 699 Million**
WBHO	
Construction cost:	R 581 Million
Professional fees at 3%:	R 17 Million
Add contingency at 20%:	R 116 Million
Total excluding VAT	**R 715 Million**

34.30 The estimated construction cost for the Valley TSF is R561 million and the recommended budget allocation is R690 million (including 20% contingency and professional fees).

Prepared by:

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In terms of Geotheta Quality Policy, this report has been reviewed, product corrected and certified okay for distribution and use.

Reviewed by:

David Smith Pr Eng
Executive Director: Geotheta (Pty) Limited

Ian Hammond Pr Eng
CEO: Geotheta (Pty) Limited

APPENDIX A: WASTE CLASSIFICATION REPORT

APPENDIX B: STAGE CAPACITY

APPENDIX C: GEOTECHNICAL INVESTIGATION REPORT

APPENDIX D: SLOPE STABILITY OUTPUTS

APPENDIX E: LEGGE AND ASSOCIATES REVIEW REPORTS

APPENDIX F: LINER CALCULATIONS

APPENDIX G: DRAWINGS

APPENDIX H: BOQ (UNPRICED)

APPENDIX I: BOQ (PRICED)

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Name/Title	Company	Copy	Date	Authorised by
Mr Trevor Leonard	Harmony Gold Mining Company Limited	Electronic	February 2024	Ian Hammond

Approval Signature: