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GAMSBERG PHASE 2 - TAILINGS STORAGE FACILITY
DESIGN REPORT

Rev	Description	Date
B	Issued in Draft	15 April 2024

Knight Piésold
CONSULTING

EXECUTIVE SUMMARY

For information only. To be completed when final.

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ABBREVIATIONS

***Project .... the Phase 2 TSF Project
ADT .... Articulated dump trucks
ABA .... Acid-Base Accounting
AMD .... Acid mine drainage
AP .... Acid generation processes
ARD .... Acid rock drainage
ANCOLD .... Australian National Committee on Large Dams
BAW .... Beach above water
BBW .... Beach below water
BCM .... Bank cubic metre
BEEH .... School of Bioresources Engineering and Environmental Hydrology
BOQ .... Bill of quantities
BMM .... Black Mountain Mining
c .... Cohesion
CBR .... California bearing ratio
CCL .... Compacted Clay Liner
CCM .... Compacted cubic metre
CCS .... Consequence classification of structures
CQA .... Construction Quality Assurance
cu .... Undrained cohesion
DWS .... Department of Water and Sanitation
EIA .... Environmental impact assessment
EIS .... Environmental impact statement
EMP .... Environmental Management Plan
EPCM .... Engineering, procurement, and construction management
FoS .... Factor of safety
g .... Gravitational acceleration
GAI .... Geochemical abundance index
GCL .... Geosynthetic Clay Liner
GMB .... Geomembrane
GN .... Government Notice
GTX-NW .... Geotextile – Non-woven
Ha .... Hectare
HDPE .... High-density polyethylene
ICFR .... Institute for Commercial Forestry Research
KP .... Knight Piésold
kg .... Kilograms
kN .... Kilonewtons
LC .... Leachable Concentrations
LCM .... Loose cubic metre
LCT .... Leachable Concentration Threshold
LIDAR .... Light Detection and Ranging
LOM .... Life of mine
m .... Metres
m^2 .... Square metres
m^3 .... Cubic metres

MAE .... Mean Annual Evaporation
MAP .... Mean Annual Precipitation
mamsl .... Metres above mean sea level
MCE .... Maximum credible earthquake
mg/kg .... milligrams per kilogram
mg/l .... milligrams per litre
mm .... Millimetres
Mt/a .... Million tonnes per annum
NEMWA .... National Environmental Management: Waste Act
NGL .... Natural Ground Line
NNP .... Net neutralisation potential
NP .... Acid-neutralizing processes
NPAG .... Non-acid generating
NPR .... Neutralising Potential Ratio
NWA .... National Water Act
OD .... Outside diameter
PAG .... Potentially acid generating
PGA .... Peak Ground Acceleration
PMP .... Probable maximum precipitation
PMF .... Probable maximum flood
PSD .... Particle size distribution
RoR .... Rate of Rise
RWD .... Return Water Dam
SANAS .... South African National Accreditation System
SANS .... South African National Standard
SAWS .... South African Weather Service
t .... tonnes
TC .... Total Concentrations
TCT .... Total Concentrations Threshold
TSF .... Tailings storage facility
WCMR .... Waste Classification and Management Regulations
XRD .... X-Ray Diffraction
yr. .... year
ø .... Friction angle
TLB .... Tractor, Loader and Backhoe

1.0 INTRODUCTION

1.1 PROJECT BACKGROUND

Vedanta Resources Plc is a globally diversified natural resources group with wide ranging interests in aluminium, copper, zinc, lead, silver, iron ore, oil and gas and power. Its operations in South Africa, Vedanta Zinc International (VZI), include Black Mountain Mining (SA)

Black Mountain Mining (Pty.) Ltd. (BMM) comprises of The Black Mountain Mine (Deeps and Swartberg Operations) and the Gamsberg Mine. Both zinc-lead mines are located near Pofadder in the Northern Cape Province, along the N14 National highway linking Upington to Springbok.

The Gamsberg Project is one of VZI's flagship projects in the journey of realizing their vision to produce 500 ktpa of finished zinc metal from Gamsberg. It will exploit one of the largest, known, undeveloped zinc ore bodies in the world. The first step was Phase 1 (4 Mtpa Mines & Concentrator) which was commissioned in September 2018 and is currently in Operation. Phase 1 is currently producing ~ 250 ktpa Zn MIC.

KP was responsible for the Detail Design and Construction Quality Supervision of the TSF Phase 1. KP was however not appointed as the Engineer of Record (EoR) for the TSF for the operational phase.

This Design report is for the TSF for Phase 2 of the project to increase the ore beneficiation capacity with an additional 4 Mtpa run of mine.

A conceptual design for the TSF Phase 2 was completed by KP in January 2022, no prefeasibility or feasibility studies were completed after the conceptual design. The conceptual layout of the TSF is presented in Figure 1-1. It was envisaged that the TSF will be extended to the north to fit within the DWS approved footprint. The exact location of the Return Water Dam (RWD) and settling ponds was preliminary at the inception of the project was determined during the detailed design process.

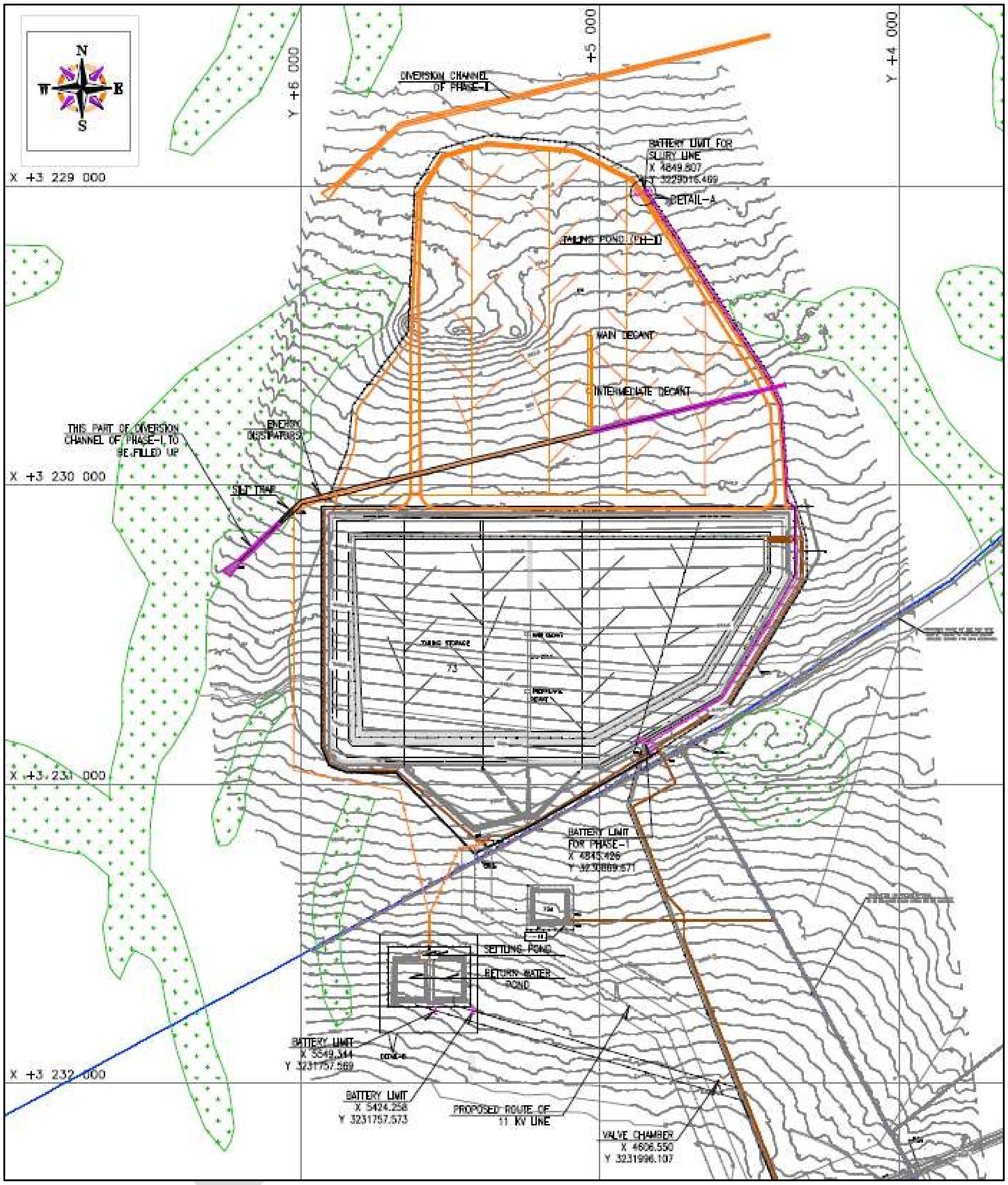


Figure 1-1 Phase 2 TSF Concept Drawing

1.2 LOCALITY

The Gamsberg mine is located in the Province of Northern Cape in South Africa. It is adjacent to the N14 national road linking Upington to Springbok, 20 km east of the Black Mountain Mine and the town of Aggeneys. The locality Map is presented in Figure 1-2 below.

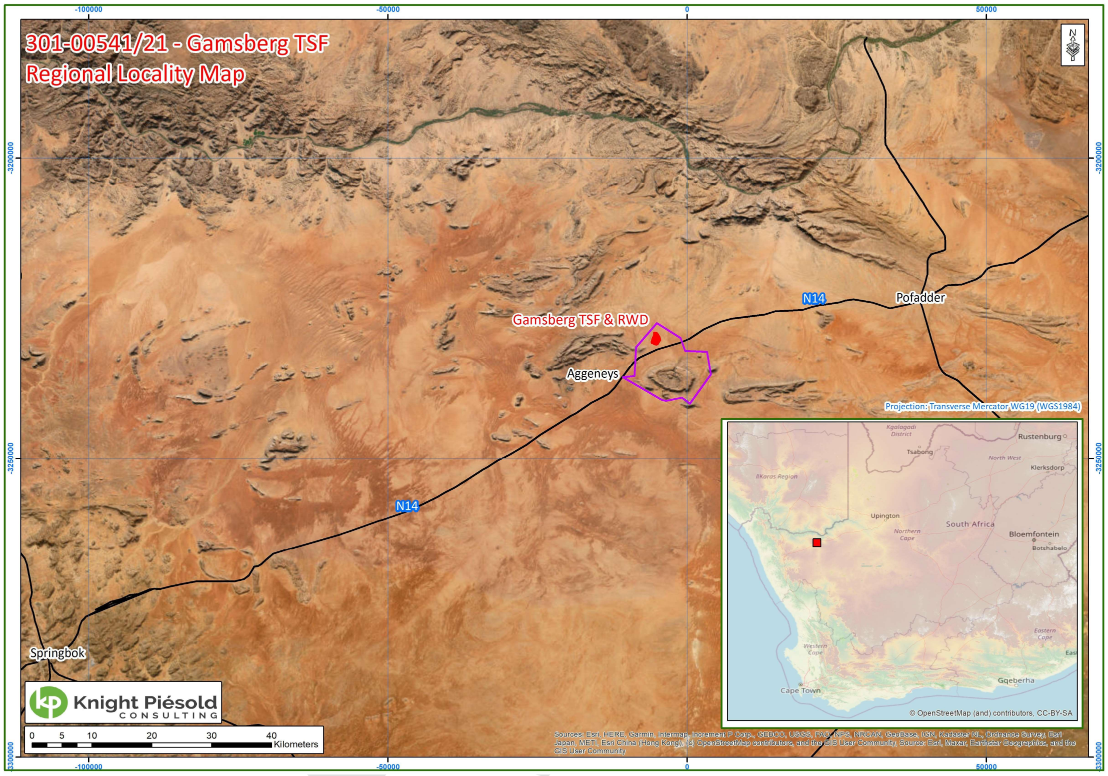


Figure 1-2: Gamsberg Mine Locality Map

The centre of the proposed Phase 2 TSF is located at the longitude and latitude coordinates of 29°11'1.71"S, 18°56'51.17"E and its location relative to the mine is presented in Figure 1-3.

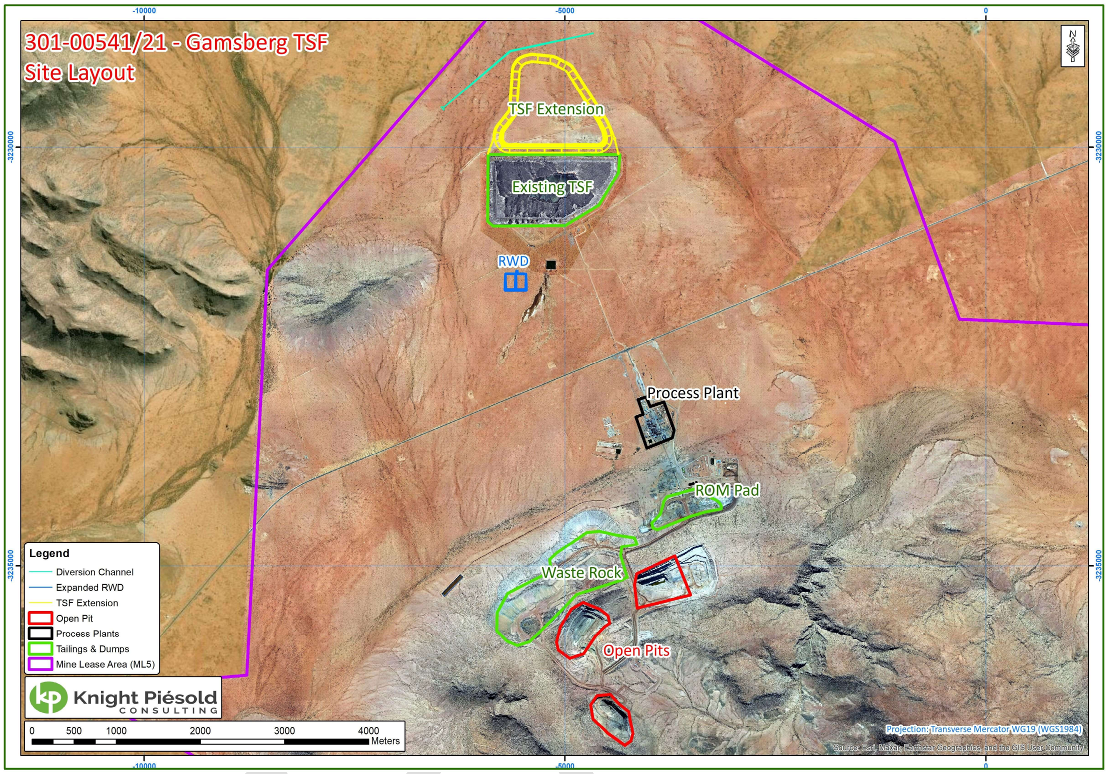


Figure 1-3: Gamsberg Phase 1 TSF locality Map

1.3 SCOPE SUMMARY

The scope for the design of the Phase 2 TSF is summarised as follows:

- Project Management
    - Progress meetings every week
    - Review of documentation
    - Site visit for relevant team members

- Geotechnical Investigation, foundation and materials study including testing of borrow materials – covered in a separate proposal.
- Tailings geotechnical test work - covered in a separate proposal.
- Topographical survey (if required)
- Waste classification of the tailings
- Hydrogeology study, including recommendations on monitoring boreholes
- The tailings deposition pipelines (slurry delivery system) around the TSF, fitted with valves and feeding the cyclone banks.
- Cyclone assessment
- A cyclone deposition plan and wall building sequence (Operating Manual)
- TSF Layout optimisation
- Stage Capacity Assessment
- Seepage and Stability Assessment
- Design of Starter walls
- Hydrology Assessment
- Water Balance
- An HDPE barrier system for the TSF according to the waste classification, including temperature measuring devices for monitoring of geomembrane temperature
- The decant system
- Access to the decant system
- Energy dissipators
- An underflow drain system consisting of slotted HDPE piping and a filter system
- Sumps where the drainage outlet pipes exit underneath the starter wall
- HDPE piping from sumps to the silt trap and from silt trap to the RWD.
- Silt Trap, Settling Pond with Sluice Gates
- Perimeter roads around the TSF.
- Fencing and access gates
- An HDPE lined RWD having two separate chambers. One chamber will be for Phase-2 and the other will be common for both Phase-1 & Phase-2 for maintenance and cleaning of the RWDs. Connections of Phase-1 pipelines with this common chamber shall be considered in design.
- Emergency spillway for the RWD
- Pump system for pipeline from the RWD towards the plant up to battery limit.
- Toe drains around the outside of the TSF for managing storm water
- An unlined storm water dam (SWD)
- A clean water diversion channel
- Dam Breach Assessment
- Deliverables
- Presentation of deliverables to DWS
- Water Use Licence Amendment Application for the RWD

It was understood that the Section 21(g) Water Use License (WUL) approval has been obtained for the TSF and RWD. An amendment to the WUL is however required for the increase in the RWD size. Based on outcomes of the studies and liaison with the DWS, it may also be required to amend the 21(g) of the TSF.

1.4 BATTERY LIMITS

- Battery limits for the TSF Package were as follows:
  - The slurry pipeline battery limit will be on the north-east side of the TSF, as indicated in Figure 1-1. Isolation valves will be included for the ring-feed system on top of the TSF.
  - The return water line battery limit will be downstream (south) of the RWD wall, at the pumpstation outlet flange. The approximate location is indicated in Figure 1-1. The final layout will be discussed with the EPC Contractor. The pipes will where possible be kept above ground.
  - The TSF access roads will connect to the existing TSF roads.
  - Electrical - Terminals of the transformer – we will produce a Protection Logic drawing to ensure the co-ordination of the interface.
  - Control and Instrumentation - The terminals of the PLC/RIO panel housed in the MCC or motor starter panel as the case may be.

2.0 DESIGN CRITERIA

The design criteria used within this design report are presented within Table 2-1. The pump, pipeline, electrical, control and instrumentation required for the TSF decant system and RWD water return have been listed below as draft as these items are currently being designed.

Table 2-1: Gamsberg Mine Phase 2 TSF Design Criteria

Parameter (Unit)	Value/Description	Source/Comments
Design By:	Knight Piésold Consulting 2023	Note
Main Design Standards	SANS 10286	Legislative Requirement
	GISTM	ICMM Requirement
SANS 10286 (1998) Hazard Rating	Medium	TSF Phase 2 Design
GISTM Consequence Classification (2020)	Significant	Tailings Dam Break Analysis
Location, Size and Geometry		
Location (Latitude/Longitude)	29°11'1.71"S, 18°56'51.17"E	Centre Point of Phase 2 TSF Design
TSF Footprint Area (ha)	116	TSF Phase 2 Design
TSF Toe Perimeter (km)	4.2	TSF Phase 2 Design
Maximum Crest Height above NGL(m)	43	TSF Phase 2 Design
Maximum Crest Elevation (mamsl)	990	TSF Phase 2 Design
Intermediate Slope (1V:?H)	3	TSF Phase 2 Design
Vertical Bench Intervals (m)	10	TSF Phase 2 Design
Bench Widths (m)	5	TSF Phase 2 Design
Overall Slope (1V:?H)	3.5	TSF Phase 2 Design
Storm Water/Safety Berm Height for all Bench Crests (m)	0.5	TSF Phase 2 Design
Design Life (years)	12	Scope BMM
Tailings Delivery		
Maximum Tonnage (Mt/yr.)	4	Scope BMM
Tonnage Range (t/month.)	333 333	Scope BMM
Availability (Hours)	8030	Scope BMM
Solids to TSF (tph)	440.38	Scope BMM
Solids SG	3.39	Scope BMM
Solids to TSF (m^3/hr)	129.88	Scope BMM
Water to TSF (m^3/hr)	380.31	Scope BMM
Slurry to TSF (tph)	820.69	Scope BMM

Slurry to TSF (m^3/hr)	510.2	Scope BMM
Solids Percentage (%)	53.66%	Scope BMM
Tailings Pulp/Slurry Density (t/m^3)	1.61	Scope BMM
Slurry Delivery Rate (tph)	792-834	Scope BMM
Tailings Delivery Line Size OD (mm)	315mm	315mm in BMM Scope
Capacity and RoR		
Required Capacity (m^3)	32 000 000	Scope BMM at 1.5 in-situ dry density
Required Capacity (tonnes)	48 000 000	Scope BMM @ 4MTPA for 12 years
Maximum Design RoR – Underflow (m/yr.)	5.5	TSF Phase 2 Design
Maximum Design RoR - Overflow (m/yr.)	4.5	TSF Phase 2 Design
Tailings Properties (Based on Testing performed on Phase 1)		
Tailings Feed PSD -Typical (% passing)	100	<1
	99	<0.425
	96	<0.25
	90	<0.15
	70	<0.075
	64	<0.06
	60	<0.05
	49	<0.035
	36	<0.02
	16	<0.006
	6	<0.002
Tailings Overflow PSD -Typical (% passing)	100	<1
	100	<0.425
	99	<0.25
	94	<0.15
	77	<0.075
	65	<0.06

	56	<0.05
	41	<0.035
	30	<0.02
	18	<0.006
	9	<0.002
Tailings Underflow PSD -Typical (% passing)	100	<1
	100	<0.425
	89	<0.25
	69	<0.15
	38	<0.075
	23	<0.06
	17	<0.05
	9	<0.035
	6	<0.02
	4	<0.006
	3	<0.002
Plasticity Underflow	NP	Based on Testing performed on Phase 1 Tailings
Plasticity Overflow	Between 2 and 3	Based on Testing performed on Phase 1 Tailings
Tailings Classification – Underflow (ASTM D2487)	SM	Based on Testing performed on Phase 1 Tailings
Tailings Classification – Overflow (ASTM D2487)	ML	Based on Testing performed on Phase 1 Tailings
Tailings Specific Gravity (Ratio)	Between 3.3 to 4.4	Based on Testing performed around Phase 1. UF varies between 4 and 4.4. OF varies between 3.3 and 3.9
Tailings Average in-situ Dry Density (t/m^3)	1.5	Based on Phase 1 Survey volumes / tonnes deposited
Tailings Permeability - Overflow (m/s)	$3.77E^{-08}$	Based on Testing performed on Phase 1 Tailings
Tailings Permeability - Underflow (m/s)	$9.75E^{-7}$	Based on Testing performed on Phase 1 Tailings
Tailings Friction Angle Underflow (°)	31	Based on Testing performed on Phase 1 Tailings
Tailings Friction Angle Overflow (°)	29	Based on Testing performed on Phase 1 Tailings

Tailings Cohesion (kPa)	0	Based on Testing performed on Phase 1 Tailings
Tailings Beach Slope (%)	0.77% (0.67% - 93%)	Based on Testing performed on Phase 1 Tailings
Tailings Waste Classification as per GN 635	Type 3	Based on Testing performed on Phase 2 Tailings sample from laboratory (Phase 2 plant has different lead circuit to that of Phase 1)
Liner Details		
Liner Requirement as per GN636	Yes, Class C	Liner Memorandum
Liner Layering Design	1080 GSM geotextile, overlayed by 1.5mm HDPE geomembrane. Drainage above and below liner.	Liner Memorandum. Double textured under and between heel and toe wall
Hydrology and Water Management		
Minimum Freeboard Requirement (m)	2	Scope BMM – validated by the water balance
1:2 yr. 24hr Storm (mm)	20	Hydrology Analysis
1:5 yr. 24hr Storm (mm)	32.53	Hydrology Analysis
1:10 yr. 24hr Storm (mm)	42	Hydrology Analysis
1:20 yr. 24hr Storm (mm)	51	Hydrology Analysis
1:50 yr. 24hr Storm (mm)	62	Hydrology Analysis
1:100 yr. 24hr Storm (mm)	70	Hydrology Analysis
1:200 yr. 24hr Storm (mm)	78	Hydrology Analysis
1:2 475 yr. 24hr Storm (mm)	107.54	Hydrology Analysis
1: 5000 yr. 24hr Storm (mm)	115.87	Hydrology Analysis
1:10 000 yr. 24hr Storm (mm)	124	Hydrology Analysis
Probable Maximum Precipitation (mm)	228.41	Hydrology Analysis
Annual Evaporation (mm)	2813	Open water evaporation
Annual Precipitation (mm)	92.22	Hydrology Analysis
Stability Requirements		
Minimum FoS - Drained	1.5	GN 632 and ANCOLD
Minimum FoS - Drained Pseudo Static	1.1	ANCOLD- Best Practise
Minimum FoS – Undrained Peak	1.5	ANCOLD- Best Practise
Minimum FoS – Undrained Residual	1.1	ANCOLD- Best Practise

Peak Gravitational Acceleration	0.1 g -1:475 year event	Phase 1 Study/BMM Scope
Return Water Dam		
Lining	1.5 mm double lined HDPE Geomembrane with leakage detection	TSF Phase 2 Design
Capacity below spillway invert (m^3)	96 706.01	Split between 2 compartments
Spillway Length (m)	10	Determined from Water Balance Storm Flows
Spillway Depth (m)	0.8	Determined from Water Balance Storm Flows
Spillway Side Slopes (-)	2	Confirmed in water balance
Pump Type	Land based pump with sump suction inlet	TSF Phase 2 Design
Pump Return rate (m^3/hr)	500	Based on 100% process water demand at lower bound slurry density. 210m^3/hr on BMM scope – minimum 340m^3/hr for system to be neutral
Pump Return water Line OD (mm) and Type	315mm	From onshore
Silt Trap		
Lining	Reinforced Concrete lined with railway tracks	TSF Phase 2 Design
No. of Compartments	2	TSF Phase 2 Design
Isolation Method	Sluice Gate	TSF Phase 2 Design
Compartment Effective Depth (m)	1.5	TSF Phase 2 Design
Compartment Effective Length (m)	25	TSF Phase 2 Design
Solution Trench/Stormwater Toe Drain		
Depth (m)	Up to 1.5m	Scope BMM
Bottom Width (m)	1.2	TSF Phase 2 Design
Side Slope (1V:?H)	1V:2H	TSF Phase 2 Design
TSF Decant		
Type	Skid Mounted pump, land based with floating suction inlet	TSF Phase 2 Design
Pump Type and Size	TBD	TSF Phase 2 Design
Delivery Pipe Diameter OD (mm) and Type	TBD	TSF Phase 2 Design
Deposition/Operation Method		
Wall Building Method	Cyclone	
Wall Building Material	Tailings Underflow	
Wall Building Maximum Height Thickness (mm)	1.5m	
Overall Slope (1V:?H)	3	
Crest Width (m)	2	

Mass Split of cyclone (UF%)	35%	
No of Cyclones per bank	9-11	Scope BMM
Cyclone size	250mm	Scope BMM and phase 1 size
Minimum operating pressure (kPa)	120	Scope BMM
Pond Control		
Minimum Operating Pond Depth (m)	0.5	Pump limitation
Minimum Pond Distance from Wall Crest (m)	200	TBC with final CSL/State parameter
Pool Control Method	Rotational Deposition	
Underdrainage/Seepage Control		
No. of Toe Drain Outlets – Above Liner	9	TSF Phase 2 Design
No. of Toe Drain Outlets – below Liner	9	TSF Phase 2 Design
Toe Drain Pipe Diameter (OD mm)	305	TSF Phase 2 Design
Internal perforated pipe Diameter (OD mm)	160	TSF Phase 2 Design
		TSF Phase 2 Design
Bench Decants		
No. of Bench Decants per Bench	TBD	
Pipe Diameter (OD) and Type	TBD	

3.0 CLIMATE AND HYDROLOGY

3.1 REGIONAL CLIMATE

The Gamsberg Zinc Mine site is located the Northern Cape Province approximately 11.7 km east from the town of Aggeneys. Gamsberg is in the Lower Orange Water Management Area, in Quaternary Catchment D82C. This catchment is known as an endorheic basin which, is a drainage basin that retains water and allows no outflow to other external bodies of water i.e., a river or the ocean. The site, along with the quaternary catchment, main rivers and non-perennial drainage lines are shown in Figure 3-1. The light blue drainage lines show that they all flow inward to the quaternary and that no drainage line flows out of the quaternary thus earning the endorheic status.

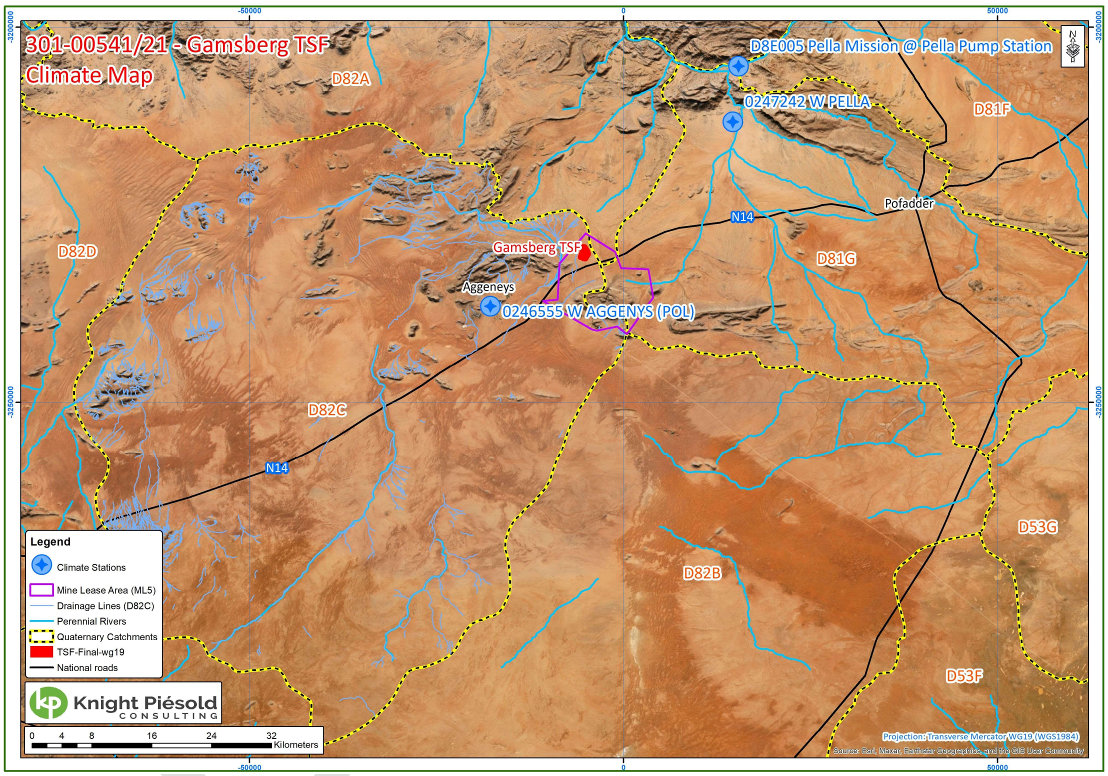


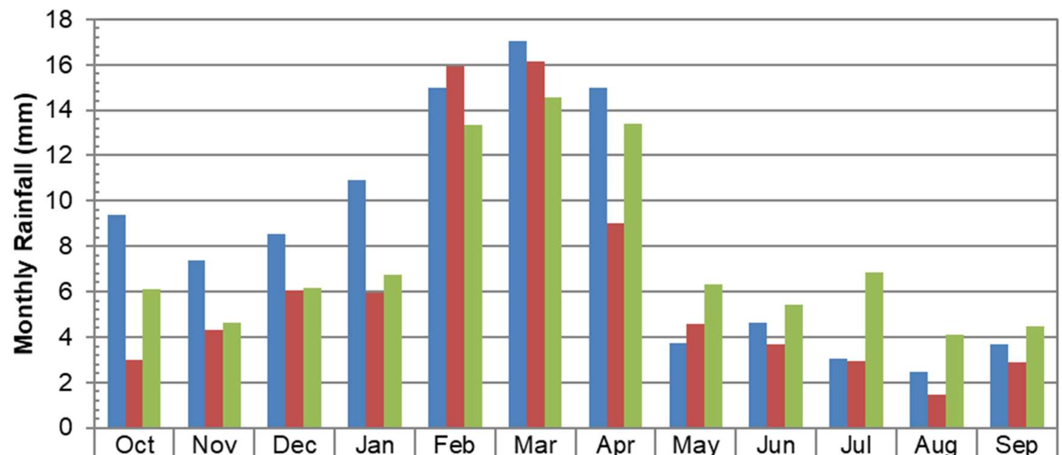
Figure 3-1 Regional locality of Gamsberg Zinc Mine

3.2 RAINFALL

The daily rainfall depths were extracted for the various SAWS rainfall stations in the area as shown in Figure 3-2 from the rainfall data extraction utility and the Department of Water and Sanitation's (DWS) website (Department of Water Affairs, 2008). The daily rainfall data extraction utility was developed by Richard Kunz, from the Institute for Commercial Forestry Research (ICFR), in conjunction with the former School of Bioresources Engineering and Environmental Hydrology (BEEH) at the University of KwaZulu-Natal, Pietermaritzburg, South Africa. The utility extracts observed and in-filled daily rainfall values from a database which was developed by Steven Lynch in the course of a Water Research Council (WRC) funded research project (K5/1156) awarded to BEEH (Kunz, 2004). Table 3-1 details the various rainfall stations used in this study.

Table 3-1 Rainfall stations considered in the study

Station	Number	Start Date End Date	No. of years data (years)	Missing data (years)	MAP (mm)	Altitude (m)
Pella Mission @ Pella Pump Station	D8E005	1983/10/01 2023/05/30	39.69	9.28	100.71	316
Pella	0247242 W	1901/01/01 2000/07/31	99.65	1.01	75.98	471
Aggeneys (POL)	0246555 W	1950/01/01 2000/07/31	50.61	0	92.22	825



	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep
D8E005	9.391	7.359	8.521	10.89	14.98	17.04	14.98	3.747	4.624	3.042	2.479	3.663
0247242_W	3.020	4.290	6.046	5.975	15.94	16.17	9.031	4.573	3.674	2.936	1.455	2.869
0246555_W	6.130	4.642	6.136	6.745	13.34	14.57	13.42	6.345	5.404	6.865	4.128	4.496

Figure 3-2 Monthly rainfall distribution for rainfall stations in Aggeneys area

From Figure 3-2 it can be seen that there is some inconsistency in the monthly rainfall data. The cumulative rainfall for the rainfall stations is shown in Figure 3-3. Based on the uniformity of the monthly rainfall distribution, the altitude and the reliability of the data, the 0246555 W Aggeneys (POL) Station was chosen as the most representative station for this study. This also happens to be the closest station to the mine.

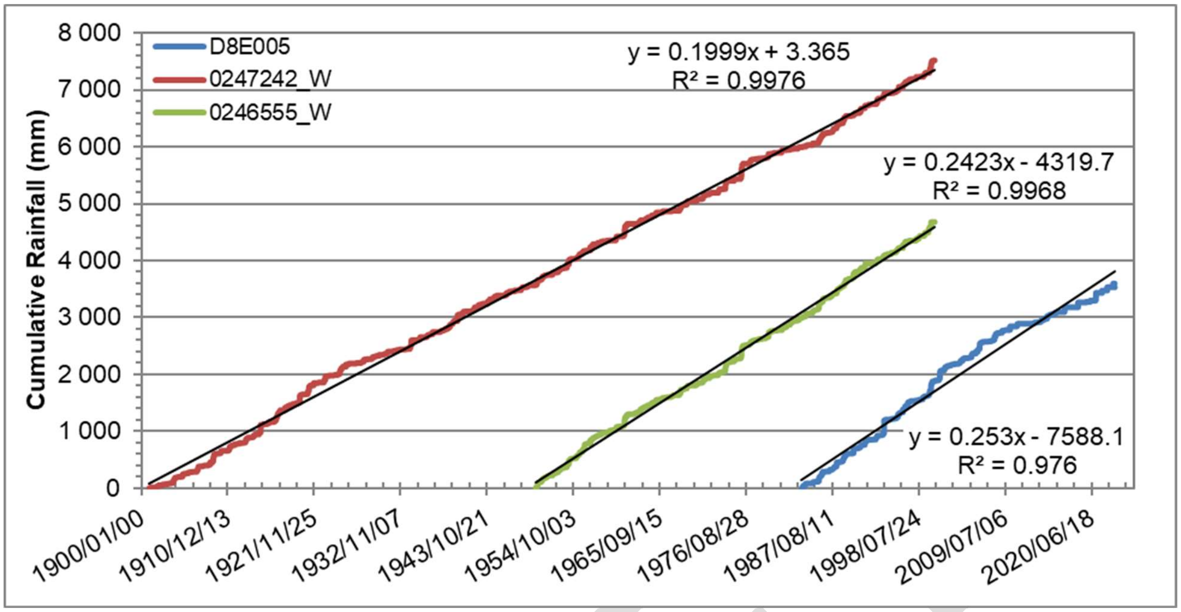


Figure 3-3 Cumulative rainfall for rainfall stations in Aggeneys area

Regression lines were fitted to the data in Figure 3-3 as an informal check on MAP estimates. These linear models show that with each incremental increase in time, the overall cumulative precipitation increases by some quantity, this is embodied in the slope parameter of the regression equations. These slope parameters smooth over the annual dry/wet season variability and give the long-term increase in cumulative precipitation with time. Taking these slope parameters and averaging them gives a value of 0.232, meaning that with each passing day, the cumulative precipitation quantity increases, on average, by 0.232 mm. Multiplying 0.232 mm/day by 365 days returns a value of 84.6 mm/year which can be interpreted as being a regionally averaged MAP for reference purposes. The slopes of the individual lines are not parallel to each other indicating that for the observed data across the region, the average increase in cumulative precipitation with time is not uniform. Further analysis is required for validation, but for this simple analysis, the consistency of the independently collected datasets is reassuring.

Figure 3-4, Figure 3-5 and Figure 3-6 show the daily rainfall, monthly boxplot and the annual rainfall for the 0246555 W Aggeneys (POL) Rainfall Station respectively.

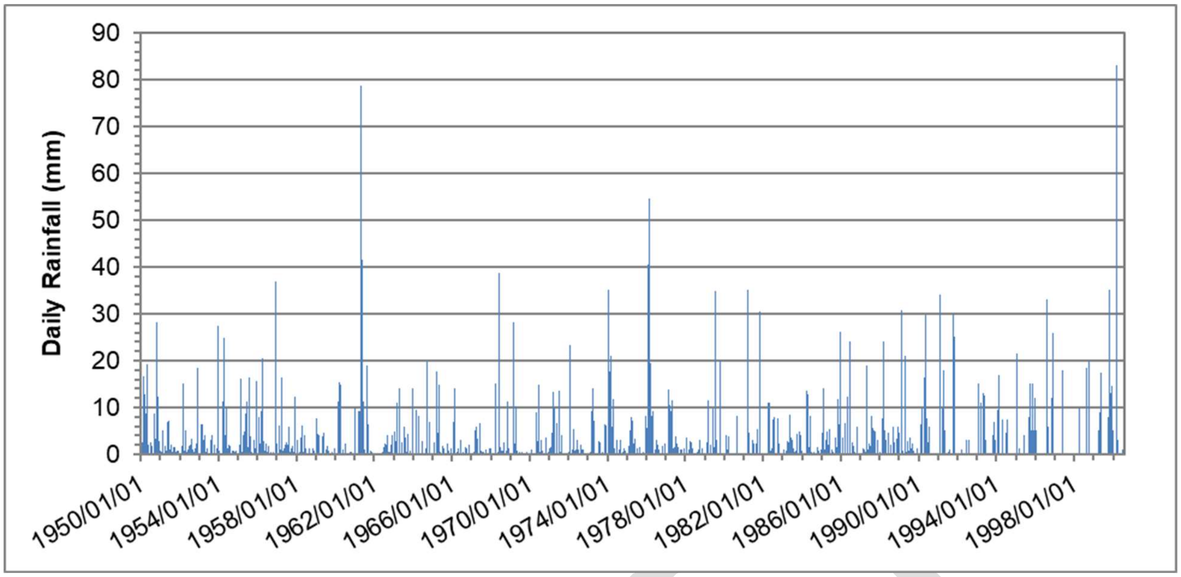


Figure 3-4 Daily rainfall for 0246555 W Aggeneys (POL) Rainfall Station

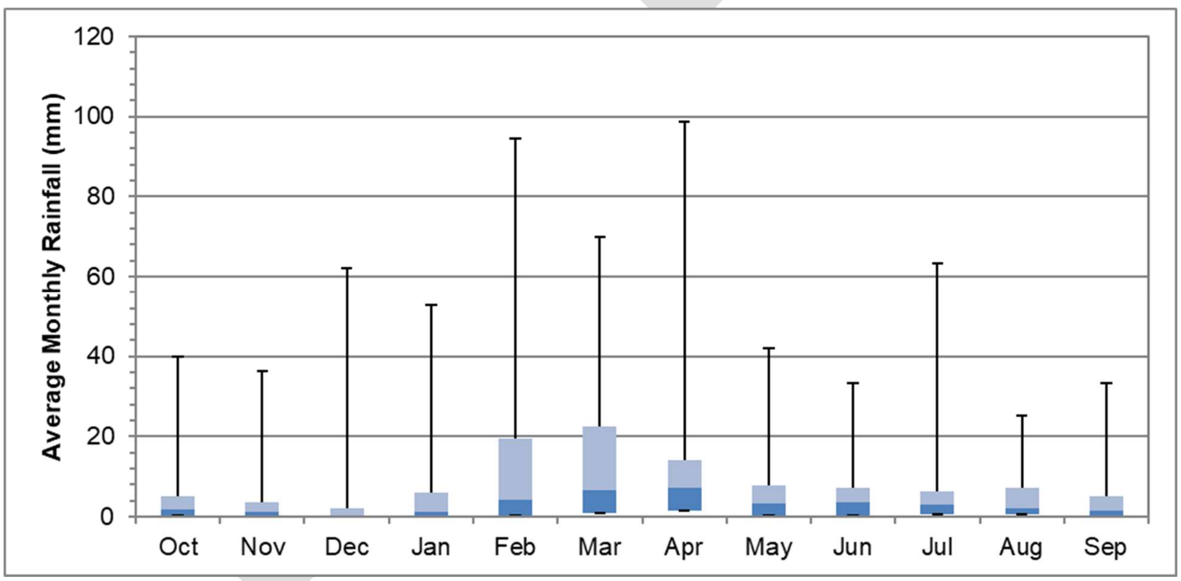


Figure 3-5 Monthly Rainfall boxplot for 0246555 W Aggeneys (POL) Rainfall Station

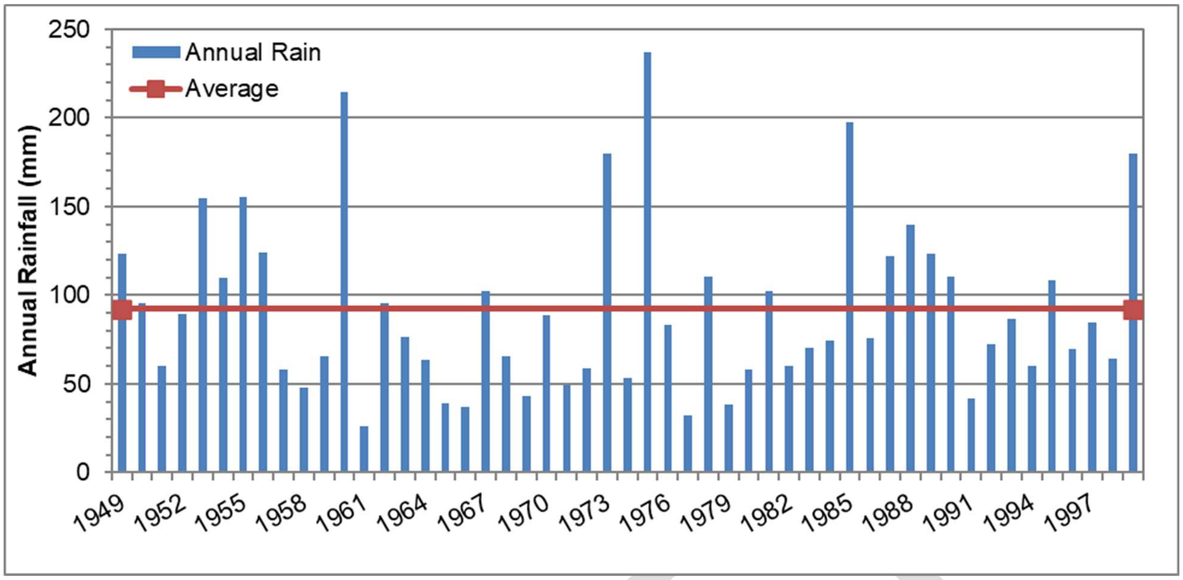


Figure 3-6 Annual Rainfall measured at 0246555 W Aggeneys (POL) Rainfall Station

The mean annual precipitation for 0246555 W is 92.22 mm. The lowest rainfall year was the 1961/1962 hydrological year with 26 mm and the highest rainfall year was the 1975/1976 hydrological year with 237.2 mm.

The 5th, 50th and 95th percentiles of the annual rainfall totals for 0246555 W is presented in Table 3-2. This table shows that for the area there was:

- Less than 37 mm/annum rainfall for 5% of the time;
- Less than 76 mm/annum rainfall for 50% of the time; and
- Less than 191 mm/annum rainfall for 95% of the time.

Table 3-2 5th, 50th and 95th percentiles of the annual rainfall totals

Station name	5th percentile	50th percentile	95th percentile
0246555 W	37.3	75.6	190.48

The rainfall data for 0246555 W shows that three events measured more than 50 mm/day and no rainfall event with more than 100 mm/day was recorded during the data period. Table 3-3 shows all the highest recorded rainfall events at the rainfall station.

Table 3-3 High rainfall events for 0246555 W Rainfall Station

Maximum recorded daily rainfall (mm)	Date of maximum rainfall
78.8	1961/04/08
54.6	1976/02/04
83	2000/02/19

The 24-hour rainfall depths for the 1 in 2, 1 in 5, 1 in 10, 1 in 20, 1 in 50, 1 in 100, 1 in 200, 1 in 2 457, 1 in 5 000 and 1 in 10 000 recurrence intervals at the station were calculated from the data available. To determine the likely magnitude of storm events, a statistical approach, using the Reg Flood program (Alexander, et al., 2003) was applied to the available recorded daily rainfall depths. The maximum 24-hour rainfall depth for each year was analysed. This method statistically analyses the maximum daily rainfall depths for each year to determine the different recurrence interval daily rainfall depths. The best fit for the rainfall station was the Log Pearson 3 distribution which resulted in the 24 h storm rainfall depths summarised in Table 3-4. The data used for these storms along with the distribution curve can be found in Appendix A.

Table 3-4 24-hour rainfall depths for different recurrence intervals in mm/day

Recurrence interval (years)	0246555 W
1 in 2	20
1 in 5	32.53
1 in 10	42
1 in 20	51
1 in 50	62
1 in 100	70
1 in 200	78
1 in 2 457	107.54
1 in 5 000	115.87
1 in 10 000	124

The Probable Maximum Precipitation for the 0246555 W station is calculated as 228.41 mm.

3.3 POTENTIAL EVAPORATION

Evaporation data was retrieved from the DWS website for the D8E005 station since it is the most recent daily recorded data available. This station had both a Symmons Pan (S-Pan) and A-Pan evaporation pans installed in 1983 however the S-Pan was removed in 2019. In addition, the S-Pan data is missing multiple years of data as shown in Table 3-5. Since both A-Pan and S-Pan data were recorded at this station,

Table 3-6 shows the monthly evaporation for both the A-Pan (which is converted to S-Pan data) and the S-Pan data. The data used in this study was the A-Pan data since the A-Pan data has a longer record and the mean annual evaporation (MAE) is approximately 110 mm less than the recorded S-Pan data and thus more conservative.

Table 3-5 Evaporation data from D8E005 station

Station	Number	Start Date End Date	No. of years data (years)	Missing data (years)	Usable data (years)
Pella Mission @ Pella Pump Station A-Pan	D8E005	1983/09/01 2023/05/30	39.77	2.73	100.71
Pella Mission @ Pella Pump Station S-Pan	D8E005	1983/09/01 2019/04/29	35.68	13.26	75.98

Table 3-6 Evaporation data

Evaporation (mm)	OCT	NOV	DEC	JAN	FEB	MAR	APR	MAY	JUN	JUL	AUG	SEP	MAE
A-Pan	398	452	502	508	432	399	292	209	148	165	218	295	4019
S-Pan (recorded)	341	392	447	457	381	349	241	167	117	134	179	245	3450
S-Pan (converted A-Pan)*	334	381	426	431	363	335	241	167	114	129	176	243	3339
Open water	270	312	353	362	320	295	212	146	97	107	142	197	2813

*This data was converted to S-Pan by means of conversions as detailed by Bosman (1990)

Figure 3-7 shows the monthly rainfall for 0246555 W Aggeneys (POL) and the open water evaporation for the D8E005 Pella Mission @ Pella Pump Station A-Pan data.

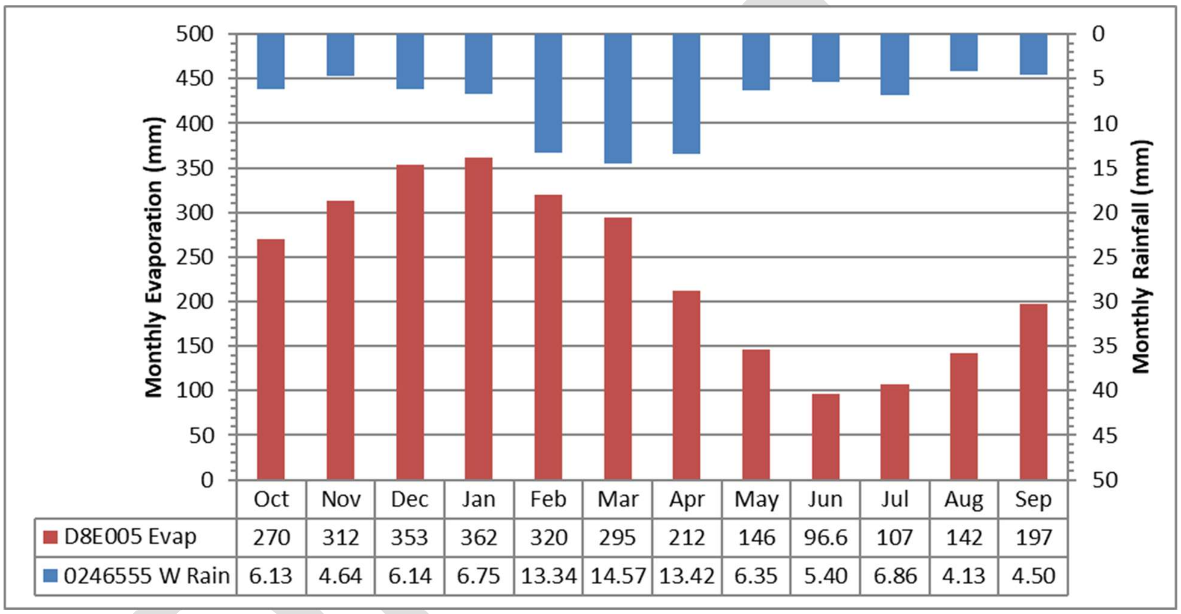


Figure 3-7 Monthly rainfall and evaporation for 0246555 W Station and D8E005 Station

4.0 GEOTECHNICAL INVESTIGATION

4.1 INTRODUCTION

A geotechnical investigation was carried out to provide an understanding of geotechnical conditions at the proposed TSF and RWD sites for design purposes. This includes the nature and extent of the underlying soils and rock, provide foundation recommendations and comment on the reuse of material for construction purposes.

A geotechnical report was compiled to document the results of the investigation which includes the desktop study, surface geophysical survey, test pitting and rotary core drilling. This final interpretative report includes all site investigation data and the laboratory results. The evaluation of the geotechnical conditions and subsequent recommendations take all the geotechnical data into account, including the latest laboratory test results. This geotechnical report along with the commensurate appendices showing test pit logs and other results is contained within Appendix B of this report.

4.2 AVAILABLE INFORMATION

KP conducted a geotechnical investigation in April 2017 as part of the Detailed Design and Construction Quality Supervision of the TSF Phase 1. Refer to report: Gamsberg Mine, New Tailings Storage Facility Geotechnical Investigation (Phase 1) Final Report, Knight Piésold, Report No. 2374 (2017) [1] 1.

The Phase 1 investigation comprised the excavation of thirty-three test pits (designated P1 to P33) and two boreholes (designated BH1 and BH2). The test pits were excavated to refusal depth of a 20-Ton excavator and logged in situ by a registered engineering geologist according to standard practice. The test pit results are summarized in Table 1 and the typical profile was recorded as follows:

- Aeolian silty sand covers the site to a maximum depth of 0.4 m.
  Calcrete horizons at various stages of development and cementation occur within the residual soils below the surface aeolian soil layer. Nodular calcrete and honeycomb calcrete occur to a maximum depth of 2.5 m.
- Hardpan calcrete occurs as a very dense very strongly cemented sandy gravel layer where excavator refusal conditions were met.
- Hardpan ferricrete occurs within the TSF extension area as very dense very strongly cemented sandy gravel to a maximum depth of 2.3 m.
- Below the pedogenic soils, very soft rock gneiss, retrieved as silty sandy gravel, occurs from an average depth of 0.8 m.
- Excavator refusal occurred at depths of between 1.0 m and 3.1 m, in all test pits, where pedocretes were not present or where they were poorly developed and could be excavated through.
- No ground water seepage was observed in any of the test pits.

4.3 GEOTECHNICAL SCOPE OF WORKS

The scope of work for the detailed design level geotechnical investigation was set out in KP's proposal

The investigation comprised a desk study of available information and site walk over to note any salient features. A geophysical investigation was carried out to provide additional information regarding the depth to bedrock and variation within the soil and rock profile.

The geotechnical assessment investigated the subsurface conditions through excavation of test pits along the TSF wall alignment and within the RWD footprint, using a 20-Ton excavator. Rotary core boreholes were allowed to investigate ground conditions at the proposed decant tower and to investigate geotechnical conditions at depth, in areas of adverse geotechnical conditions, if found.

The design programme had been modified by the design team prior to the commencement of the investigation to change the previously envisaged penstock to a floating structure, thus no decant tower is planned or required geotechnical investigation.

In addition to the geotechnical conditions on site, the characterization and determination of the critical state line (assuming 1 type of fine tailings) and determination of interface shear strength between the tailings, clay geosynthetic liner and geomembrane was required.

Laboratory testing was carried out in accordance with the geotechnical conditions encountered on site.

This report serves as an interpretive report to present the site investigation results, as well as subsequent evaluation of the geotechnical conditions, conclusions, and recommendations.

4.4 SITE DESCRIPTION

Gamsberg Mine is located in the Namakwa District of the Northern Cape Province. The mine occurs approximately 11 km east of Aggeneys town and approximately 45 km west of Pofadder town, south of the N14 national road. Refer to the Site Locality Plan shown in Figure 1-2.

The investigation is conducted on the TSF facility north of the Gamsberg mine and N14 road. Two areas of investigation are detailed in this report, namely the Phase 2 TSF site that is approximately 130 ha in size, directly north of the existing Phase 1 TSF, and the new RWD, approximately 5 ha in extent southwest of the existing TSF and west of the existing RWD.

The sites are undeveloped with minor borrow activities in the TSF footprint area. They comprise arid short, scattered shrubs and limited grass cover. The co-ordinates of the central point of the TSF and RWD sites are 29°10'55.59"S 18°56'49.34"E and 29°12'5.38"S 18°56'33.20"E, respectively.

The site topography is relatively flat, sloping gently downwards from the north to south, from 960mamsl to 948 mamsl. A seasonal river drains the area in a southerly direction, approximately 2.8 km west from the TSF site with visible sheet wash features draining towards the river. No ground water seepage was observed during the investigation, and a few rock sub-outcrops were encountered during excavation of test pits.

4.5 GEOLOGY AND SOIL

4.5.1 REGIONAL GEOLOGY

The mine lies on the Bushmanland Group and Gladkop Metamorphic Suite of the Mokolian age. The Bushmanland group is structurally complex with a poly-metamorphosed geology, it predominantly comprises sedimentary and volcanic rocks of the Khurisberg, Aggeneys and Kamiesberg Subgroups. The rocks of the area have been intensely foliated with a highly variable orientation, dipping between 10˚ and 80˚ in various directions. Refer to Figure 2 within Appendix B for an excerpt of the regional geology map.

These rocks are overlain by Quaternary deposits comprising sand, scree, rubble and sandy soils. From aerial imagery, the rock contacts between rock types are not clearly defined.

4.5.2 SITE GEOLOGY

According to the published 1:250 000 scale Geological series map sheet 2918 Pofadder [3], the sites are underlain by calc-silicate gneiss, schist, amphibolite and minor lenticular quartzite belonging to the Wortel Formation, Aggeneys Subgroup and leucogneiss belonging to the Koeipoort Gneiss, Gladkop Metamorphic Suite.

The Quaternary sand deposit covers the majority of the sites and occurs as a thin surface layer. Observation of aerial imagery of the site indicates several approximately east-west striking lineaments. The transported and in situ residual soils include variable degree of pedogenic deposits as described in Section 4.5.3 below.

4.5.3 CLIMATE AND WEATHERING

Climate determines the mode of weathering and rate of weathering. The effect of climate on the weathering process (i.e., soil formation) is determined by the climatic N-value defined by Weinert. The climatic N-value is greater than 30 for this site, which indicates mechanical disintegration is the dominant mode of weathering with no secondary minerals development. This typically results in thin residual soil profiles of coarse gravel developed from the disintegrating rock.

Residual soils in these climatic environments often undergo various degrees of pedogenic cementation, such as calcification. Calcrete is a pedogenic soil that is produced by the cementation of calcium carbonate ($CaCO_3$). Various development stages of calcrete can occur, depending on the degree of cementation. These deposits are often erratically deposited leading to variable ground conditions over short distances. Furthermore, these cemented horizons may lead to excavation difficulties.

4.6 SEISMICITY

South Africa is located on the African Tectonic Plate which, in comparison to other tectonic plates, is fairly stable with low degrees of movement. Much of the African Plate, except the East African Rift Zone, is considered to be a zone of low tectonic activity. This does not suggest that no seismic activity occurs but rather that the probability of some is much lower. Seismic hazard is represented by the peak horizontal ground acceleration (PGA) of any particular area: the greater the PGA the greater the probability of seismic activity.

The image below provides the indicative seismic risk across Southern Africa and the corresponding peak ground accelerations with a 10% probability of exceedance within a 50-year period. The PGA on site is indicated to be approximately 0.08g which equates to a “V” Degree classification on the Modified Mercalli Scale.

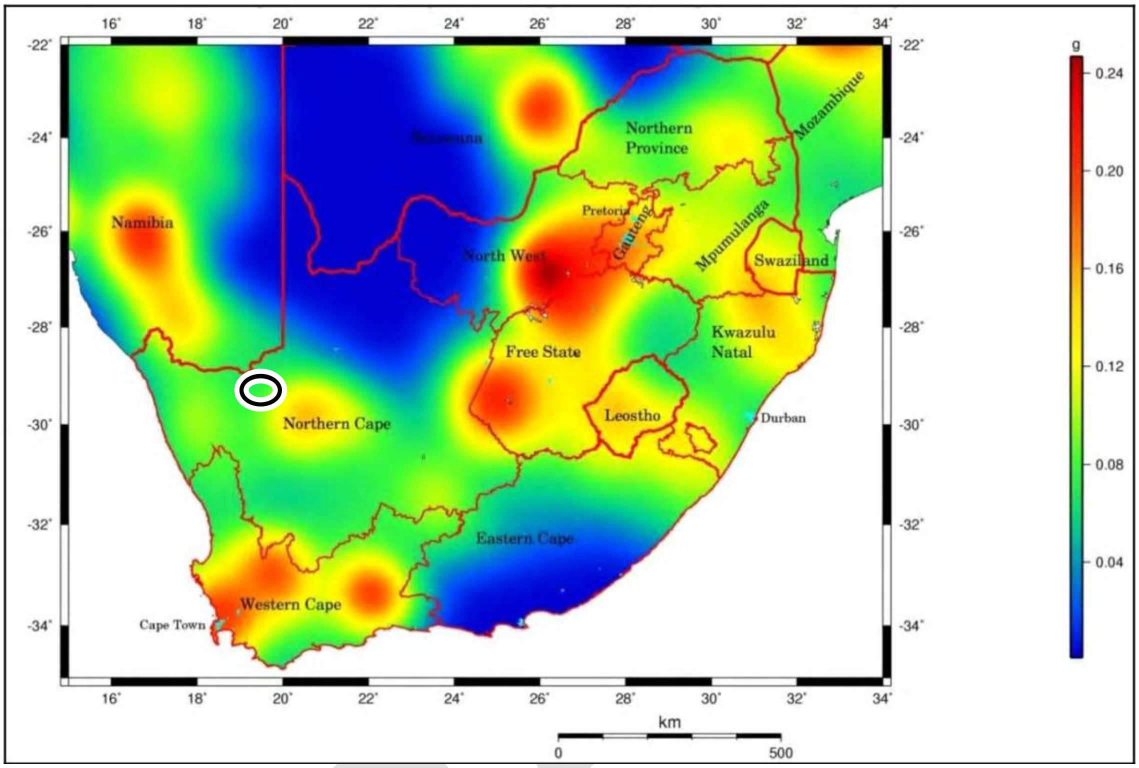


Figure 4-1: Seismicity Map of South Africa

In addition to the regional seismic data, a site-specific study was carried out for the Gamsberg Zinc Mine as documented in Knight Piésold’s Memorandum Letter reference RI21-00398 dated 11 November 2022. The pertinent information from that document is included below:

- The mine site is located in a region of low seismicity, typical of an intra-plate region, characterised by generally low levels of seismic activity. Higher seismicity zones such as the Witwatersrand Basin and the Ceres Cluster are more than 500 km away from the mine site.
- The most significant source for potential earthquakes within 200km radius of the Gamsberg Zinc Mine site is the Pofadder Shear Zone, a NW-trending shear zone approximately 500km long located ~35km northeast of the site in southern Namibia and northwestern South Africa.
- Higher seismicity zones such as the Witwatersrand Basin and the Ceres Cluster are more than 400 km away from the mine site.
- Earthquake ground motion parameters (PGA and spectral accelerations) have been provided for site conditions with V_{s30} values of 760 m/sec and 360 m/sec (corresponding to very dense soil or soft rock). Seismic coefficients are provided for simplified (screening level) seismic stability analyses for the TSF, and have been calculated for a range of AEP values (1:475 to 1:10,000) for foundation conditions represented by V_{s30} values of 760 m/sec and 360 m/sec.

- The PGA obtained for the Gamsberg project site probabilistic seismic hazard analysis for the return period of 475 years, 2,475 years, and 10,000 years are 0.02g, 0.05g, and 0.12g for the site based on a V_{s30} value of 760m/s, while a PGA of 0.03g, 0.07g, and 0.15g were obtained for the same period with a shear wave velocity of 360m/s. A 50% to 25% increase (amplification) in PGA is observed for a V_{s30} decrease from 760m/s to 360m/s.
- For an annual exceedance probability of 1:10,000 the TSF, the horizontal seismic coefficient is 0.06 and 0.075 (50% of the PGA of 0.12g and 0.15 g) for V_{s30} values of 760 m/sec and 360 m/sec respectively.
- It is recommended that a dynamic site response analysis is undertaken to determine the amplification of ground motions as seismic waves propagate through the foundation soils and the TSF. Measuring the site seismic waves propagation can be done using either surface-wave survey, refraction survey, down-hole survey, and cross-hole survey.

4.7 METHOD OF INVESTIGATION

4.7.1 GEOPHYSICAL SURVEY

The geophysical survey was conducted by GeoFocus (Pty) Ltd in November 2022 as part of this investigation, and comprised two perpendicular traverses, one orientated in north-south direction and the other in an east-west direction. The traverses were conducted on the TSF site and the orientations were selected to intersect anticipated lineament orientations perpendicularly, to provide the best resolution of these structures, if present. Three different geophysical methods were undertaken across each traverse, namely:

- 3 500m of Electric Resistivity Tomography (ERT): to indicate resistivity variations within the soil/ rock profile, which often represent variations in geotechnical conditions, geological structures like faults and intrusions as well as zones of weathering that can influence ground water flow dynamics.
- 3 040m of Seismic Refraction (SRF) to indicate variation in the seismic velocity of the soil/ rock profile, which is generally related to density and strength. A 10m geophone spacing was used to provide information to aid the delineation of bedrock depth.
- 3 090m of Multichannel Analysis of Surface waves (MASW) to indicate variation in shear wave velocity of the soil/ rock profile, which can be correlated to stiffness parameters. A 5m geophone spacing was used to provide stiffness parameters and an understanding of the soil/ rock behaviour under strain.

The geophysical data was used to position the test pits and boreholes to ensure areas of significant variation or detrimental geotechnical conditions were not overlooked. The geophysical data was also used to extrapolate the data from the point information (test pits and boreholes) across the site. The geophysical report is presented in Appendix A and is evaluated in conjunction with the other results below.

4.7.2 TEST PITS

The investigation of the shallow geotechnical conditions comprised test pits excavated from 30 November to 11 December 2022. The positions were selected to provide representative information along the TSF wall to supplement the existing basin information and within the RWD to provide information regarding the foundation conditions. The test pits were excavated using either a 320D or 330D excavator machine (provided by Fraser Alexander) to refusal condition, at a maximum depth of

3.9 m and 3.6 m at the TSF and RWD, respectively. A total of 41 test pits were excavated for the project of which 33 test pits (TP201 to TP233) were excavated at the TSF site and 8 test pits (RWDTP1 to RWDTP8) at the RWD.

The test pits were profiled and photographed in-situ by an engineering geologist according to current practice. The TSF test pit soil profiles are presented in Appendix B.

The positions of the test pits were recorded using a hand-held GPS with an accuracy of approximately 3m. The coordinates of these positions are displayed on the test pit profiles in WGS84 datum and South African Grid (Lo19).

4.7.3 ROTARY CORE DRILLING

Two rotary core boreholes (TSFBH1 to TSFBH2) were drilled at the TSF site and one rotary core borehole (RWDBH3) was drilled at the RWD, by the drilling contractor Geomechanics, between the 3 and 12 December 2022. The boreholes were drilled into sound bedrock to depths ranging between 11.5m and 14.13m below surface level.

Standard Penetration Tests (SPTs) were carried out where suitable conditions were encountered, however refusal occurred with all SPTs attempted.

The core boxes were photographed and logged by a KP engineering geologist according to standard practice. The positions of the boreholes were recorded using a hand-held GPS with an accuracy of approximately 3m. The coordinates of these positions are displayed on the borehole logs in South African Grid (Lo19) format and WGS84 datum. The rotary core borehole profiles are summarised in Table 4-1, while the full profiles and core photographs are presented in Appendix B.

Table 4-1: Summary of Boreholes

BH No.	TOTAL DEPTH (m)	THICKNESS OF LAYERS (m) - (m)							
		TRANSPORTED SOILS				ROCK			
		FILL	Alluvium (*Calcified)	Silicified Alluvium	Calcrete Cobbles and Boulders	Gneiss			
						Very soft rock	Soft rock	Hard rock	Very hard rock
TSFBH1	11.5	-	0 - 1.30	-	-	1.3 - 6.14	6.14 - 11.5	-	-
TSFBH2	6.86	0 - 1.3	-	-	-	-	-	1.3 - 1.75	1.75 - 6.86
RWDBH3	14.13	-	0 - 0.3 *0.85 - 1.5	1.5 - 12.15	0.3 - 0.85	-	12.15 - 14.13	-	-

4.7.4 LABORATORY TESTING

Disturbed soil samples were collected from representative horizons at the TSF and RWD. The samples were submitted for testing at Specialised Testing Laboratory in Pretoria.

Representative disturbed tailings samples were also taken on the southern and western wall of the existing TSF on site. These samples were also tested at Specialised Testing Laboratory in Pretoria. The full laboratory results presented in Appendix C.

4.8 INVESTIGATION RESULTS

4.8.1 GEOPHYSICAL SURVEY

The geophysical survey undertaken comprised seismic refraction, MASW and electrical resistivity tomography. The results are discussed below.

Seismic surveys are generally aimed at mapping the depth to bedrock, and through correlation of the seismic velocities of the different layers encountered. These can be correlated with rock mass properties (e.g., load bearing capacity) with depth. MASW surveys provide shear-wave velocities (Vs), which can be corelated to stiffness parameters allowing for the modelling of strain. ERT is mostly aimed at mapping geological structures like faults and intrusions as well as zones of weathering that can influence ground water flow dynamics. ERT is also used to map the transition to hard rock where a change in resistivity provides sufficient resolution.

Two lines were surveyed perpendicular to each other over the centre of the TSF site where line 1 was surveyed in an east-west orientation and line 2 in a north-south orientation. The traverses were positioned to intersect anticipated lineament orientations perpendicularly, to provide the best resolution of these structures.

The geophysical report describes the presence of a loose surface soil overlying weathered rock at variable and shallow depths. The contact between the calcified surface soils and weathered bedrock is gradual and not distinct. It is anticipated that sound bedrock is found from 5m to 10m below surface. Notable subvertical joints/discontinuities are noted generally towards the ends of the surveyed sections as substantial, lateral, resistive heterogeneity within the bedrock, dominant along the E-W line 1 compared to N-S line 2, which may be indicative of a dominant N-S structural orientation and preferential weathering along linear features. A deeper weathered profile is indicated in the central portion of the site, at and south of the survey line intersection as shown clearly by the SRF N-S section (line 2 between 6770250 N and 6769975 N).

4.8.2 TYPICAL SOIL PROFILE

4.8.2.1 TAILINGS STORAGE FACILITY

The TSF comprises two typical profiles, the more common along the north and eastern sides of the site includes a thin colluvium layer (<0.3 m thick) overlying scattered calcrete and shallow very soft to soft rock gneiss from depths as shallow as 0.1 m in places. The common profile over the central and western parts of the TSF site comprises thin alluvium (<0.4 m thick) overlying gneiss bedrock from as shallow as 0.1 m in places.

In general, the TSF is underlain by shallow (from a depth of 0.4 m to 1.6 m) bedrock of varying rock type bands comprising gneiss, granite-gneiss, schist, amphibolite and quartzite. Excavator refusal was encountered between surface and depths of 3.9 m in all excavated test pits at the TSF.

Where present, along the northern perimeter of the existing TSF, the fill is typically less than 0.4 m thick overlying calcrete on the east and western border.

4.8.2.2 RETURN WATER DAM

The typical profile at the return water dam is that of calcrete and silcrete in varying proportions, indurating alluvial sand and gravel above the bedrock. The typical profile across this relatively small area is detailed below.

Table 4-2: Typical RWD profile

	Depth	Description
	0-0.1 m	Dry, orange brown, very loose to loose, silty fine to coarse sand, alluvium.
	0.1-0.6 m	Dry, white and orange, medium dense to dense, nodular, honeycomb and hardpan calcrete gravel, cobbles and boulders.
	0.6-1.2 m	Dry, white, loose to medium dense, intact silty sandy gravel, nodular calcrete. Calcrete often silicified as well but predominantly calcified.
	1.2 m +	Dry, orange brown, medium dense becomes very dense with depth, silty fine to coarse sand, silicified alluvium.
	2.6-3.6 m	Refusal in all TP's (except RWDTP5) in silicified alluvium. RWDTP5 refused on quartzite/ quartzite pegmatite.

4.8.3 LABORATORY TESTING

Laboratory testing was carried out on representative samples to define the material properties in the TSF and the RWD to provide information for the design.

Foundation indicators, Proctor compaction, California Bearing Ratio, remoulded shearbox, remoulded permeability, chemical dispersivity, double hydrometer, Basson Index and pH and electrical conductivity tests were performed on representative samples.

4.8.3.1 TSF GENERAL

The material tested at the TSF typically comprises alluvium (calcified or ferruginous) or soft rock gneiss with one fill sample being included. The colluvium encountered on site was limited to less than 0.3 m in thickness.

The fill found along the existing TSF northern perimeter comprises slightly plastic sandy gravel with a grading modulus (GM) of 2.14.

4.8.3.2 TSF ALLUVIUM

The alluvium comprises coarse, calcified and ferruginous alluvium to shallow depths of less than 0.7 m, it was retrieved as sandy gravel to gravelly sand, with a fines (silt and clay) percentage of less than 13% and plasticity index (PI) of less than 11%. The Unified Soil Classification system categorises the material as SC to SM. The pH is slightly basic at 7.9 and the electrical conductivity of 0.038 S/m indicates corrosive soils.

Standard Proctor compaction results indicate a maximum dry density (MDD) at proctor compaction of between 1934 kg/m^3 and 1956 kg/m^3 with an OMC of 10.7%. The California Bearing Ratio is 16% to 18%, at 93% Mod AASHTO density. The material classifies as a G7 quality material (COTO, 2020).

The calcified alluvium was tested for remoulded shear strength parameters and permeability. The shear box test conducted at 93% MDD of Standard Proctor Compaction revealed internal friction (φ) of 38° and cohesion (c) of 0 kPa. The coefficient of permeability is 1.38x10^{-8} m/s for the remoulded calcified alluvium. The ferruginised coarse alluvium revealed φ=36° and c=13 kPa. The cohesion is anticipated to be an apparent cohesion generated due to the particle interlock and should be cautiously used in calculations.

4.8.3.3 TSF SOFT ROCK GNEISS

The soft rock gneiss is excavated/ retrieved as a soil comprising slightly to non-plastic sandy gravel or gravelly sand with a fines percentage of less than 5% and a grading modulus (GM) of more than 2.11.

The pH is slightly basic at 8.3 and the electrical conductivity of 0.021 S/m indicates corrosive material.

Laboratory results indicate a maximum dry density (MDD) at standard Proctor compaction of approximately 2004 kg/m^3 with an OMC of 9.4%. The California Bearing Ratio is 28%, at 93% Mod AASHTO. The material classifies as a G6 quality material (COLTO, 1998).

The soft rock gneiss was also tested for remoulded shear strength parameters and permeability. The shear box test conducted at 93% MDD of Standard Proctor Compaction revealed remoulded friction (φ) of 41° and cohesion (c) of 0 kPa and permeability of 1.26x10^{-8} m/s.

4.8.3.4 RWD GENERAL

The site is generally underlain by alluvium silicified above the calcrete horizon overlying the gneiss bedrock at depth.

4.8.3.5 RWD ALLUVIUM

The alluvium includes silicified alluvium with a typical component of calcified soil within the shallow profile. The alluvium comprises sandy gravel to gravelly sand with a fines (silt and clay) percentage of less than 13% and plasticity index of less than 15%. The Unified Soil Classification system categorises the material as SC to SM. The pH is slightly basic at 8.3 and the electrical conductivity of 0.323 S/m which indicates very corrosive soils.

Laboratory results indicate a maximum dry density (MDD) at standard Proctor compaction between 1832 kg/m^3 to 1920 kg/m^3 with an OMC of 11.7% to 13.6%. The California Bearing Ratio is 13% to 19%, at 93% Mod AASHTO. The material classifies as a G7 to G8 quality material (COLTO, 1998).

The calcified alluvium was tested for remoulded shear strength parameters and was the same as for the RWD with remoulded friction (φ) of 38° and cohesion (c) of 0. The silicified alluvium tested weaker and more permeable than the calcified alluvium with remoulded friction (φ) of 36° and cohesion (c) of 0 kPa and coefficient of permeability 5.79x10^{-7} m/s.

4.8.3.6 RWD PEDOCRETE

The pedocrete includes nodular calcrete and honeycombed with occasional calcrete boulders. The pedocrete comprises sandy gravel to gravelly sand with fines (silt and clay) percentage of less than 22% and plasticity index of less than 18%. The Unified Soil Classification system categorises the material as SC to GM-GC. The pH is slightly basic at 8 and the electrical conductivity of 0.191 S/m indicates very corrosive soils.

Laboratory results indicate a maximum dry density (MDD) at standard Proctor compaction of 1803 kg/m^3 with an OMC of 15%. The California Bearing Ratio is 15%, at 93% Mod AASHTO. The material classifies as a G7 quality material (COLTO, 1998).

The nodular calcrete was similar to the calcified alluvium when tested for remoulded shear strength parameters. The shear box test revealed 93% MDD of Standard Proctor Compaction remoulded friction (φ) of 38° and cohesion (c) of 0 kPa.

4.8.3.7 CHEMICAL TESTS

4.8.3.7.1 Dispersivity

Chemical dispersion and Double hydrometer tests were carried out to evaluate the soil based on the propensity of the material to erode pipes and gullies due to soil dispersivity. The material underlying the TSF is considered non-dispersive but the material underlying the proposed RWD is considered dispersive [6].

4.8.3.7.2 Aggressivity

The chemical test (Basson Index) was conducted on the soil samples from the site to determine the aggressiveness towards concrete and corrosivity toward steel. The result indicates the following:

Table 4-3: Chemical Test (Basson Index)

Basson Parameter	TP205/1	TP205/2	TP215/1	RWDTP3/3	RWDTP7/2	RWDTP7/3
Material type	**Calcified Alluvium**	**Soft rock Gneiss**	**Ferruginised coarse Alluvium**	**Calcified Alluvium**	**Nodular Calcrete**	**Silicified Alluvium**
pH of the sample (corrected at 20° C)	8.5	9.0	9.9	7.8	7.8	7.8
the Langelier Index for the sample	0.0	-0.1	-0.2	0.3	0.4	0.5
Ryznar Index for the sample	8.4	9.1	11.9	7.6	7.5	7.4
corrosivity ratio	6.7	2.8	4.8	262	248	307
Aggressiveness Index (Nc)	322	443	1799	-717	-855	-889
Aggressiveness	Aggressive	Aggressive	Aggressive	Aggressive	Aggressive	Aggressive
Corrosive (steel)	Aggressive	Aggressive	Aggressive	Aggressive	Aggressive	Aggressive
Overall aggressiveness towards concrete	Mild to Moderate	Mild to Moderate	Very High	None to mild	None to mild	None to mild

The table below provides an interpretation for the above results.

Table 4-4: Interpretation of Chemical Test Results

Index	Aggressive	Neutral	Non-Aggressive
Stability Ph, (Ph)	7 < pH	7 = pH	7 > pH
Langelier Index	Negative Value	Zero	Positive Value
Ryznar Index	>7.5	6 - 7	< 6
Corrosivity towards steel	>0.2		

The following table provides guidelines for assessing the overall Aggressiveness (N_c).

Table 4-5: Aggressiveness Guidelines

N_c	Aggressiveness
Less than 300	None to mild
400 – 700	Mild to Moderate
800 – 1000	High
= or > 1 100	Very High

The TSF soils vary from mild to very highly aggressive towards concrete and are corrosive towards steel. The RWD soils are generally not aggressive to mildly aggressive toward concrete and corrosive towards steel.

4.9 GEOTECHNICAL EVALUATION

4.9.1 EXCAVATABILITY

The excavation characteristics of different soil horizons on site have been evaluated according to SANS 1200D which details the standardised classification for earthworks excavations. The excavation class descriptions can be described as follows:

- “Soft Excavation”: Excavation in material that can be efficiently removed by a back acting excavator of flywheel power approximately 0.10 kW per millimetre of tined-bucket width, without the use of pneumatic tools such as paving breakers.
- “Intermediate Excavation”: Excavation in material that requires a back-acting excavator of flywheel power exceeding 0.10 kW per millimetre of tined-bucket width or the use of pneumatic tools before removal by equipment equivalent to that specified for soft excavation.
- “Hard Rock Excavation”: Rock that will be very difficult to excavate with an excavator and may require blasting, splitting and/or the use of rock breaking equipment, typically from medium hard to hard rock.

The test pits on the TSF site were excavated using an excavator. Across the site, excavations predominantly classify as “soft excavation” to a typical depth of 0.4 m. However, “intermediate” conditions were observed to an average depth of 1.6 m below surface. Local variations have indicated bedrock and “intermediate” to “hard rock” excavation conditions are anticipated from a depth as shallow as 0.1 m but typically from between 0.1 m to 0.8 m in the north between TP219 and TP226. “Intermediate” to “hard rock” excavation is generally deeper than 1.5 m along the northern boundary of the existing TSF and from 2.0 m in the southern portion of the TSF extension area. The TSF basin is variable and “hard rock” conditions are found from 0.35 m to 2.7 m below surface.

4.9.2 REUSE OF MATERIALS

4.9.2.1 TSF

The alluvium at the TSF may be suitable for reuse as a G7 quality material, although this material is non-dispersive it is classified as aggressive towards concrete and corrosive towards steel. The material has a low PI (<11%) and may be reused. The alluvium when remoulded is capable of achieving φ-values of 38° and cohesion of 0 kPa. The permeability is in the order of 10^{-8} m/s.

The colluvium and calcrete is limited in distribution across the site and was thus not considered for bulk earthworks. It is anticipated that the calcrete may be suitable for reuse.

The soft rock gneiss is non-dispersive but aggressive towards concrete and corrosive towards steel. This material may be suitable for reuse as G6 quality material. The soft rock gneiss when remoulded is capable of achieving φ-values of 41° and cohesion of 0 kPa. The permeability is in the order of 10^{-8} m/s.

4.9.2.2 RWD

The alluvium at the RWD was found through laboratory testing to be similar but slightly weaker (CBR strength) than that at the TSF. Laboratory results of the alluvium at the RWD indicates dispersive soils and aggressivity towards concrete and corrosivity towards steel. This material may be considered for reuse as a G7 to G8 quality material but is not recommended for reuse below water bearing structures due to the dispersive nature of the material, alternatively if this material is required measures to neutralise the dispersivity can be considered by the design engineer. The alluvium when remoulded is capable of achieving φ-values between 36° and 38° and cohesion of 0 kPa. The permeability is in the order of 10^{-7} m/s.

The calcrete that was tested indicates high dispersivity, aggressivity towards concrete and corrosivity towards steel. This material may be considered for reuse as a G7 quality material but is not recommended for reuse below water bearing structures due to the highly dispersive nature of the material, alternatively if this material is required measures to neutralise the dispersivity can be considered by the design engineer. The calcrete when remoulded can achieve φ-values of 38° and cohesion of 0 kPa.

All materials used for construction purposes, should be overseen by a suitably experienced materials engineer, and should be tested regularly and consistently to confirm the materials are in accordance with the required specifications.

4.9.3 MATERIAL STRENGTH AND PERMEABILITY

The anticipated values for the various materials considered for re-use are provided based on published literature:

Table 4-6:Anticipated Material Strength and Permeability

Material	Cohesion (c') kPa	Friction (φ') °	Permeability (k) m/s
TSF Alluvium	0 - 5	31 - 36	10^{-6} to 10^{-7}
TSF Soft Rock Gneiss	0	32 - 37	10^{-5} to 10^{-7}
RWD Alluvium	0 - 5	30 - 34	10^{-6} to 10^{-7}
RWD Calcrete	0 - 5	31 - 36	10^{-5} to 10^{-7}

The samples selected for shearbox and permeability testing were taken from disturbed samples for indications of the reuse of materials. The results returned values that were in-line and slightly better than anticipated values from literature when remoulded to 93% of Maximum Dry Density (Standard Proctor Effort). This may be attributed to the good compaction effect achieved for the material.

Table 4-7: Laboratory Results - Material Strength and Permeability

Material	Cohesion (c') kPa	Friction (φ') °	Permeability (k) m/s
TSF Alluvium	0	36 - 38	10^{-8}
TSF Soft Rock Gneiss	0	41	10^{-8}
RWD Alluvium	0	36 - 38	10^{-7}
RWD Calcrete	0	36	Not tested

4.10 CONCLUSION AND RECOMMENDATIONS

Two areas have been investigated for the phase 2 TSF expansion at Gamsberg Zinc Mine. The proposed phase 2 TSF will be an expansion to the north of the existing TSF while the proposed RWD is south-west of the existing TSF. The investigations comprised geophysical surveys, test pit excavation, rotary core drilling and laboratory testing.

The soil profiles generally comprised pedogenic soils, and were underlain at shallow depth by weathered bedrock, with the exception of deeper soil profiles comprising pedogenic alluvial soils. A prominent quartzite and gneiss ridge is outcropping in the western, central portion of the proposed TSF area. Similar shallow hard rock features are anticipated across the site at shallow depth as encountered during the construction of the existing TSF.

The RWD is generally underlain by deep alluvial soils, becoming silicified with depth.

4.10.1 TAILINGS STORAGE FACILITY

The TSF expansion area is underlain by alluvium, calcrete in varying stages of pedogenesis and shallow granite/gneiss bedrock. The soil in this area is generally thin with soft or medium hard rock varying from depths of 0.1m below surface to 0.7m. The transported, pedogenic and residual soils are considered suitable for reuse and should be removed or ripped and recompacted to remove loose pockets and prevent settlement of the thin soil profile. Soft excavation is typically anticipated to depths of at least 1.2m below ground level across the site but localised intermediate to hard excavation is anticipated, particularly where quartzite and shallow rock bands were observed (western central portion of the site).

Surface water and river channel water must be diverted to prevent seepage, ponding and excess water below the TSF.

In-situ permeability of the compacted excavation floors should be carried out to determine compliance with the design barrier system.

The following earthworks are proposed:

- Excavate and stockpile the upper 150 mm (organic content) at the TSF footprint for future (topsoil) remediation.
- Should deeper foundations be required according to the design, excavate or localised rip/blast and stockpile the material for reuse.
- The excavation floor must be ripped 150 mm deep and compacted to 95% MOD AASHTO Maximum Dry Density (MDD) at Optimum Moisture Content (OMC) to densify the loose in-situ soil.
- Where subsurface drains are required, localised intermediate to hard excavation is anticipated across the majority of site.
- Provision should be made for a protective layer below the barrier system.

4.10.2 RETURN WATER DAM

The RWD area is underlain by alluvium with shallow calcrete horizons becoming silicified with depth before granite/gneiss bedrock is encountered. The thick soil horizon is anticipated to have loose horizons or pockets as observed in the test pit profiles to at least 3.5m below surface. The transported and pedogenic soils are considered for reuse however, the soils are indicated to be dispersive and not suitable below water bearing structures.

Surface water and river channel water must be diverted to prevent seepage, ponding and excess water below the RWD.

In situ permeability tests of the compacted excavation floors should be carried out to determine compliance with the design barrier system.

The following earthworks are proposed:

- Excavate and stockpile the upper 150 mm at the RWD footprint for future (topsoil) remediation.
- Found the RWD at least 3.5m below ground level.
- Excavate 150 mm below the proposed founding depth, in situ rip and recompact the excavation floor to 95% MOD AASHTO Maximum Dry Density (MDD) at Optimum Moisture Content (OMC) to densify the loose in-situ soil.
- Localised in situ densification may be required should loose pockets be encountered in the excavation floor.
- Excavated material may be stockpiled based on material reuse requirements.
- The excavation side slopes should not be steeper than 1V:3H to prevent side wall collapse.
- Provision should be considered for a protective layer below the barrier system.
- Localised intermediate to hard rock excavations may be anticipated from a depth of 2.6m below surface in very dense silicified alluvium or pegmatite vein (as encountered in RWDTP5).

5.0 TAILINGS PHYSICAL CHARACTERISATION

In order to assess the anticipated tailings physical and chemical characteristics several samples of tailings were taken from the Phase 1 TSF. The chemical analysis is described within the subsequent section of this report, waste classification. KP made use of samples taken from this study as well as previous studies which KP had access to at the time of writing this report to compile the information presented within this section.

5.1 SAMPLE COLLECTION AND LOCATION

The samples taken from Phase 1 TSF are presented in Figure 1-1Figure 5-1 below. These sample locations were selected in order to be representative of both the underflow tailings and overflow tailings. In particular several samples taken on the tailings beach between the pond and underflow wall were selected to because of the natural segregation of hydraulically deposited tailings with the expectation that the overflow tailings would be coarsest at the beach head and finest at the pond. More than 20kg of material was taken at each of the locations presented blow.

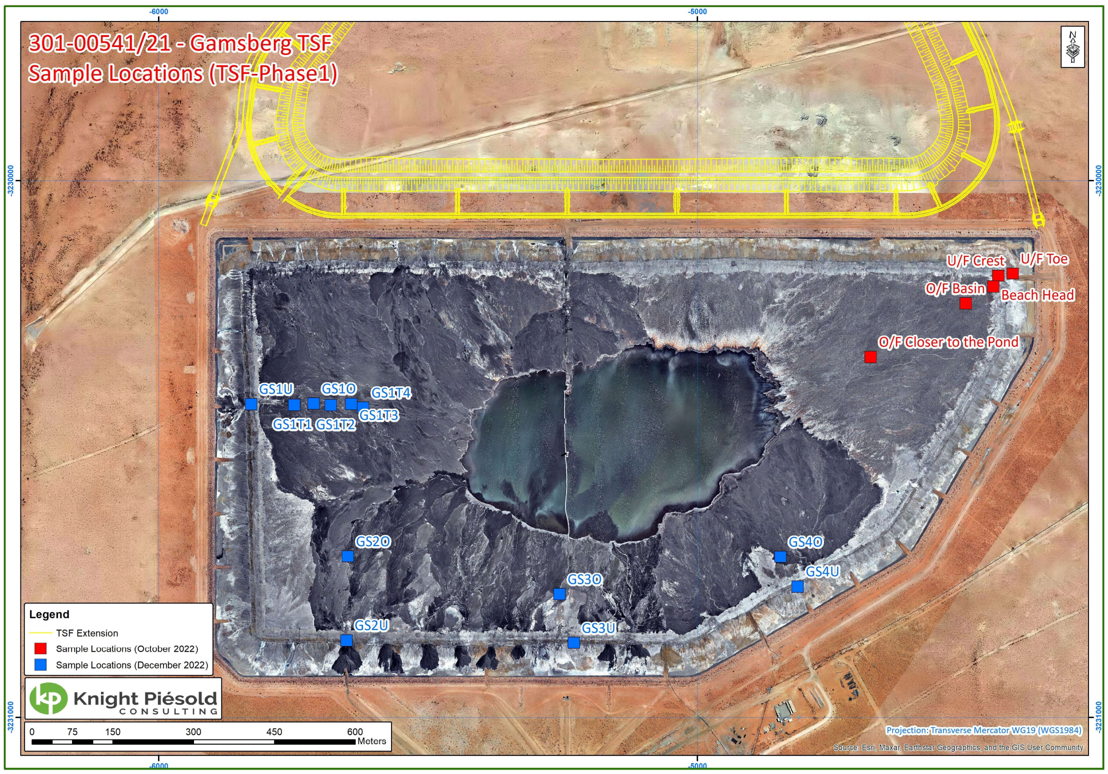


Figure 5-1: Tailings Sample Locations

5.2 LABORATORY TESTING RESULTS

The samples above were taken for various laboratory tests presented within this section. The feed tailings particle size distribution (PSD) was taken from rheology testing results described within the slurry pipeline design section of this report. The details of the tests can be found in

5.2.1 FOUNDATION INDICATOR AND SPECIFIC GRAVITY

The PSD's are presented in Figure 5-2 below.

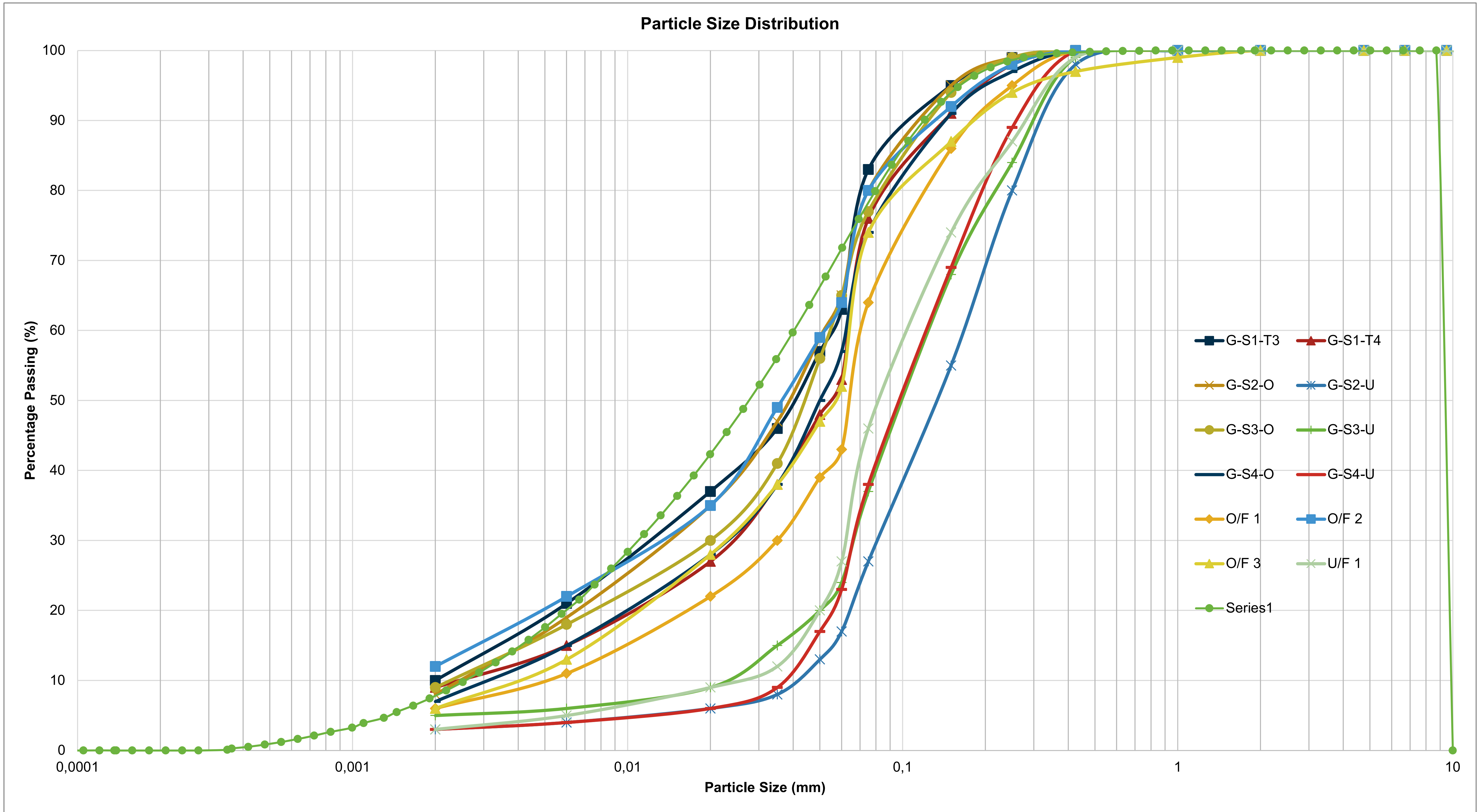


Figure 5-2: Tailings Particle Size Distribution

The interpretation of these PSD's , Atterberg limits and pycnometer testing are presented in

Table 5-1: Foundation Indicator Summary and Interpretation Including SG

Indicator	Feed Tailings	G-S1-O	G-S1-U	G-S1-T1	G-S1-T2	G-S1-T3	G-S1-T4	G-S2-O	G-S2-U	G-S3-O	G-S3-U	G-S4-O	G-S4-U	O/F 1	O/F 2	O/F 3	U/F 1	U/F 2
53	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
37.5	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
26.5	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
19	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
13.2	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
9.5	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
6.7	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
4.75	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
2	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
1	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	99	100	100
0.425	100	100	98	100	100	100	100	100	98	100	99	100	100	100	100	97	99	98
0.25	98	96	83	99	97	99	98	99	80	99	84	97	89	95	98	94	87	85
0.15	92	89	64	93	91	95	91	95	55	94	68	91	69	86	92	87	74	67
0.075	74	72	33	77	77	83	76	80	27	77	37	74	38	64	80	74	46	30
0.06	68	56	22	62	53	63	53	65	17	65	24	57	23	43	64	52	27	17
0.05	62	49	18	55	48	57	48	59	13	56	20	50	17	39	59	47	20	14
0.035	52	37	12	44	39	46	38	47	8	41	15	38	9	30	49	38	12	9
0.02	39	25	9	34	27	37	27	35	6	30	9	28	6	22	35	28	9	7
0.006	18	11	6	19	16	21	15	19	4	18	6	15	4	11	22	13	5	4
0.002	7	6	5	8	9	10	9	8	3	9	5	7	3	6	12	6	3	3
Liquid Limit (%)	N/T	-	-	17	18	18	17	18	-	17	-	-	-	-	17	-	-	-
Plastic Limit (%)	N/T	-	-	14	15	15	15	16	-	15	-	-	-	-	14	-	-	-
Plasticity Index (%)	N/T	NP	NP	3	3	3	2	2	NP	2	NP	SP	NP	NP	3	NP	NP	NP
Linear Shrinkage (%)	N/T	0	0	1.5	1.5	1.5	1	1	0	1	0	0.5	0	0	1.5	0	0	0
PI of Whole Sample	N/T	-	-	3	3	3	2	2	-	2	-	-	-	-	3	-	-	-
% Gravel		0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
% Sand		44	78	38	47	37	47	35	83	35	76	43	77	57	36	48	73	83
% Silt		50	17	54	44	53	44	57	14	56	19	50	20	37	52	46	24	14
% Clay		6	5	8	9	10	9	8	3	9	5	7	3	6	12	6	3	3
Activity		0	0	0.4	0.3	0.3	0.2	0.3	0	0.2	0	0	0	0	0.3	0	0	0
% Soil Mortar		100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100	100
Grading Modulus		0.28	0.69	0.23	0.23	0.17	0.24	0.2	0.75	0.23	0.64	0.26	0.62	0.36	0.2	0.29	0.55	0.72
Moisture Content (%)	N/T	16.5	0.7	16.9	18	21.9	17.5	23.3	1.3	21.5	1.5	19.8	0.5	N/T	N/T	N/T	N/T	N/T
Relative Density (SG)		3.599	4.043	3.291	3.331	3.301	3.334	3.331	3.937	3.559	4.256	3.365	4.24	3.438	3.538	3.624	4.056	4.412
Unified (ASTM D2487)		ML	SM	ML	ML	ML	ML	ML	SM	ML	SM	ML	SM	ML	ML	ML	SM	SM
AASHTO (M145-91)		A - 4	A - 2 - 4	A - 4	A - 4	A - 4	A - 4	A - 4	A - 2 - 4	A - 4	A - 4	A - 4	A - 4	A - 4	A - 4	A - 4	A - 4	A - 2 - 4

5.2.2 PERMEABILITY

Flexible wall permeability tests were performed on representative samples of underflow and overflow. These results are summarised in Table 5-2

Table 5-2: Flexible Wall Permeability Test Results

Test Material	Coefficient of Permeability (m/s)		
	Minimum	Maximum	Average
Underflow (G -S1 -U)	7.09E-07	1.25E-06	9.75E-07
Overflow (G-S3-0)	2.89E-08	4.24E-08	3.77E-08

5.2.3 MOISTURE – DENSITY RELATIONSHIP

Moisture density relationship testing was carried out on the representative samples of underflow and overflow. Standard Proctor compaction energy was selected . The results summary of these tests are presented in Figure 5-3 for the underflow and overflow tailings respectively.

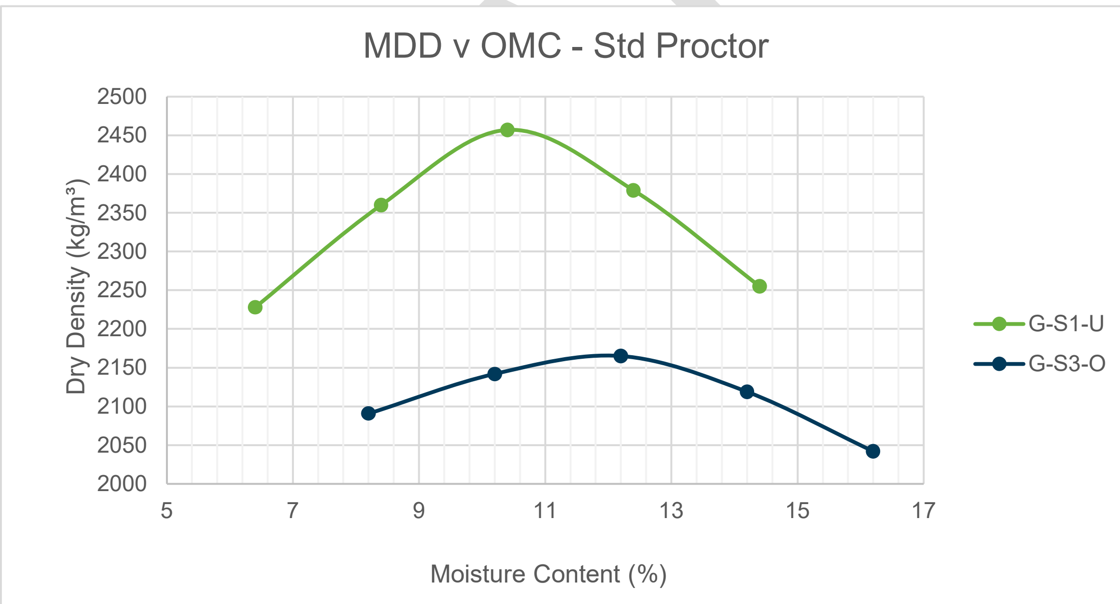


Figure 5-3: Underflow

5.2.4 TRIAXIAL TESTS

Two sets of triaxial tests were carried out on representative samples of underflow and overflow tailings to determine the strength of the tailings. The void ratios and confining stress values were varied in order to determine the strength and the critical state line(CSL).

The triaxial results of the tests for strength are presented within Figure 5-4 and Figure 5-5. The friction angle for underflow was 32° while overflow was at 29°. No cohesion was observed from the testing.

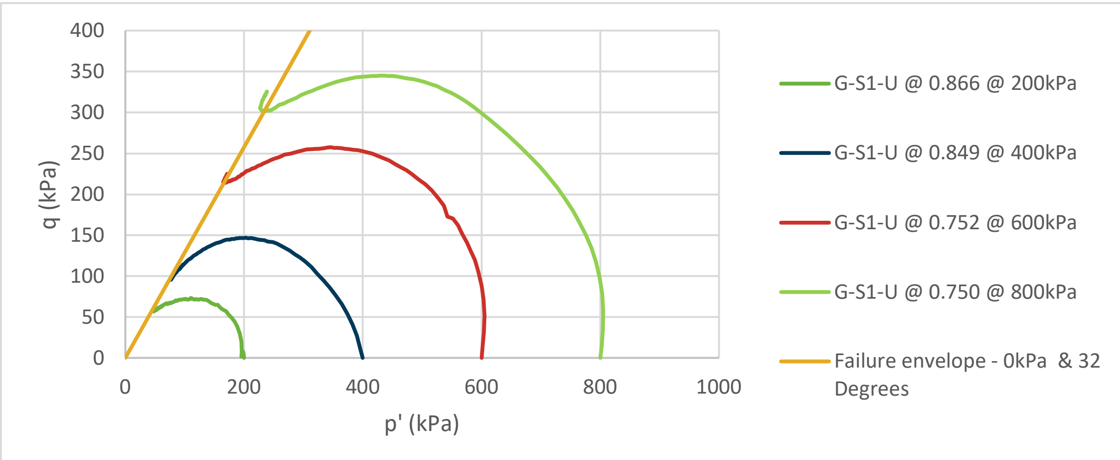


Figure 5-4: CU Triaxial Tests – Underflow

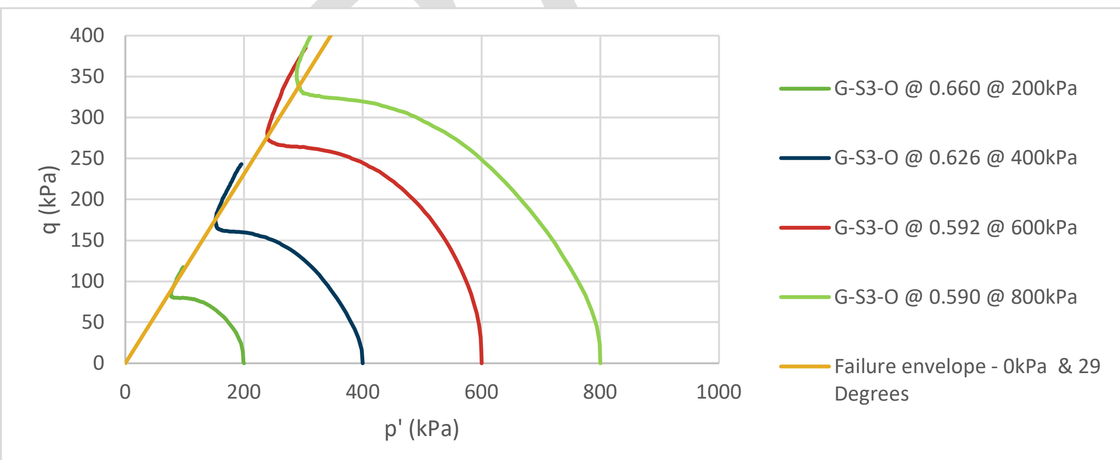


Figure 5-5: CU Triaxial Tests – Overflow

5.2.5 OEDOMETER TESTS

Flexible wall permeability tests were performed on representative samples of underflow and overflow. These results are presented in Figure 5-6,Figure 5-7,Figure 5-8 and Figure 5-9.

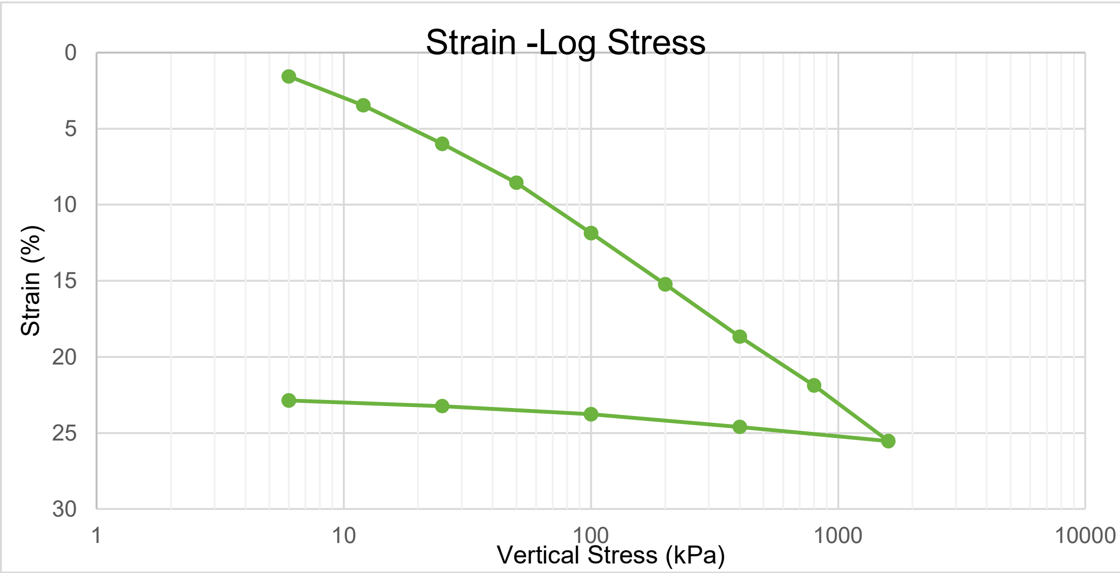


Figure 5-6: Strain v Log Stress - Underflow

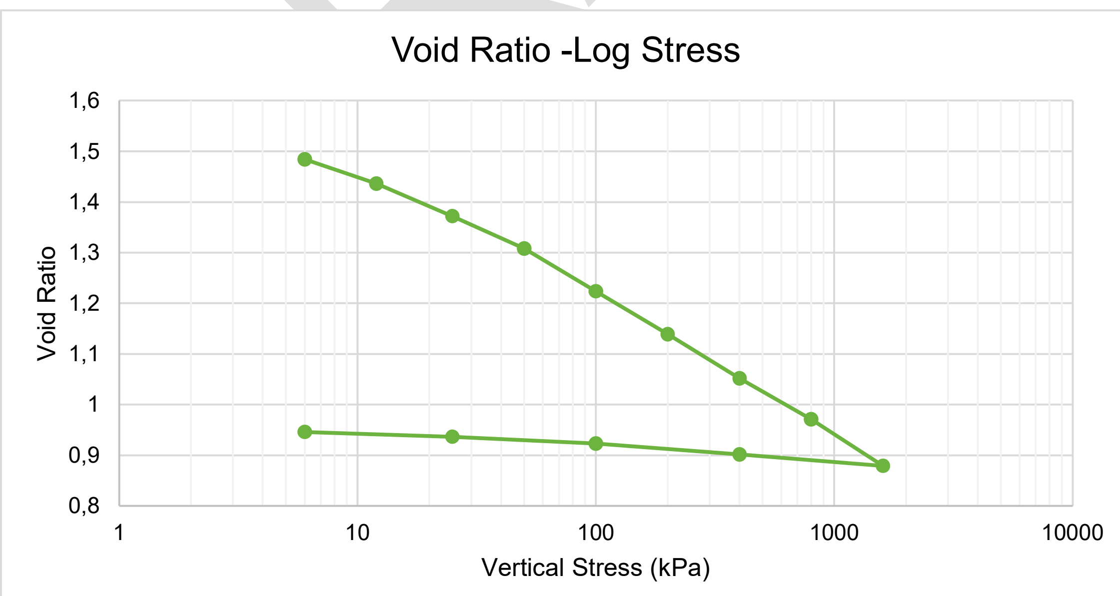


Figure 5-7: Void Ratio v Log Stress - Underflow

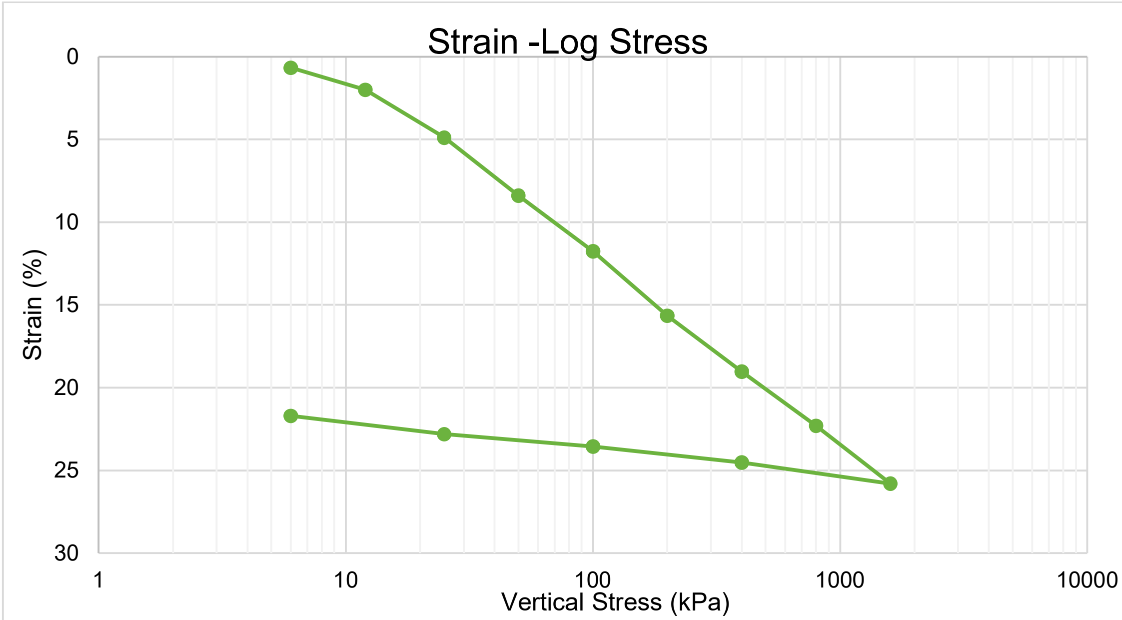


Figure 5-8: Strain v Log Stress - Overflow

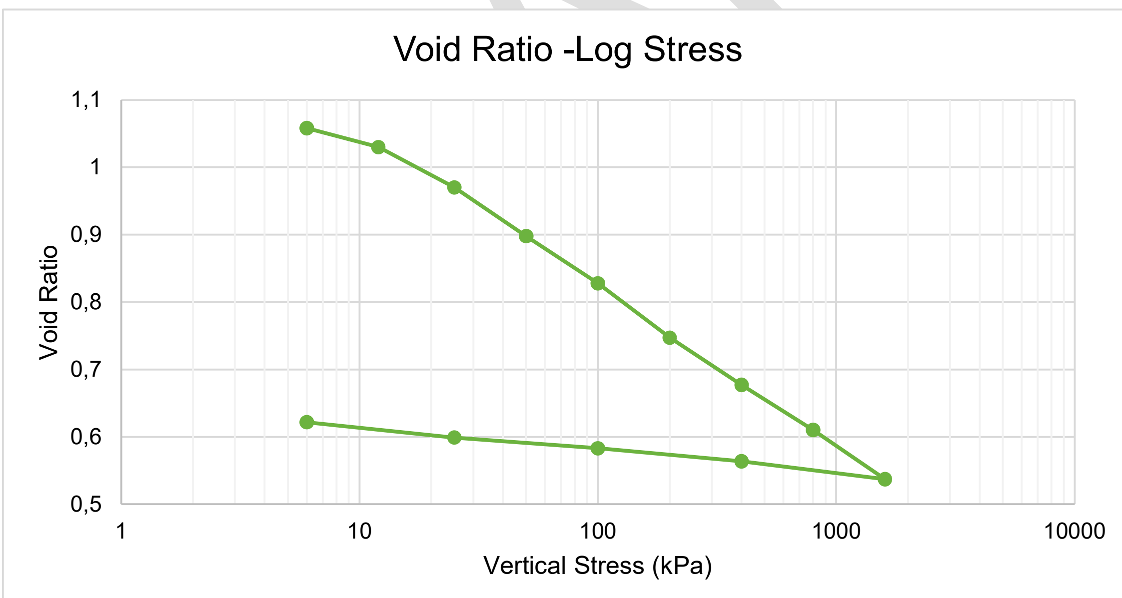


Figure 5-9: Void Ratio v Log Stress - Underflow

5.2.6 CRITICAL STATE LINE EVALUATION AND BRITTLENESS EVALUATION

Triaxial testing to determine the Critical State line(CSL) are being performed at current. Several tests have been completed and a preliminary line has been determined. Once all the tests have been completed the results will be presented here. The triaxial tests completed have been analysed for brittleness as per Appendix B within ICOLD 2022 (ICOLD, 2022).

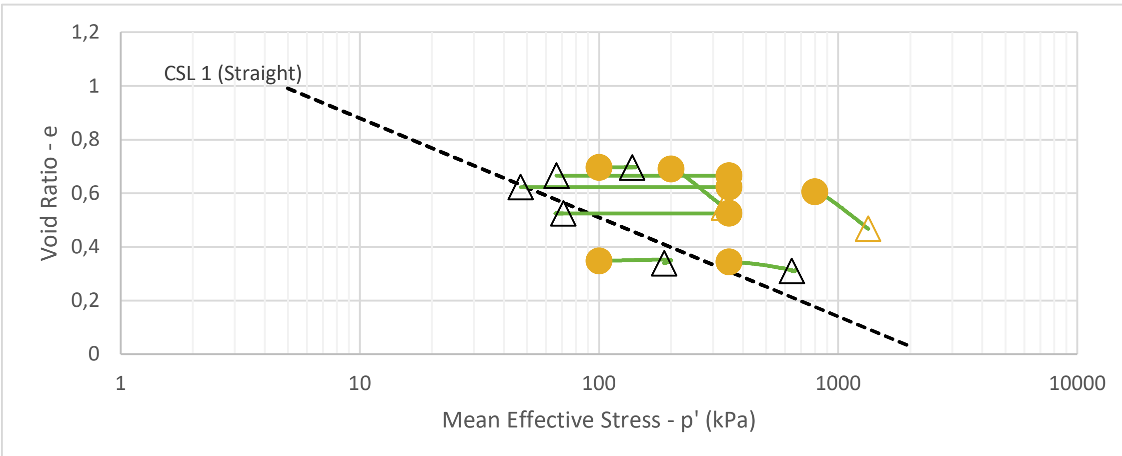


Figure 5-10: Critical State Line – Overflow

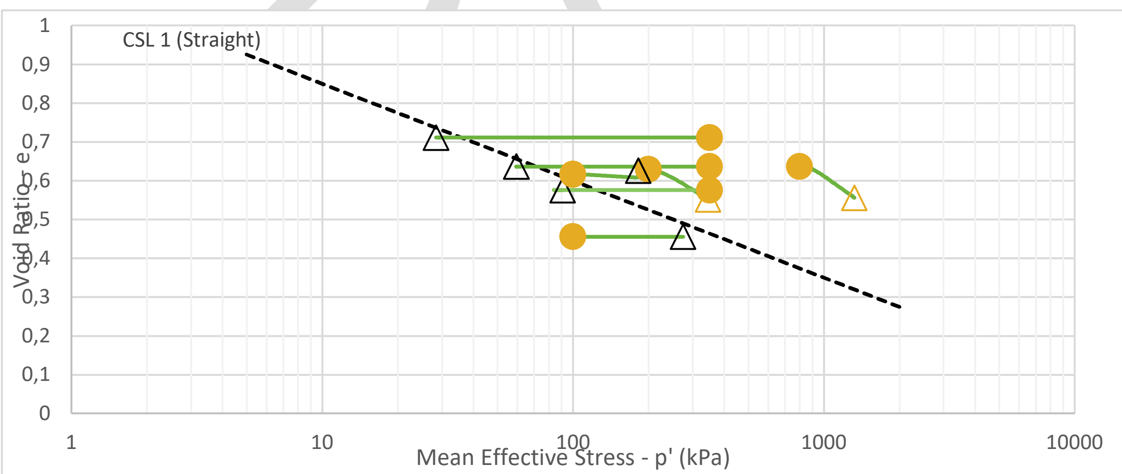


Figure 5-11: Figure 5-12: Critical State Line – Underflow

6.0 WASTE CLASSIFICATION

6.1 INTRODUCTION

Two sets of samples were taken from the Gamsberg TSF site to assess the geochemistry and subsequently classify the waste type as per GN635 regulations. The initial sampling took five samples from various TSF locations on Phase 1. Subsequent to the sampling, various geochemical tests were carried out on these five samples. These results, however, were discarded upon confirmation of a circuit change within the Phase 2 plant. A second round of sampling therefore took place, 4 samples were taken to verify the Phase 1 results described above and one sample was collected by KP from SGS laboratory. This sample referred to as manufactured tailings was considered representative of what tailings will be produced from the Phase 2 plant. All 10 sample results with full geochemical analysis can be found within Appendix D of this report.

A summary of the waste classification for the manufactured tailings sample (KPGM-SO8 T) is detailed below. The final waste classification and acid rock generation report is contained within Appendix D.

6.2 WASTE ASSESSMENT

The manufactured tailings sample represents the tailings that will be generated in the new process circuit which reportedly includes the removal of lead and cadmium.

The waste assessment is in accordance with South African GN R635 National Norms and Standards for the Assessment of Waste for Landfill Disposal (2013). The process includes identifying the chemical substances present in the waste through analysis of the Total Concentrations (TC) and Leachable Concentrations (LC) of the samples taken. These results are compared to Total Concentration Threshold (TCT) and Leachable Concentration Threshold (LCT) limits specified in GN R635 and the outcome is used to establish the type of waste and disposal requirements. Various threshold levels for the TCT (TCT0, TCT1, TCT2) and LCT (LCT0, LCT1, LCT2, and LCT3) are provided which, in combination, determine the Risk Profile and corresponding waste types.

6.3 LEACHATE NORMS AND STANDARDS

The classification of the waste sample in terms of GN 635, requires the Australian Standard Leaching Procedure (ASLP), which uses aqua regia digestion to determine the total concentrations (TC) of the elements from the samples. While the leachable concentrations (LC) are determined using the Toxicity Characteristic Leaching Procedure (TCLP) which involves distilled water as the tailings will be mono disposal with no putrescible materials.

The results for the manufactured tailings samples, for the total and leachate concentrations, along with applicable threshold limits used for the classification of the samples is shown in Table 1.

The manufactured tailings sample (KPGM-SO8 T) shows that leachable concentrations (LC) for the majority of the parameters fall below the LCT0 limits, however some parameters do exceed these limits. The manufactured tailings sample shows elevated concentrations for the parameters, Manganese (Mn), lead (Pb), Cadmium (Cd) and Zinc (Zn), exceeding the LCT0 limits but below the LCT1 limits.

The TC for the manufactured tailings sample shows that the majority of the parameters fall below their respective TCT0 limits. However, the parameters, Arsenic (As), Barium (Ba), Copper (Cu), Mn, Nickel (Ni), Pb and Zn all exceed the TCT0 limits but do not exceed the TCT1 limits.

Based on the TCT and LCT limits, the manufactured tailings sample falls with LCT0<LC<LCT1 and TCT0<TC<TCT1 which classifies this sample as a **Type 3 waste** and can be disposed of at a Class C landfill. A summary assessment of this is presented in Table 6-1.

Table 6-1:Total & Leachable concentrations - Manufactured Tailings Sample (KPGM-S08 T)

Parameter	KPGM-S08 T		Leachable Concentrations Thresholds				Total Concentrations Thresholds		
	LC (mg/l)	TC (mg/kg)	LCT 0 (mg/l)	LCT 1 (mg/l)	LCT 2 (mg/l)	LCT 3 (mg/l)	TCT 0 (mg/kg)	TCT 1 (mg/kg)	TCT 2 (mg/kg)
As, Arsenic	0.004	388	0.01	0.5	1	4	5.8	500	2 000
B, Boron	<0.025	81	0.5	25	50	200	150	15 000	6 000
Ba, Barium	0.047	113	0.7	35	70	280	62.5	6 250	25 000
Cd, Cadmium	0.018	7.60	0.003	0.15	0.3	1.2	7.5	260	1 040
Co, Cobalt	<0.025	45	0.5	25	50	200	50	5 000	20 000
Cr_{Tot}, Chromium Total	<0.025	386	0.1	5	10	40	46 000	800 000	N/A
Cr 6+, Chromium (VI)	<0.010	<0.200	0.05	2.5	5	20	6.5	500	2 000
Cu, Copper	0.022	99	2.0	100	200	800	16	19 500	78 000
Hg, Mercury	<0.001	<0.400	0.006	0.3	0.6	2.4	0.93	160	640
Mn, Manganese	5.44	4317	0.5	25	50	200	1 000	25 000	100 000
Mo, Molybdenum	<0.025	<10	0.07	3.5	7	28	40	1 000	4 000
Ni, Nickel	0.063	177	0.07	3.5	7	28	91	10 600	42 400
Pb, Lead	0.271	1161	0.01	0.5	1	4	20	1 900	7 600
Sb, Antimony	0.001	1.00	0.02	1.0	2	8	10	75	300
Se, Selenium	<0.001	4.00	0.01	0.5	1	4	10	50	200
V, Vanadium	<0.025	<10	0.2	10	20	80	150	2 680	10 720
Zn, Zinc	16	5600	5.0	250	500	2 000	240	160 000	640 000
pH	5.6	-							
Chloride as Cl	5	3432.00	300	15 000	30 000	120 000	N/A	N/A	N/A
Sulphate as SO4	89	3211.00	250	12 500	25 000	100 000	N/A	N/A	N/A
Nitrate as N	<0.1	<5	11	550	1 100	4 400	N/A	N/A	N/A
Fluoride as F	1.1	1.47	1.5	75	150	600	100	10 000	40 000
Total Cyanide as CN	<0.07	<1.55	0.07	3.5	7	28	14	10 500	42 000

1. **NOTES:** ONLY THE MANUFACTURED TAILINGS SAMPLES PRODUCED WITHIN THE LABORATORY IS SHOWN ABOVE. IT IS ASSUMED THAT THE MANUFACTURED TAILINGS SAMPLE IS REPRESENTATIVE OF THE TAILINGS WHICH WILL BE PRODUCED FROM THE PHASE 2 PLANT. NORMAL QUALIFICATIONS WITHIN WATER USE LICENSES REQUIRE TESTING ONCE THE PLANT IS OPERATIONAL TO CONFIRM THE WASTE CLASSIFICATION

7.0 BARRIER SYSTEM DESIGN

A waste classification and geochemistry evaluation were done in accordance with the Waste Classification and Management Regulations (WCMR), GN R634 (2013) and the Mineral and Petroleum Resources Development Act (Act 28 of 2002) (MPRDA) (2002).

7.1 REGULATORY REQUIREMENTS

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

- National Environmental Management: Waste Act, 2008 (Act 59 of 2008) (NEMWA) (2008);
- Waste Classification and Management Regulations (GN R634 of 23 August 2013) (2013);
- National Norms and Standards for Assessment of Waste for Landfill Disposal (GN R635 of 23 August 2013) (2013); and
- National Norms and Standards for Disposal of Waste to Landfill (GN R636 of 23 August 2013) (2013).
- Petroleum Resources Development Act (Act 28 of 2002) (MPRDA) (2002).

7.2 BARRIER DESIGN

The initial tests were done on samples from the existing TSF – refer to the Waste Classification Assessment (December 2022). The results were interpreted, and the waste classified as a Type 1 waste.

Subsequently, an additional sample (KPGM-SO8 T Sample #:185964), was manufactured based on the new plant design specification and this sample was tested – refer to Waste Assessment Summary for Manufactured Tailings Sample, dated 31 March 2023. The new sample was classified as a Type 3 waste and therefore in accordance with GN R636 (23 August 2013), National Norms and Standards for Disposal of Waste to Landfill, the minimum required barrier system is a Class C barrier system as presented in Figure 7-1.

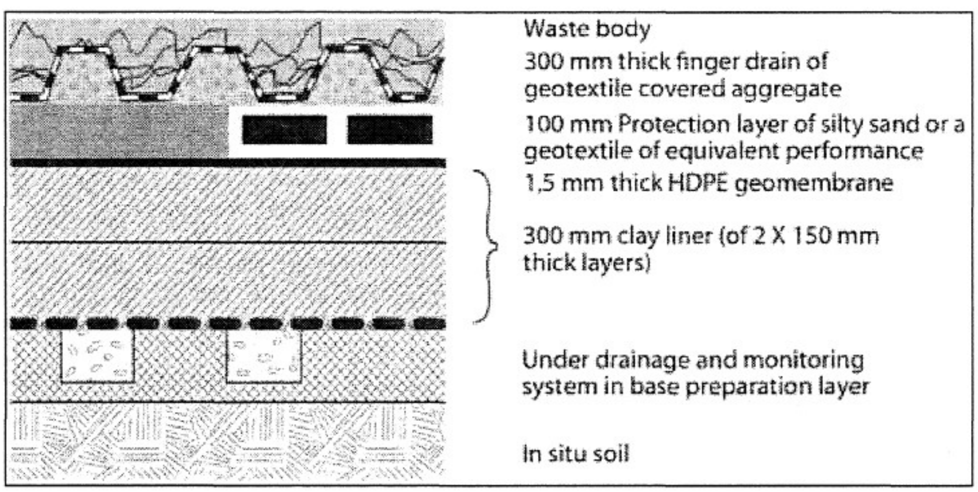


Figure 7-1: Standard Class C Barrier System

This report considers the tailings parameters, the available materials on site and recent developments in barrier construction to provide a suitable barrier design. The method of leakage rate calculation for the proposed barrier system is also discussed.

7.3 BARRIER DESIGN

Contrary to the previous barrier design based on the first set of waste samples, where several options were considered for the secondary layer of the Class A barrier, the Class C Barrier only consists of a primary barrier and a drainage system on top of the barrier.

At the site of the proposed tailings facility the groundwater is at about 27 meters (MBH9). This means that the vertical thickness of the unsaturated pathway below the tailings is at least 25 meters. This also renders aquifer vulnerability very low. Due to the depth of the groundwater on site, no ground water drains will be required underneath the barrier.

The options to consider for a Class C Barrier are:

1. Apply the standard Barrier Design as per Figure 7-1. Refer to Section 7.3.2 for discussion on the standard barrier and Section 7.4.1 and 7.4.2 for leakage rate estimations.
2. Use an inverted barrier, 1.5 mm Geomembrane (GMB) on top of a Non-woven Geotextile (GTX-NW), on top of the prepared base, covered with tailings, as per the current TSF – refer to Section 7.3.3 for discussion on an inverted Barrier and Section 7.4.3 for the leakage rate estimation. Note that the tailings must be from the new plant since the classification of the existing plant's tailings require a higher-class barrier.
3. Use an inverted barrier, 2 mm GMB on top of prepared base, covered with 300 mm crushed and compacted calcrete layer – for ballast.

The HDPE GMB will be covered with an appropriate protection GTX-NW in the drains in all cases. A separate Memorandum was prepared to assess the different leakage rates of the options considered and a high-level cost comparison presented. The memorandum has been attached as Appendix D.

7.3.1 BASE PREPARATION

The TSF floor is covered with residual soils and very soft rock gneiss with nodular to hardpan calcrete overlain by aeolian sand. The aeolian sand layer thickness varies between 0.1 m to 0.4 m. It is recommended to remove the topsoil in the TSF footprint. In general, the basin will be ripped 150 mm deep and re-compacted to 95% MOD AASHTO maximum dry density at ±2% OMC. The basin will be shaped to assist with the installation of drainage lines and fill the borrow areas.

The TSF is a HDPE lined facility and therefore requires a smooth layer of compacted material to protect the barrier system. A 200 mm cushion layer is required in areas where the honeycomb to hardpan calcrete/ferricrete is exposed or at any variable floor topography. The cushion material must be a well compacted sandy material or a nodular calcrete material. These materials are available on site. The barrier system will then be installed onto the prepared subgrade.

7.3.2 STANDARD CLASS C BARRIER

The standard Class C Barrier, Figure 7-1, requires a 300 mm clay layer underneath the GMB. This clay can be replaced by a Geosynthetic Clay Liner (GCL) or an equivalent low permeability layer. The alternatives were discussed in Appendix D. Of the options noted, only a polymer modified GCL or a Coated GCL may be viable for this project for the following reasons:

- There is very little to no clay available;
- Normal GCL's are susceptible to cation exchange, reducing its effectiveness;
- Bentonite enriched soil has a significant cost and requires very stringent Quality Control (QC) to ensure a homogenous mixture.

The polymer modified GCL and the coated GCL also have significant additional cost implications.

A 400 mm minimum thickness ballast layer will have to be provided on the GMB to prevent swelling of the hydrated GCL underneath the GMB.

The other alternatives are discussed below.

7.3.3 AN INVERTED BARRIER

Developments and research for example (Mnisi & Bester, 2018), have driven for the alteration of the standard configuration (Figure 7-1), where the Compacted Clay Liner (CCL) is placed on top of the Geomembrane (GMB), thereby inverting the barrier.

The main advantage of inverting the barrier is the ability to use readily available fine tailings as a replacement for the clay. The fine tailings should have a low permeability in the order of about $1x10^{9}$ m/s. This reduces costs at mine sites where clay is not readily available, or where the waste is not compatible with GCLs. Some tailing samples have been tested in the past and it was found to have a permeability of $4x10^{-8}$ m/s, but more testing is underway to confirm this value.

The tailings material is fine grained and delivered to the facility as wet slurry. For this reason, it is considered that the tailings material will form a protection layer over the geomembrane over time.

The fine tailings layer placed on top of GMB provides a ballast layer above the geomembrane to reduce discontinuities and defects (wrinkles and holes) (Mnisi & Bester, 2018). Placing dry tailings material during construction delivers the best results but would not be possible for this Phase 2 of the Gamsberg project. Hydraulic placement will have to be done at commissioning. To prevent wind uplift and forming of excessive wrinkles, sandbags will be required. It is recommended to use large (0.5 ton) UV protected bags, filled with sand by means of a front-end loader, as the GMB is installed (to avoid traffic on the GMB). Small bags tend to result in significantly more foot traffic on the GMB and are seldom placed dense enough to completely avoid movement of the GMB.

If possible, compaction of the low permeability layer on top of the GMB may improve the barrier since the permeability of the low permeability layer is greatly reduced by the compaction (Rowe, 2012). The leakage rate is reduced by placement of more tailings on top during operation, thereby increasing the effective stress.

There is also some research that suggests that the clay/tailings may get eroded through the defect (piping) (Chou, 2018). The risk of piping through the defect can be reduced by placing a non-woven geotextile (GTX-NW) underneath the GMB. The fine tailings tend to blind the GTX-NW, which will then reduce the risk of piping.

Placing a GTX-NW or a sand layer underneath the GMB as a protection layer, will typically reduce the risk of irregularities in the prepared base, but in this case a GTX-NW will be required as a protection layer above the base layer.

Research has shown that in addition to protecting the GMB, the leakage rate is also reduced by the GTX-NW. A thicker GTX-NW will reduce the leakage further (Fan & Rowe, December 2022). This is due to the blinding noted above.

It is considered viable to implement an inverted barrier for reasons as noted in Section 7.3.2.

7.3.4 CALCRETE BALLAST LAYER

Since suitable dry tailings material would not be available at the start of construction and the extreme temperature variations in the area, an alternative of using crushed calcrete as a ballast layer on top of the GMB was considered. This material is readily available on site but will require crushing and could result in damage of the GMB. There should be no GTX-NW between the calcrete layer and the GMB, to ensure good contact, therefore the GMB is increased to 2 mm to account for possible damage. The main advantage of this alternative is the provision of a ballast layer that completely covers the GMB, which reduces the likelihood of any wrinkles. Due to the higher permeability of the calcrete, the leakage rate will however be higher.

Prior to defining the final choice of barrier, the leakage rates for the proposed options will be discussed.

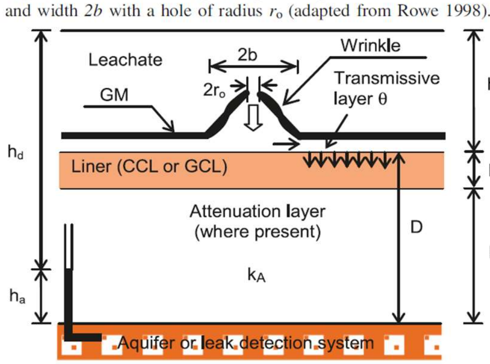
7.4 LEAKAGE ESTIMATION

Calculation of the estimated leakage rate through the various layers of the barrier system are discussed below. The first step was to determine the base leakage rate for a standard Class C Barrier. The proposed alternatives were then compared with the base case.

7.4.1 STANDARD CLASS C BARRIER LEAKAGE

For the standard Class C barrier as per Figure 7-1, the following equation (Rowe, 2012) is used.

Fig. 10. Schematic showing leakage through a wrinkle of length L and width $2b$ with a hole of radius r_o (adapted from Rowe 1998).



[6] $$Q = 2L[kb + (kD\theta)^{0.5}]h_d/D$$

where Q is the leakage (m³/s); L is the length of the connected wrinkle (m); k is either the hydraulic conductivity (m/s) of the clay liner, k_L, if there is no AL or the harmonic mean of the CL and AL hydraulic conductivities, k_s, if there is an AL; $2b$ is the width of the wrinkle (m); $D = H_L + H_A$ is the thickness of the CL and AL (m); θ is the transmissivity of the GM–CL interface (m²/s); and $h_d = (h_w + H_L + H_A - h_a)$ is the head loss across the composite liner (m). All of

Figure 7-2: Leakage Rate through GMB over low permeability layer

The following notes need to be considered for the above calculation:

1. The length of the wrinkle will vary with temperature and the area, various lengths were considered, but for the benchmark, a 200 m length wrinkle was considered.
2. Width of the wrinkle would be between 100 mm and 300 mm.
3. Assumed 3-5 wrinkles per Ha and 1 hole per wrinkle.
4. Assumed good contact between the GMB and CCL for transmissivity value.
5. The primary leakage was applied to the entire TSF area, as though the entire TSF is covered by a pool.
6. The head for the primary leakage was taken at 10 m, for similar reason as above. After 10 m, the tailings will start to consolidate and therefore reduce permeability and then the calculation done for the inverted barrier will become more applicable.
7. The primary clay layer was set at 600 mm.
8. Actual liquid outputs may be limited by liquid inputs.

The results for the theoretical leakage rate calculations are presented in Table 7-1. These values are conservative and will be limited by the liquid available. Keeping a small pool on the TSF will reduce these numbers.

Table 7-1: Standard Class C – Primary Barrier – GMB over CCL

Water depth	Wrinkle length	No. of wrinkles per Ha (primary)	Leakage rate primary	Leakage rate primary	Leakage rate primary for TSF[1]
[m]	[m]	[-]	[m³/s/wrinkle]	[L/Ha/day]	[L/day]
5	200	5	1.08265E-05	4677	559096
10	200	5	2.16531E-05	9354	1118192

NOTES

1. THE LEAKAGE MANAGEMENT SYSTEM SHOULD BE DESIGNED TO BE ABLE TO CAPTURE THE MAJORITY OF THE PRIMARY LEAKAGE.

7.4.2 ALTERNATIVE 1 - GMB OVER A GCL

For this case, the leakage rate through the GMB over an attenuation/transmissivity layer can be calculated using Equation [6] (Rowe, 2012) as per Figure 7-2. The same notes from Section 7.4.1 will apply.

The results for the leakage rate calculations are presented in Table 7-2.

Table 7-2: Alternative 1 - GMB over GCL

Water depth	Wrinkle length	No. of wrinkles per Ha (primary)	Leakage rate primary	Leakage rate primary	Leakage rate primary for TSF[1]
[m]	[m]	[-]	[m³/s/wrinkle]	[L/Ha/day]	[L/day]
5	200	5	5.8284E-07	252	30099
10	200	5	1.1657E-05	504	60197

NOTES

1. THE LEAKAGE RATE IS SIGNIFICANTLY LOWER THAN THE STANDARD CLASS C BARRIER WITH A CCL.

7.4.3 ALTERNATIVE 2 - LOW PERMEABILITY LAYER OVER A GMB (INVERTED BARRIER)

The leakage rate through the proposed inverted primary barrier can be calculated using the following equation.

Calculations of leakage (Q) using the following W–F equation (Wissa and Fuleihan 1993):

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

(Chou, et al., 2021) where:

- k is the hydraulic conductivity of the tailings above the geomembrane (assumed to be uniform),
- h is the head above the geomembrane,
- t is the geomembrane thickness, and
- d is the diameter of the hole in the geomembrane.

The following notes need to be considered for the above calculation:

1. Hydraulic conductivity of the tailings above the geomembrane taken as 4 x 10^{-8} m/s
2. The thickness of the head above the geomembrane will vary with the thickness of tailings placed on top of the GMB.
3. GMB thickness taken as 1.5 mm.
4. The diameter of the hole was varied from 5 mm to 100 mm.
5. The equation neglects resistance to flow from the material (GTX-NW) beneath the geomembrane. As noted in Section 7.3.3, placing a GTX-NW underneath the barrier will reduce permeability even further.

The results for the leakage rate for 5 holes per Ha are presented in Table 7-3.

Table 7-3: Alternative 2 - Inverted Barrier

Head above GMB	Hole Diameter	Leakage Rate	Holes per Ha	Leakage rate primary	Leakage rate primary for TSF[1]
[m]	[m]	[m³/s]	[-]	[L/Ha/day]	[L/day]
5	0.010	2.89E-09	5	1.3	149
10	0.010	5.79E-09	5	2.5	299
5	0.050	1.86E-08	5	8.0	960
10	0.050	3.72E-08	5	16.1	1919
5	0.100	3.85E-08	5	16.6	1990
10	0.100	7.71E-08	5	33.3	3979

Note that the leakage rate per Ha per day is less than calculated for the two standard Class C barriers. The major difference is that the W-F equation is independent of the thickness of the low permeability layer, while the first equation (Section 7.4.1), incorporates the wrinkle length and is much more conservative. It is clear that the inverted barrier has a lower leakage rate than the standard barrier.

7.4.4 ALTERNATIVE 3 – INVERTED, CALCRETE OVER GMB

The third option of an inverted barrier, using crushed and compacted calcrete on a 2 mm GMB, will require the W-F equation as used in Section 7.4.3.

The following notes need to be considered for the above calculation:

1. Hydraulic conductivity of the calcrete above the geomembrane taken as 4×10^{-6} m/s
2. GMB thickness taken as 2 mm.
3. The diameter of the hole was varied from 5 mm to 100 mm.
4. The equation neglects resistance to flow from the material beneath the geomembrane.
5. As noted in Section 7.3.3, placing a GTX-NW underneath the barrier will reduce permeability even further.

The results for the leakage rate for 5 holes per Ha are presented in Table 7-4.

Table 7-4: Alternative 3 – Calcrete over GMB

Head above GMB	Hole Diameter	Leakage Rate	Holes per Ha	Leakage rate primary	Leakage rate primary for TSF[1]
[m]	[m]	[m³/s]	[-]	[L/Ha/day]	[L/day]
5	0.010	1.64E-08	5	7.1	848
10	0.010	3.28E-08	5	14.2	1695
5	0.050	2.48E-07	5	107.0	12791
10	0.050	4.95E-07	5	214.0	25583
5	0.100	6.63E-07	5	286.2	34215
10	0.100	1.33E-06	5	572.5	68431

NOTES

2. THE RESULT FOR THE 100 MM HOLE CALCULATION, IS SIMILAR TO THE RESULT FOR ALTERNATIVE 1 (GMB OVER GCL).

7.5 PROPOSED BARRIER SYSTEM

Based on the discussion above, the leakage rates and the estimated cost, Alternative 2 as noted in was chosen as the optimum case.

Therefore, the typical arrangement as presented in Figure 7-3 is proposed for the inverted Class C barrier system for the project.

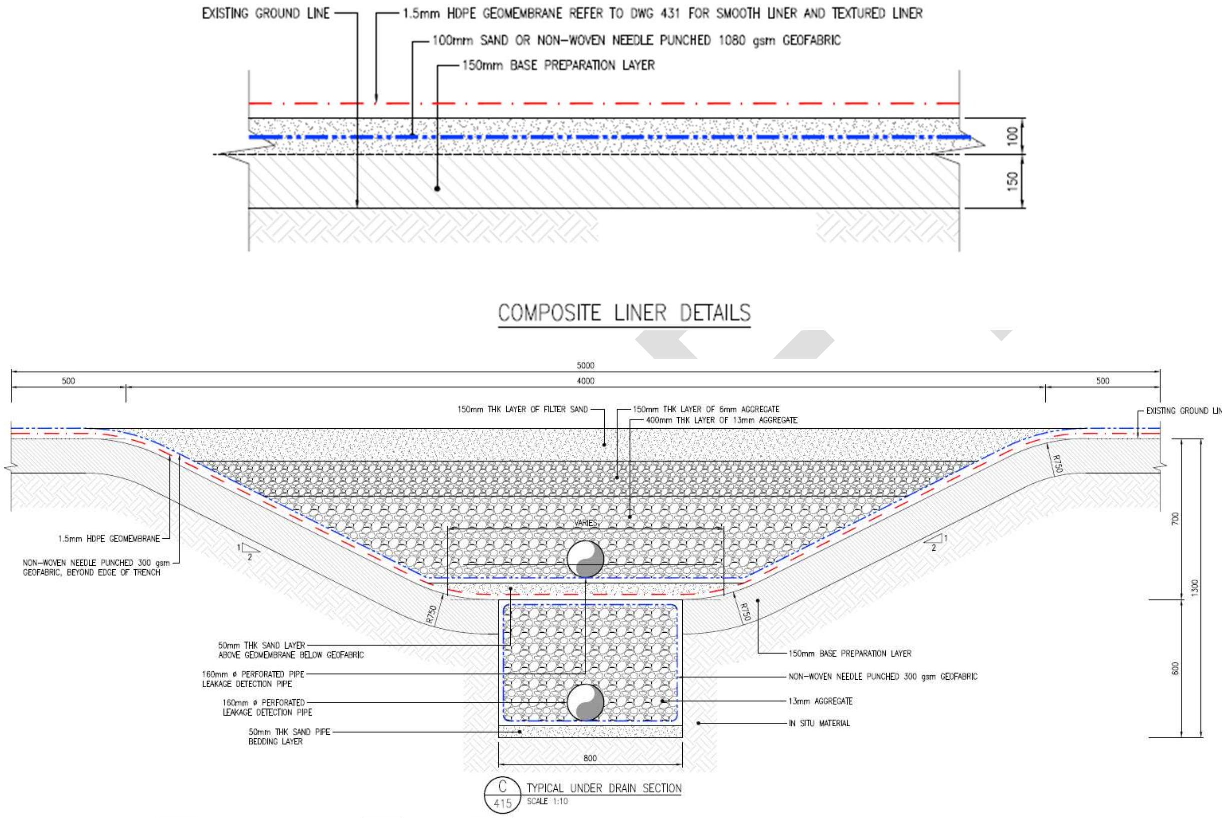


Figure 7-3: Proposed Inverted Class C, Inverted Barrier System

The layers are discussed from the bottom upwards.

1. Base Preparation Layer

During construction of the previous phase of the TSF, it was difficult to achieve a smooth surface over the entire basin, for the prepared base layer, which consisted of ripped and recompacted weathered calcrete to provide a low permeability layer. The site had a thin layer of sand on top of the calcrete, which was stripped. Where the base could not be prepared to an acceptable smoothness for geomembrane installation, a layer of sand or GTX-NW was placed for protection of the geomembrane. Since the base does not need to be a low permeability layer, the alluvium can be left in place (if grading is acceptable, D_{max} < 3mm) and just ripped and re-compacted to 95% MOD AASHTO maximum dry density at ± 2 % OMC.

2. Protection Layer

Since a GMB will be placed on top of the base preparation layer, which may contain irregularities as noted above, it will require protection. It is proposed to provide a GTX-NW (see Table 7-5 for specification) or a 100 mm sand layer (if sufficient volume is available).

Table 7-5: GTX-NW Protection layer specification

Property	Test Method (ASTM)	Unit	Value
Mass per unit area	D5261	g/m^2	1080
Grab tensile strength	D4632	kN	4.5
Grab tensile elongation	D4632	%	50
Trap. tear strength	D4533	kN	1.5
Puncture (CBR) strength	D6241	kN	11.5

NOTES

1. ALL VALUES ARE MINIMUM AVERAGE ROLL VALUE (MARV)
2. GEOTEXTILE MASS PER UNIT AREA SELECTED USING THE GRI -GT12(A)*- ASTM VERSION STANDARD SPECIFICATION

3. HDPE GMB

A 1.5 mm thick HDPE GMB layer will be placed on the protection layer.

4. GTX-NW Protection Layer

Due to the fine PSD of the tailings, a protection layer will not be required on the GMB over the majority of the basin. However, to prevent the granular material of the finger drain system to damage the GMB, a GTX-NW will be placed on the GMB in the drains. The specification of the Protection layer GTX-NW is as listed in Table 7-5.

5. Under Drainage System (Leachate Collection)

A drainage system consisting of granular material (13 mm stone, 6 mm stone and sand layers), surrounding perforated pipes, will be placed on the GTX-NW Protection Layer. The number of and spacing of the pipes will be determined with the seepage analysis.

7.6 BARRIER SERVICE LIFE ASSESSMENT

The service life for a waste disposal facility barrier system can be considered the end of waste disposal due to the recharge of the phreatic surface due to excess moisture plus the rainfall, until the facility is closed by means of a capping layer reducing rainfall infiltration. The current facility is designed to reach capacity after 20 years; however, closure generally can take much longer to be implemented (100 years).

For a geomembrane the main factors affecting service life is UV exposure as well as temperature. Whilst the geomembrane will be covered during construction, therefore UV exposure can be omitted, temperature will have a degradation effect. The temperature of the flow is assumed to be 25° based on monitoring undertaken at facilities with similar designs where monitoring on the drainage outlet has shown an average temperature of 24.76° over a period of one month in summer. Based on research conducted by the Geosynthetics Institute (Geosynthetic Institute, 2011), it is reported that a geomembrane in unexposed conditions at 25° will have a design life of more than 250 years. It shall be noted that such time is for the geomembrane to reach the so called “half-life”, meaning the antioxidant in the geomembrane has reached 50% of their original value.

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

7.7 CONCLUSION

The proposed barrier system will make use of the low permeability of the tailings to build an inverted barrier system which results in leakage rates much lower than a typical Class C barrier system.

In conclusion the following barrier system is proposed:

- Inverted Barrier – 1.5 mm GMB covered with tailings.
- Base preparation underneath the GMB.

8.0 HYDROGEOLOGY

A hydrogeology study was performed on the proposed Phase 2 TSF. This study was an expansion onto the Phase 1 TSF study completed in April 2023 (Knight Piesold, 2023). The Phase 2 hydrogeology study was conducted with the use of numerical transport model.

8.1 MODEL CALIBRATION

A numerical groundwater model of the current TSF at Gamsberg Zinc mine was constructed to evaluate the potential impacts on the groundwater system and potential receptors surrounding the TSF. The 3-D numerical flow and contaminant transport model was designed using the programme FEFLOW, to determine the possible extent of migration of any potential contaminant plume for the current and phase 2 TSF extension at Gamsberg Zinc mine. Two scenarios were included in the contaminant transport model:

- The first scenario is representative of the future phase 2 TSF operational period at Gamsberg Zinc mine
- The second scenario includes the post operational period at Gamsberg Zinc mine
- Although the TSF has been lined with a HDPE geomembrane liner system, as a worst-case scenario, the TSF was modelled with circular defects or tears i.e. represent leakage from the liner

To calibrate the model, input data was obtained from various studies, reports (monitoring, geochemical, geotechnical, and hydrological) on the Gamsberg Zinc Mine. The steady-state flow calibration was conducted by making minor changes to the model input parameters, mainly the permeability and recharge as well as the storage coefficients to simulate the current groundwater flow conditions.

The current groundwater monitoring data for the boreholes surrounding the TSF were incorporated into the steady state calibration, the groundwater concentrations for SO4 and Zn were used to ensure that the developed plume is consistent with the actual groundwater concentrations. The steady state calibration represents a six year period for the current operational phase of the TSF at Gamsberg Zinc mine. Over the six-year period, two solute migratory patterns for both parameters (SO4 and Zn) are evident with flow towards the north west (NW) and south east (SE). The main component of the plume is located towards the NW, while a small component of flow is evident towards the SE. The modelled concentrations were consistent with the borehole data, and the initial calibration was finalised. The results of the steady state calibration are shown in Figure 8-1 below.

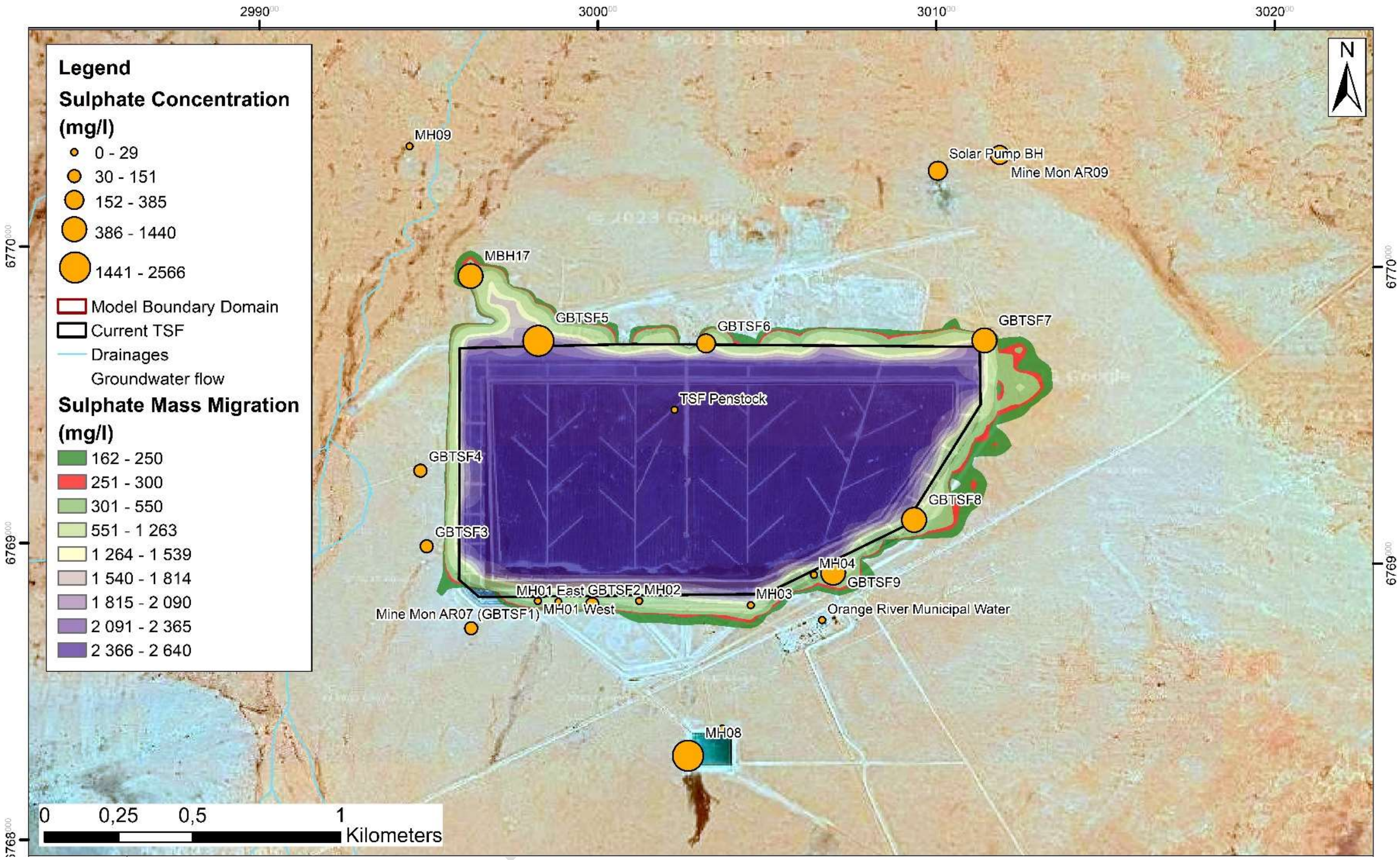


Figure 8-1: Numerical model Steady State Calibration

8.2 PREDICTIVE SIMULATIONS

The first transient simulation included a 13-year period that represents that expected life of design for the phase 2 TSF extension at Gamsberg Zinc mine. The inputs have remained the same as the steady state calibration input, with additional source concentrations applied to the new TSF extension footprint.

The current TSF at Gamsberg has been lined with a 1.5mm thick High-Density Polyethylene (HDPE) geomembrane with a 200 mm thick soil layer liner system.

The second transient simulation includes a 30 year period that represents the post operational period at Gamsberg Zinc mine. For this scenario the TSF at Gamsberg Zinc mine is no longer in operation, with no new deposition occurring. The model inputs have changed for this scenario, with lower source concentrations assigned to the TSF facilities as well as removing the increased recharge component on the TSF (flux).

The results for both Scenarios 1 and 2 are shown graphically in Figure 8-2 and Figure 8-3below:

Both the scenarios show that the lined TSF, both current and phase 2 extension, have kept the high concentrations within their boundaries, showing the effectiveness of the modelled liners.

The modelled simulations (Scenario 1&2) show migratory paths of the potential plume from the TSF sources towards the north and north west, as well as towards the south east. The hydraulic gradient is towards the south for this area and is the expected direction for any potential plume to migrate.

Overall, the spatial extent of the modelled plumes is within a 250 m radius of the TSF, both the current and phase 2 extension. The vertical extent of the plume for the TSF could reach a 65 mbgl, indicating the potential to impact the shallow and deep aquifers local to the area. The current simulations show that the risk of the potential contaminant plume from the TSF impacting any groundwater users is low.

Following the development of the numerical model at Gamsberg Zinc mine, KP recommends the following:

- Continue the quarterly groundwater monitoring at Gamsberg Zinc mine, this will ensure that any leak and/or contamination will be detected, and the correct mitigation measure can be implemented effectively.
- The current borehole monitoring network infrastructure must be maintained at the Gamsberg Zinc mine, particularly the boreholes surround the TSF to identify any increasing trends. Particularly NW of the current TSF.
- The numerical flow and transport model should be updated annually with the new monitoring data as a management tool so that any mitigation that may be required can be modelled and planned timeously.
- Several of the boreholes could be used as potential scavenger boreholes to act as seepage capture for the future operations if required. Particularly the boreholes located S and SE of the TSF (GBTSF 8 and 9).
- Following the construction of the phase 2 TSF extension, it is recommended that monitoring boreholes are installed north of TSF to ensure that any leak and/or contamination is detected. Four proposed locations are identified in Figure 8-3.

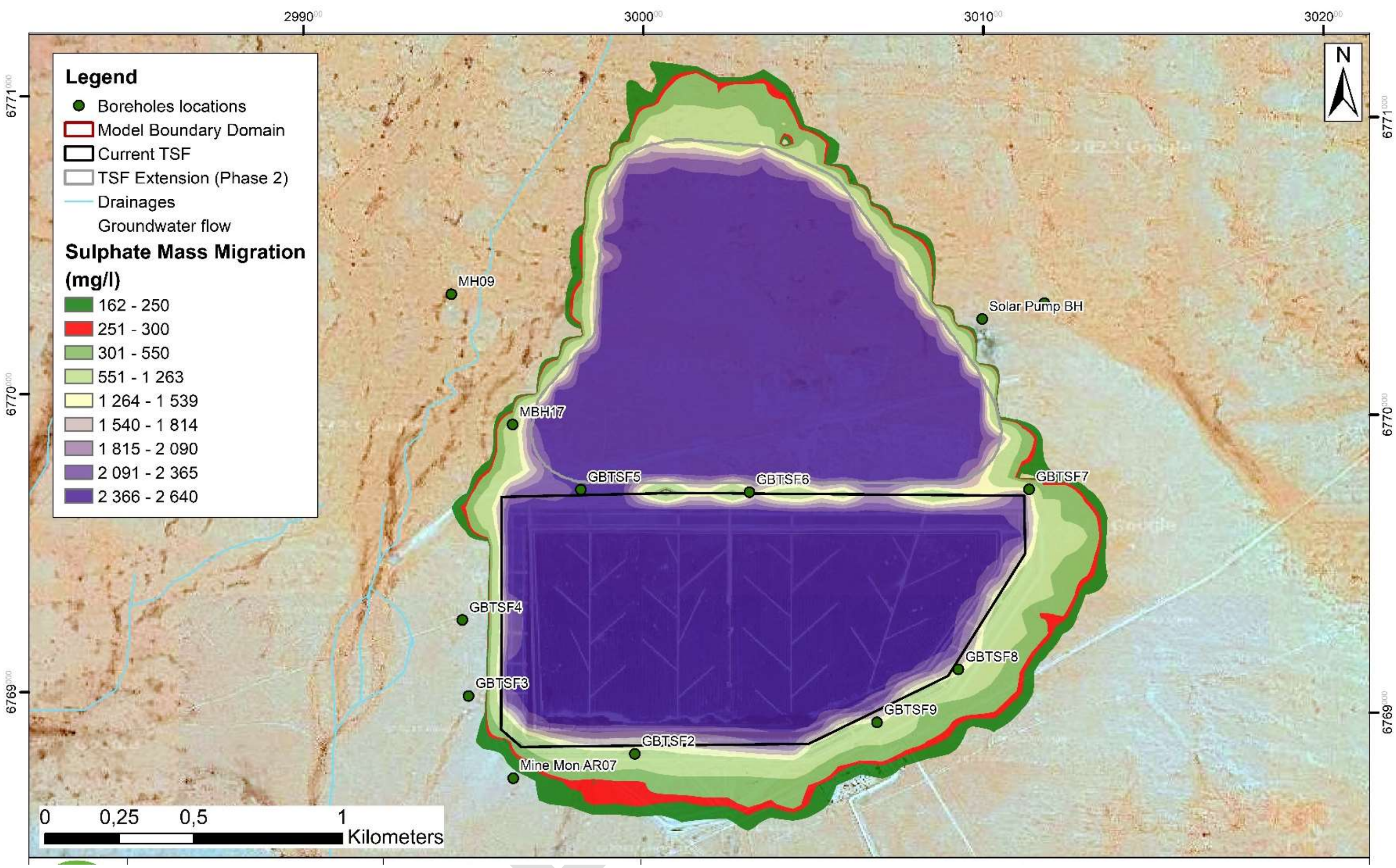


Figure 8-2: Scenario 2-Sulphate Simulation

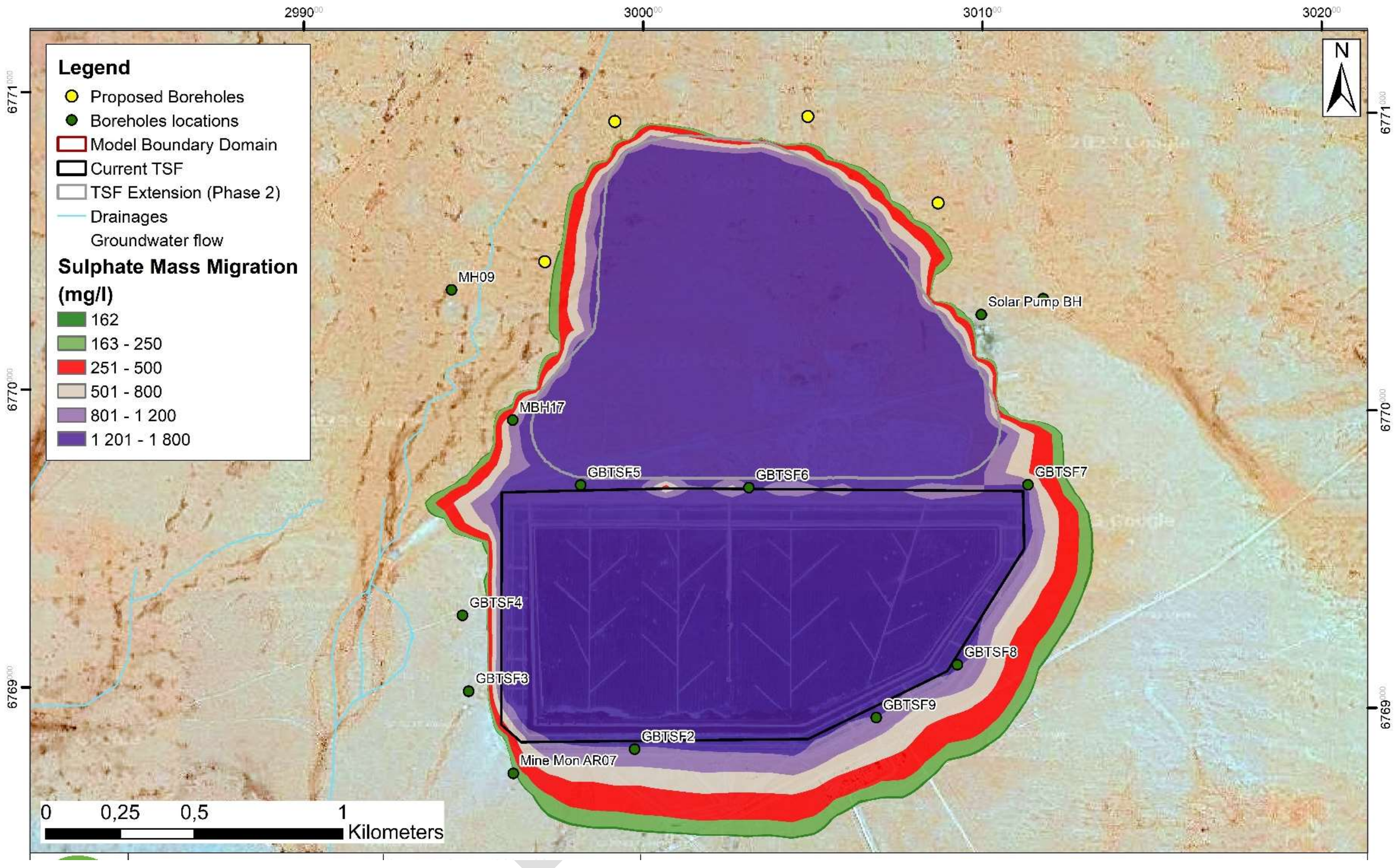


Figure 8-3: Scenario 2-Sulphate Simulation

9.0 CAPACITY ASSESSMENT

The TSF footprint was limited to the approved water use license (WUL) area to the north of the Phase 1 TSF. This defined WUL area is presented in Figure 9-1.

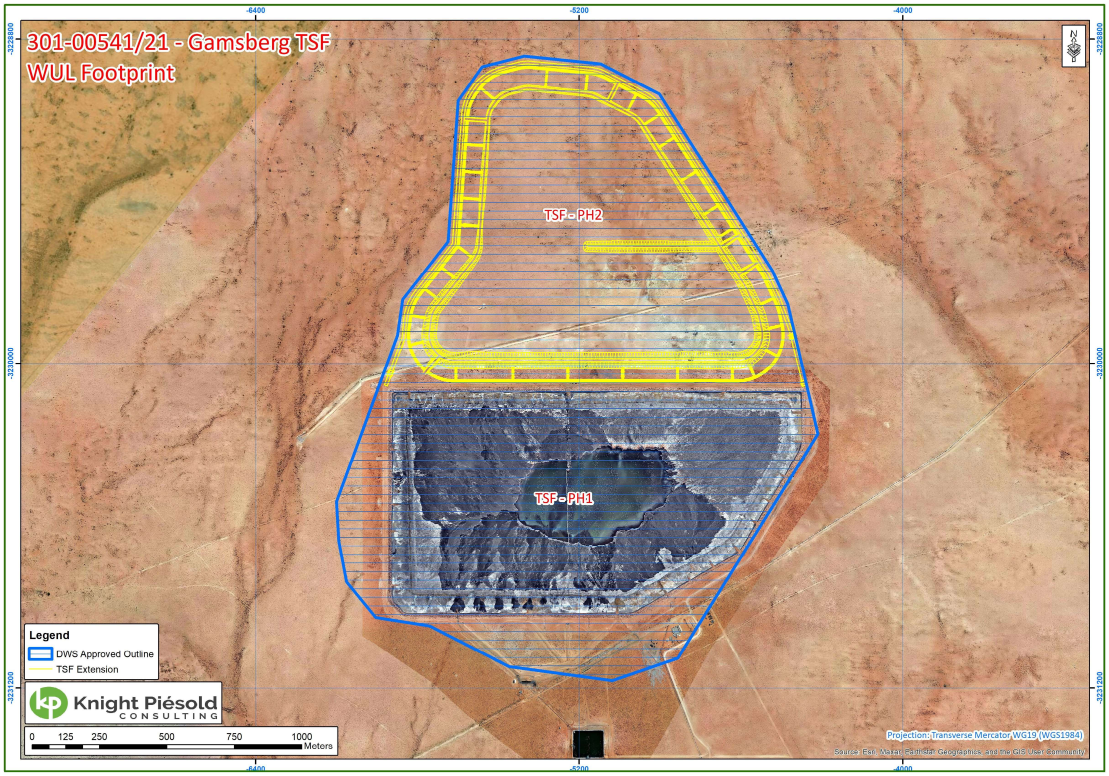


Figure 9-1: Approved Water Use License Footprint

Two options were explored during the preliminary design stages namely abut Phase 1 TSF northern wall or design an independent TSF north of Phase 1 TSF. Both options had sufficient capacity to contain the design tonnage of 48 mega tonnes. It was decided not to abut Phase 1 TSF due to:

- The current state of Phase 1 wall development. Insufficient underflow down streaming has taken place on Phase 1 up to date. This would prevent Phase 2 TSF design utilising the wall as is. No placing of wet overflow material onto this wall can take place as it would cause serious stability issues. Therefore, a sizable starter (heel) wall would still be required for Phase 2 TSF. While Phase 1's toe wall could be utilised for Phase 2 with this option, little cost saving was possible from this solution.

- Use of underflow from Phase 2 onto this undeveloped northern wall was also explored, the underflow split for this option showed a possible solution but required the full mass split currently achieved on Phase 1. This ,however, would not allow for redundancy during start up. New TSF’s require redundancy due to plant start up conditions which produce highly variable tailings, often with increased water content.
- Phase 1 TSF has more water contained within its basin then was designed for and this has likely affected its stability (Stability investigation underway, yet to be completed).
- The foundation north of the Phase 1 TSF is categorised as very hard. This foundation would require blasting to get the underdrains out of the Phase 2 TSF, particularly on the southern side of the TSF which requires deep excavations to get the required underdrain slopes.
- There was uncertainty at the time on whether Phase 1 TSF waste was classified as Type 3. Mixing of various waste types on one TSF would require the more onerous waste type to be designed for and therefore

The options were modelled in two software packages Muk 3D and Rift. The software packages produced similar overall volumes and rate of rise data. Once the stability assessment described in Section 16.0 was completed, the overall slope required to balance stability and capacity was determined to be 1V:3.5H which equates a 1V:3H slope with 5m wide benches at 10m intervals. The Muk3D model is presented in Figure 9-2, the stage capacity curves from this model are presented in

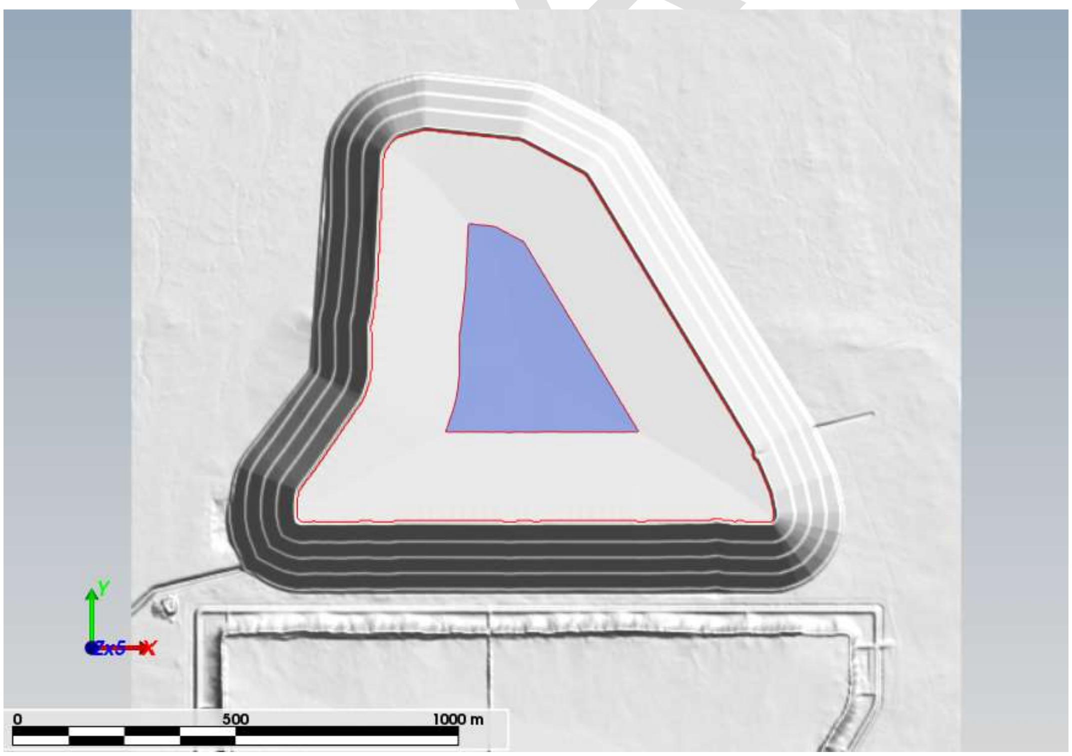


Figure 9-2: Muk3D TSF Model

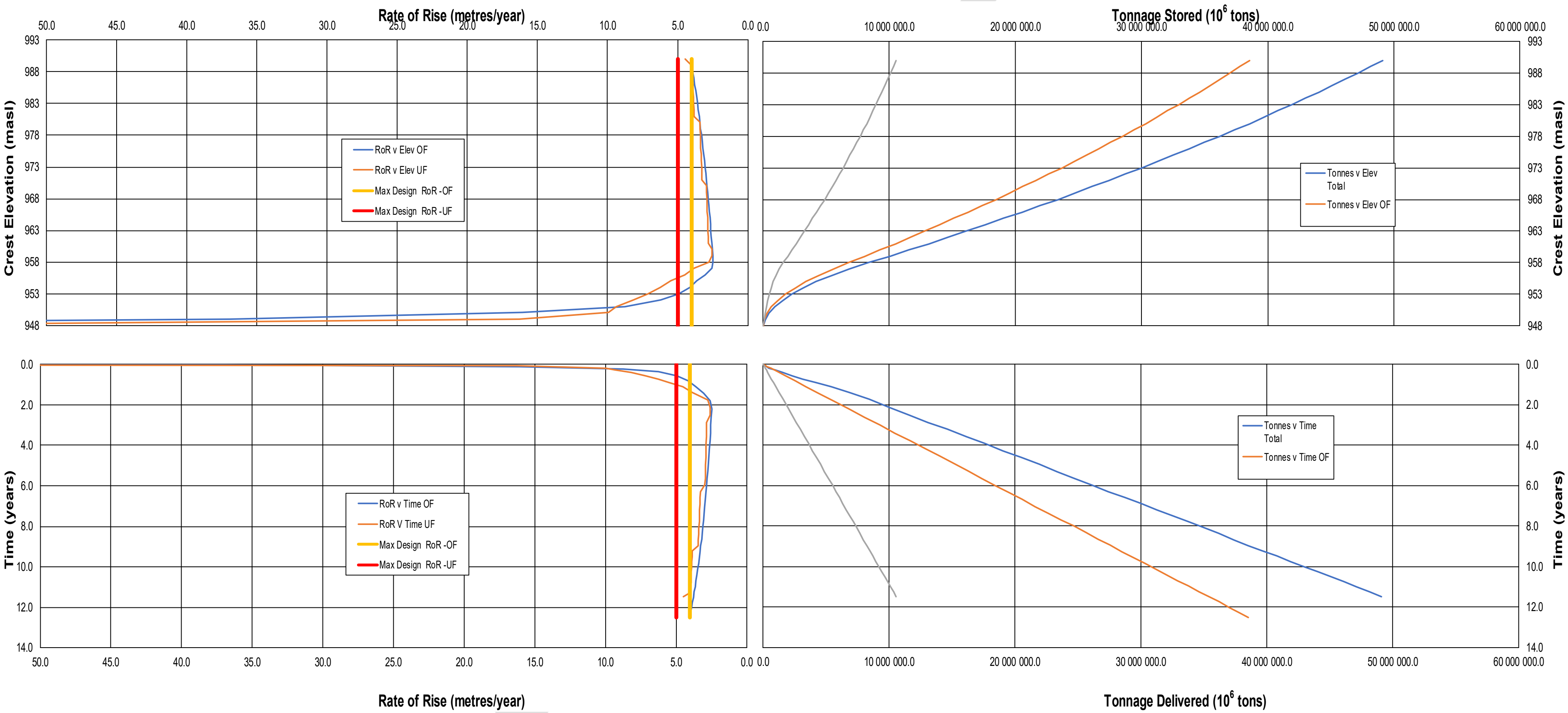


Figure 9-3: TSF Model Stage Capacity Curves

10.0 SEEPAGE MANAGEMENT

Underains have been provided above and below the liner. The drains above the liner will facilitate drainage and reduce the over all pore pressure regime, increasing water recovery and the stability of the facility. The drains below the liner have been designed to detect any potential leakage from the primary barrier. Typical drain details are presented in Figure 10-1.

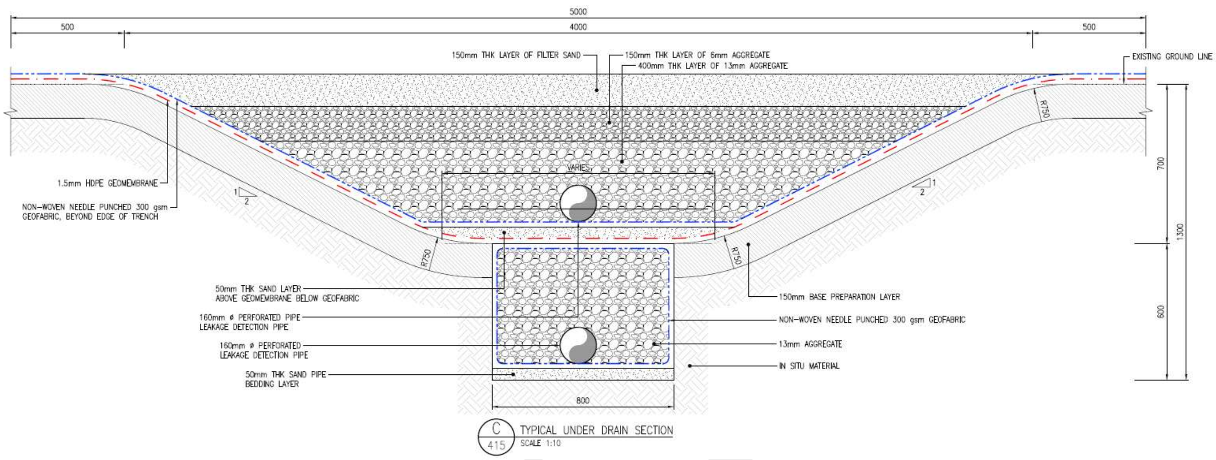


Figure 10-1: Typical Underdrain Details

A filter envelope was determined for the drain's material using filter criteria established by Sherard. The bands are within the technical specification document within Appendix G.

11.0 SEEPAGE AND STABILITY ANALYSIS

11.1 DESIGN CRITERIA

The slope stability design criteria were taken as per ANCOLD requirements. Factors of safety (FOS) as required by ANCOLD (2019), are as follows for the various analyses:

- Drained analysis: 1.5
- Long-term undrained: 1.3 (with no potential loss of containment)
- Post-earthquake: 1.1
- Seismic: 1.1

The long-term analyses typically require:

- effective stress strength parameters for the embankment fills, and
- undrained strength parameters at yield for the tailings and upper foundation soils.

The post-earthquake analyses typically require:

- a reduced effective stress strength for the foundation soils and embankment fills, (typically 20 % reduction), and
- residual (liquefied) undrained strength parameters for the tailings.

The 20 % reduction for the post-earthquake analyses, is a typical industry norm.

The stability analyses were carried out using Slide2 and RS2 software from Rocscience.

11.2 PROFILE AND PARAMETERS

The soil profiles generally consist of residual soils to very soft rock gneiss with nodular to hardpan calcrete in the upper portion of the soil horizon, overlain by aeolian sand. The typical soil profile is discussed as follows and the parameters are presented in Table 11-1:

- Aeolian sand deposits virtually cover the entire site. It comprises silty sand. The aeolian material has a very loose consistency. The layer thickness varies between 0.1 m to 0.4 m across the site. This was removed during construction.
- Calcrete occurs within the residual soil. The calcrete varies in stages of development from nodular calcrete to hardpan calcrete. The nodular calcrete occurs either above or below the honeycomb to hardpan calcrete. Layer thicknesses vary between 0.4 m to 2.5 m. Nodular calcrete has a dense consistency and consists of silty sand with abundant calcrete nodules.
- Honeycomb calcrete occurs in most of the test pits and transitions to a hardpan calcrete in some test pits. Honeycomb calcrete concretions are very dense very strongly cemented sandy gravel with irregular calcite concretions.
- Hardpan calcrete occurs as very dense very strongly cemented sandy gravel layer. Refusal occurred on the hardpan calcrete in the test pits mentioned above.
- Hardpan ferricrete occurs as very dense very strongly cemented sandy gravel. Refusal occurred in some test pits at a depth between 0.1 m to 0.3 m below ground level.
- Very soft rock gneiss occurs below the calcrete/ferricrete layer and was excavated as silty sandy gravel. The very soft rock gneiss has a foliated structure.

- Refusal occurred on soft rock gneiss in all the test pits which were not overlain by hardpan calcrete/ferricrete. The refusal depth varies between 1.0 m – 3.1 m. The soft rock gneiss is characterized by a distinct foliated structure.
- No groundwater seepage was encountered in any of the test pits.

Table 11-1: Geotechnical parameters used in stability assessment

Description	Unit weight (kN/m³)	Peak Eff Stress		Post-Seismic Eff Stress		Peak Cohesion (kPa)	Permeability (m/s)
		Friction Angle [deg]	Undrained Peak Ratio	Friction Angle [deg]	Undrained Residual Ratio		
Tailings – Overflow	17	29	-	23	-	0	3.77E-08
Tailings - Overflow UNDRAINED	17	-	0.14	-	0.09	0	3.77E-08
Tailings - Underflow	17	31	-	25	-	0	9.75E-07
Tailings – Underflow UNDRAINED	17	-	0.15	-	0.09	0	9.75E-07
Dense Sand	20	37	-	30	-	0	1.00E-05
Calcrete	20	34	-	-	-	0	1.00E-07
Bedrock	22	40	-	-	-	10	1.00E-07
Toe Drains	17	35	-	28	-	0	5.00E-04
Embankment	20	34	-	-	-	0	1.00E-07
Tailings/Double Textured HDPE interface	9	22	-	22	-	0	1.00E-15
Tailings/Smooth HDPE interface	9	10	-	10	-	0	1.00E-15
Calcrete / HDPE interface	9	20	-	20	-	0	1.00E-15
Double Textured HDPE / Geotextile interface	9	30	-	30	-	0	1.00E-15
Smooth HDPE / Geotextile interface	9	15	-	15	-	0	1.00E-15
Geotextile/Calcrete interface	9	18	-	18	-	0	1.00E-15

NOTES:

1. VSR = VERTICAL STRESS RATIO
2. THE PERMEABILITY PARAMETERS WERE FROM LAB RESULTS.

The Laboratory results of the Gamsberg tailings indicates a friction angle of 29° for the Fine Tailings and 31° for the Coarse tailings. It was however assumed that the tailings generally would have no cohesion. The undrained shear strength parameters were assumed based on similar material on other projects.

Triaxial and CSL test results indicated undrained peak values ranging from 0.15 to 0.22 for underflow tailings and from 0.14 to 0.19 for overflow tailings. Undrained residual results ranged from 0.04 to 0.19 and 0.09 to 0.14 for underflow and overflow tailings respectively.

The toe drains were added to the stability model and was analysed as follow:

- All drains are in working condition for the Peak and Post-Peak condition.
- Heel drain blocked for the Peak Condition.
- All drains blocked for the Peak Condition.

As an upset condition the effect of the internal finger drains were not considered in the stability analysis.

The strength of the embankment fill (walls) was not reduced with the typical 20 % reduction for post-earthquake analyses, since these walls were constructed in 200 mm thick, compacted layers.

Each of the tailings materials have an undrained equivalent, which uses the undrained shear strength (vertical stress ratio) instead of the Mohr-Coulomb parameters for the undrained analyses. The undrained parameters were only assigned to material under the phreatic surface, which is assumed to be undrained.

Initially a conservative horizontal seismic coefficient of 0.1 was used for the seismic analyses. According to a recently submitted Seismic Hazard Assessment memorandum (Knight Piésold, May 2022), for an annual exceedance probability of 1:10 000, representing the design earthquake for the TSF, the horizontal seismic coefficient is 0.08 (50% of the PGA of 0.154 g). This lower value was used in some of the analyses (when the resulting FoS was too low when 0.1 was used).

The pool position was estimated to be 200m from the crest, assuming a 1V:200H beach slope with 1m freeboard.

The stiffness parameters used in the settlement analyses are presented in Table 11-2. These values were estimated from typical values (Look, 2014).

Table 11-2: Stiffness parameters used in stability assessment

Description	Poison's Ratio	Young's Modulus (kPa)
Tailings - Fine	0.35	15 000
Tailings - Coarse	0.35	30 000
Dense Sand	0.35	40 000
Calcrete	0.25	70 000
Bedrock	0.35	120 000
Toe Drains	0.30	20 000
Embankment	0.25	70 000

The barrier system was presented as a weak layer of low permeability material, with a friction angle of 10° for smooth and 20° for textured liner as the worst-case interfaces. The smooth barrier properties were used on the basin and the textured barrier properties over the walls.

This was the best approximation with the software capability. RS2 has the functionality to incorporate a barrier system into the model by adding a Structural Interface, with joint elements above and below the support element as presented in Figure 11-1. The support element is the HDPE geomembrane, and the joint elements are the tailings or soil, above or below the liner.

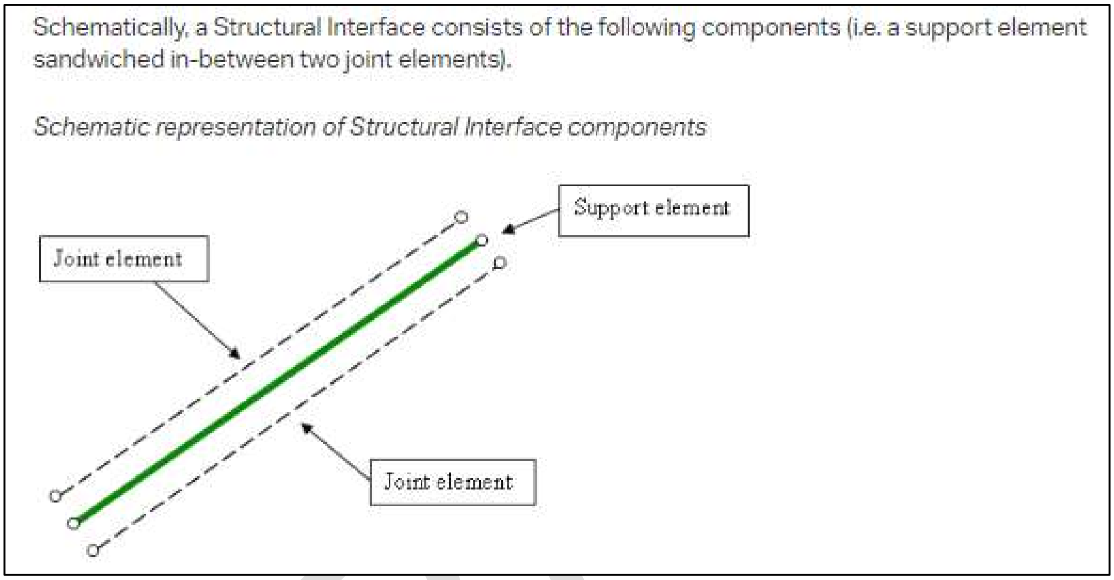


Figure 11-1: Adding a liner to the model (RS2 Help File)

The following parameters were assigned to the barrier system:

Table 11-3: Liner system parameters

Parameter	Unit	Value
Tensile Modulus	kN/m	150
Ultimate Tensile Strength	kN/m	6.28
Creep Reduction Factor	-	1.1
Installation Reduction Factor	-	1.1
Deterioration Reduction Factor	-	1.1
Factor of Safety	-	1.1
Allowable Tensile Strength	kN/m	4.19

Table 11-4: Barrier system parameters

Parameter	Unit	Tailings Smooth	Tailings Textured	Sand (Bottom)
Peak Cohesion	kPa	0	0	0
Peak Friction Angle	°	10	20	18

11.3 SEEPAGE ANALYSIS

The hydraulic parameters combined with a finite element analysis were used to determine the phreatic surface needed for the slope stability analysis. The analysed cross section is displayed in Figure 11-2 and Figure 11-3.

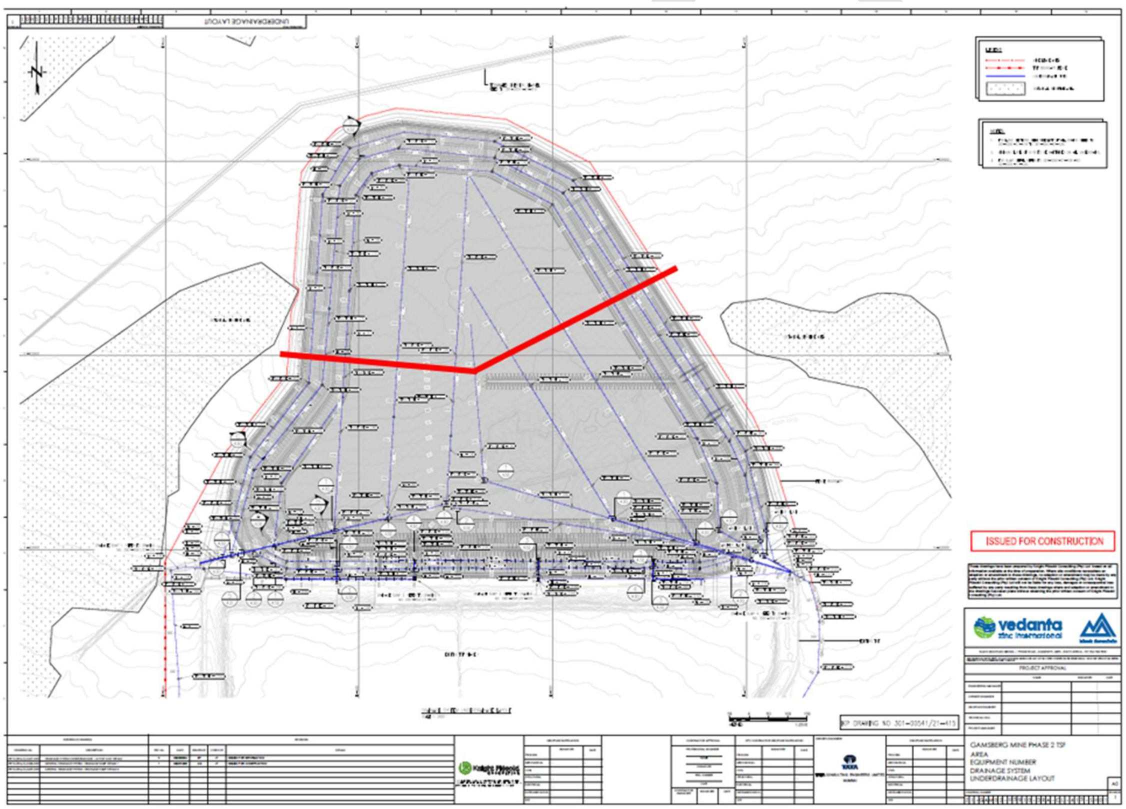


Figure 11-2: Plan of analysed cross section

The seepage rates of the drains were analysed by running a full elevation height model as well as a model that has only been constructed to 5m above the starter wall. The design volumetric flow rates of the drains are presented in Table 11-5.

Table 11-5: Design Volumetric Flow Rates

Drain position	**Volumetric Flow Rate [m^3/s/m length]**
External Toe Drain (Heel drain)	5.00E-08
Internal Toe Drain (Starter wall drain)	1.00E-07
Internal Drains (Finger drains)	5.00E-07

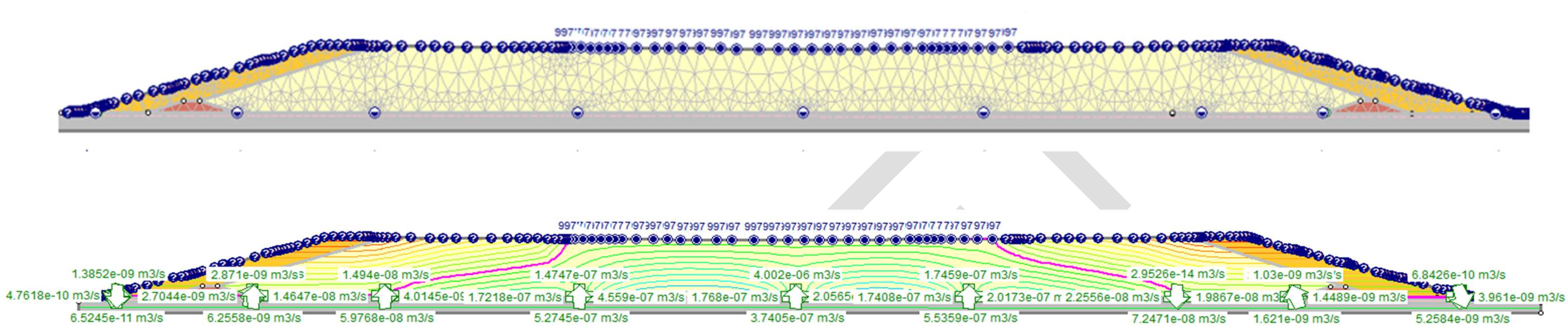


Figure 11-3: Analysed cross section

11.4 STABILITY ASSESSMENT

Slope stability analyses were carried out using the computer software Slide 2 Modeler (which forms part of the Rocscience software package). The limit equilibrium was applied through the method of slices as defined by Morgenstern-Price. The output of the analyses section is a Factor of Safety (FoS).

The results of the various analyses are presented in

Table 11-6.

Table 11-6: Stability Assessment Results

Condition	Drain Condition	Achieved Factor of Safety	Required Factor of Safety
Peak - Drained	Drains working	2.0	1.5
Peak - Undrained	Drains working	1.5	1.3
	Heel drain blocked [1]	1.2*	1.1*
	All drains blocked [1]	1.2*	1.1*
Residual - Undrained	Drains working	1.2	1.1
Seismic (Peak – Drained)	Drains working	1.4	1.1

NOTES

1. IN THIS SCENARIO THERE IS A DOUBLE UPSET CONDITION. THE SCENARIO IS A COMBINATION OF UNDRAINED PARAMETERS WHILE AT THE SAME TIME DRAINS ARE ALSO BLOCKED.

11.5 SETTLEMENT AND STRAIN ASSESSMENTS

RS2 were utilised to determine the magnitude of expected displacement on the TSF as well as the maximum shear strain within the HDPE liner during the critical Strength Reduction Factor (SRF) event.

Small strain theory implies that there is no distinction to be made between the undeformed and deformed configuration. Since RS2 uses Cauchy strain (or engineering strain) definition, the results will be accurate up to a strain of 1%. The accuracy of the results decreases as the values increase beyond 1%. The following results were obtained:

Table 11-7: SRF and Displacement Assessment Results

Condition	Drain Condition	Maximum Displacement [m]	Maximum Shear Strain	Critical SRF (Strength Reduction Factor)
Peak - Drained	Drains working	0.14	0.01	2.02
Peak - Undrained	Drains working	0.24	0.01	2.02
	Heel drain blocked	0.38	0.22	1.73
	All drains blocked	0.31	0.12	1.60
Residual - Undrained	Drains working	0.01	0.03	1.09
	Heel drain blocked	0.35	0.27	1.73
	All drains blocked	0.32	0.12	1.60

Table 11-8: Shear Strain in the Liner

Condition	Drain Condition	Maximum Shear Strain	SRF (Strength Reduction Factor)
HDPE Liner	Drains working	1.65 %	2.02
		1.43 %	1.00

11.6 RETURN WATER DAM STABILITY

11.6.1 PROFILE PARAMETERS

Table 11-9: Geotechnical parameters used in stability assessment

Description		Unit weight (kN/m³)	Friction Angle (deg)	Peak Cohesion (kPa)	Permeability (m/s)
Calcrete		20	34	0	$1.00E^{-07}$
Bedrock		22	40	10	$1.00E^{-07}$
Geocells	X	24	40	5	$1.00E^{-10}$
Smooth HDPE / Geotextile Interface		9	15	0	$1.00E^{-15}$

11.6.2 STABILITY ANALYSIS

Slope stability analyses were carried out using the computer software Slide 2 Modeler (which forms part of the Rocscience software package). The limit equilibrium was applied through the method of slices as defined by Morgenstern-Price. The output of the analyses section is a Factor of Safety (FoS). The results of the various analyses are presented in

Table 11-10.

Should a tear in the liner occur a higher risk of failure has been determined during rapid drawdown. It should be noted that due to the sand-like calcrete foundation of the RWD any leakage is be expected to dissipate into the surrounding soil and should not result in significant pore water pressures within the divider wall. The RWD also has a double liner barrier system and is protected by a hyson cell layer, this scenario is considered as an additional upset test.

Table 11-10: Stability Assessment Results (West – East section)

Condition		Achieved Factor of Safety	Required Factor of Safety
Full Supply Level	Downstream	1.7	1.4
	Upstream	3.0	1.4
	Divider wall	2.6	1.4
Empty Supply Level / After Construction	Downstream	1.7	1.3
	Upstream	2.2	1.3
	Divider wall	2.2	1.3
Rapid Drawdown	External wall	1.1	1.25
	Divider wall	1.1	1.25

NOTES

1. DUE TO THE SAND-LIKE CALCRETE FOUNDATION OF THE RWD ANY LEAKAGE IS BE EXPECTED TO DISSIPATE INTO THE SURROUNDING SOIL AND SHOULD NOT RESULT IN SIGNIFICANT PORE WATER PRESSURES WITHIN THE DIVIDER WALL.

12.0 SURFACE WATER MANAGEMENT

12.1 DECANT SYSTEM

The decant system design is currently underway. The system will be an electrically supplied land-based pump system with a floating suction head. The system will supply water from the TSF pool to the silt trap prior to the return water dam. The return rate will be 800m^3/hr.

12.2 BENCH DECANTS

Bench decants with a head wall and removable sluice gates have been designed for the TSF benches. They are spaced approximately 200 m apart and are situated at the outside TSF bench crest. At each outlet, the bench should slope at 0.5% toward the decant, the remaining bench areas should slope upstream at 0.5%. They have been sized for the 1:100-year 24-hour rainfall event. The use of the sluice gates will depend on the occurrence of spills and/or pipe bursts which should be prevented from leaving the crest and contaminating lower slopes which may have been rehabilitated.

12.3 SILT TRAP

Before the TSF decant water enters their respective RWD the water goes through a silt trap in order to allow sediment to filter out of solution. There is an existing silt trap prior to RWD1, and this section shows the design for the new silt trap located just prior to the RWD2.

All dirty water draining to the RWD2 drains into a silt trap which is located in the middle of the two compartments in. The silt trap has been designed to comfortably pass the 50-year storm event. A double silt trap measuring 25 m x 15 m x 1.5 m deep with a drying bed measuring 25 m x 4.7 m wide is located on the northern side of the return water dam. The double silt trap has been designed with sluice gates to control water flow into the relevant silt trap that is operational at the time.

Silt from dirty water runoff is captured in the silt trap, once the majority of silt is removed from the dirty water it is then directed into the return water dam. Cleaning of the silt trap is by means of a TLB. The silt trap has been designed for (8 t vehicle) or similar weight and type of vehicle. In the absence of a vehicle fleet list, the wheel loader considered in the design is a JCB-3CX series with a maximum bucket capacity of approximately 1.1 m^3 and a bucket width of 2.35 m. The vehicle and bucket specifications are to be confirmed with the final vehicle fleet.

Each of the compartments will need to be cleaned out as required based on regular inspections to ensure that no overflow occurs. The wet silt will be loaded onto an adjacent drying bed provided for this purpose from where the dried-out silt must be regularly removed and appropriately dispatched to TSF2.

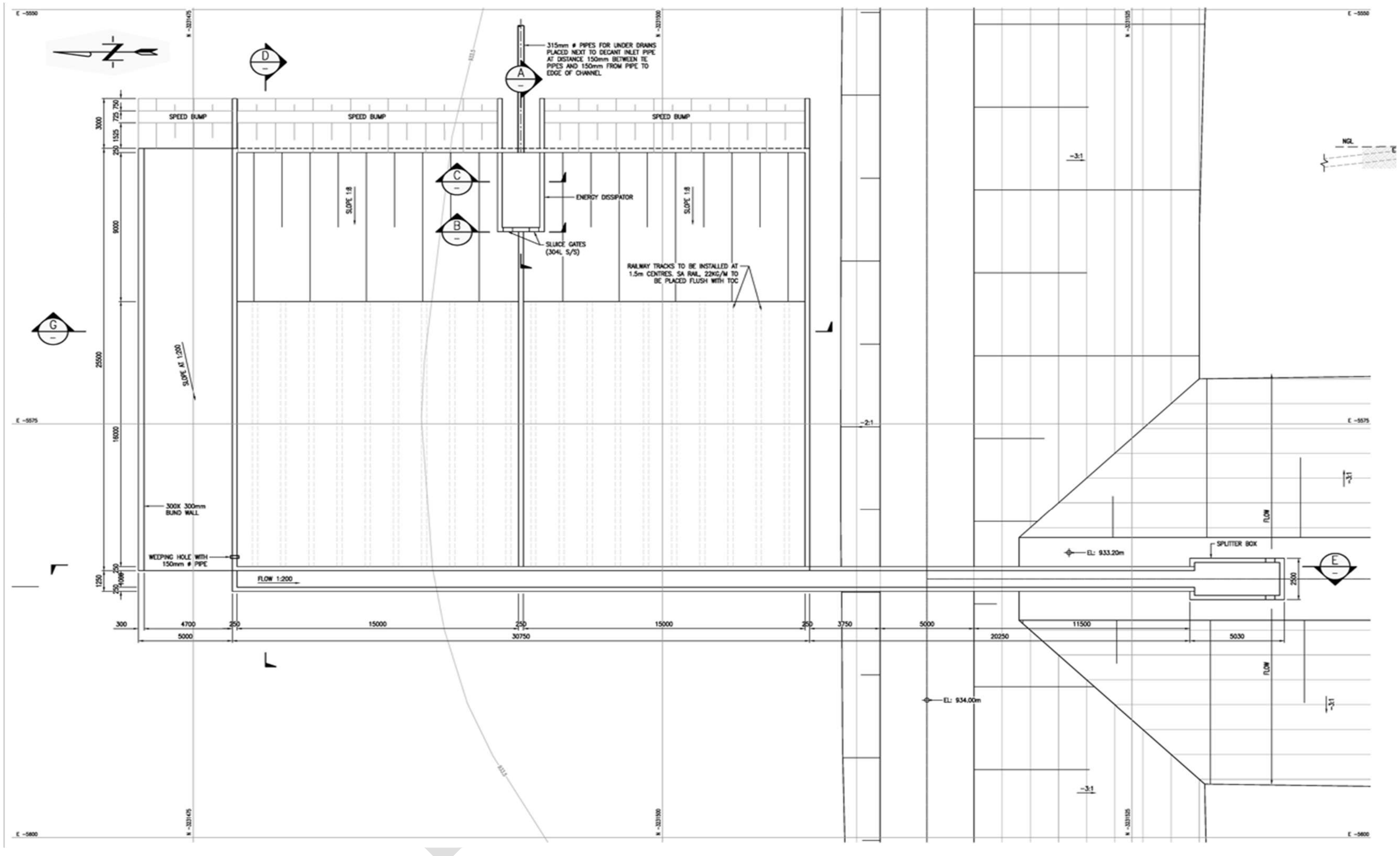


Figure 12-1 Double Silt Trap and Drying Bed at the Return Water Dam

12.4 RETURN WATER DAM

12.4.1 WALL AND BASIN

The return water dam(RWD) was sized to accommodate the water from Phase 1 and Phase 2 for both GISTM and GN 704 compliance as discussed in Section 13.0 of this report. Two separate compartments were designed in order to allow cleaning of an individual compartments. The final size required was 120 000 m^3 with each compartment having approximately 60 000 m^3 of storage capacity.

The RWD will be double lined with 1.5mm HDPE liner, a geonet layer in between the liners with act as leakage detection and drainage layer should the top liner leak. The geonet layer feeds to two sumps at the southern ends of the compartments which can be drained with a pump, this should routinely be checked for leakage. The protection layer is a concrete filled HDPE hyson cells in order to allow cleaning with light machinery such as TLB's. The layout of the RWD is presented in Figure 12-2.

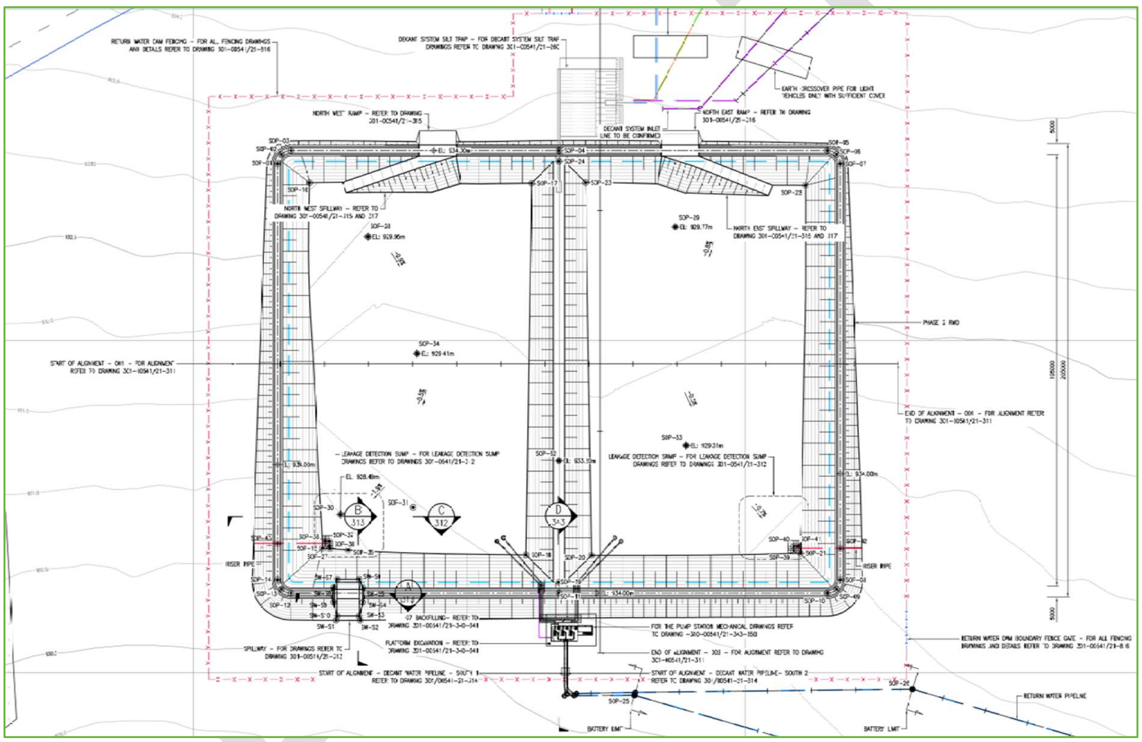


Figure 12-2: RWD layout

12.4.2 PUMP STATIONS

The decant system design is currently underway. The system will be an electrically supplied land-based pump system with a floating suction head. The system will supply water from the TSF pool to the silt trap prior to the return water dam.

12.5 STORMWATER TOE DRAIN

There are existing stormwater toe drains around TSF Phase 1. New stormwater toe drains around TSF Phase 2 have been sized, and these drains flow into the existing toe drains and terminate into the Stormwater Dam (SWD). The contributing catchment areas and toe drains are shown in Figure 12-3.

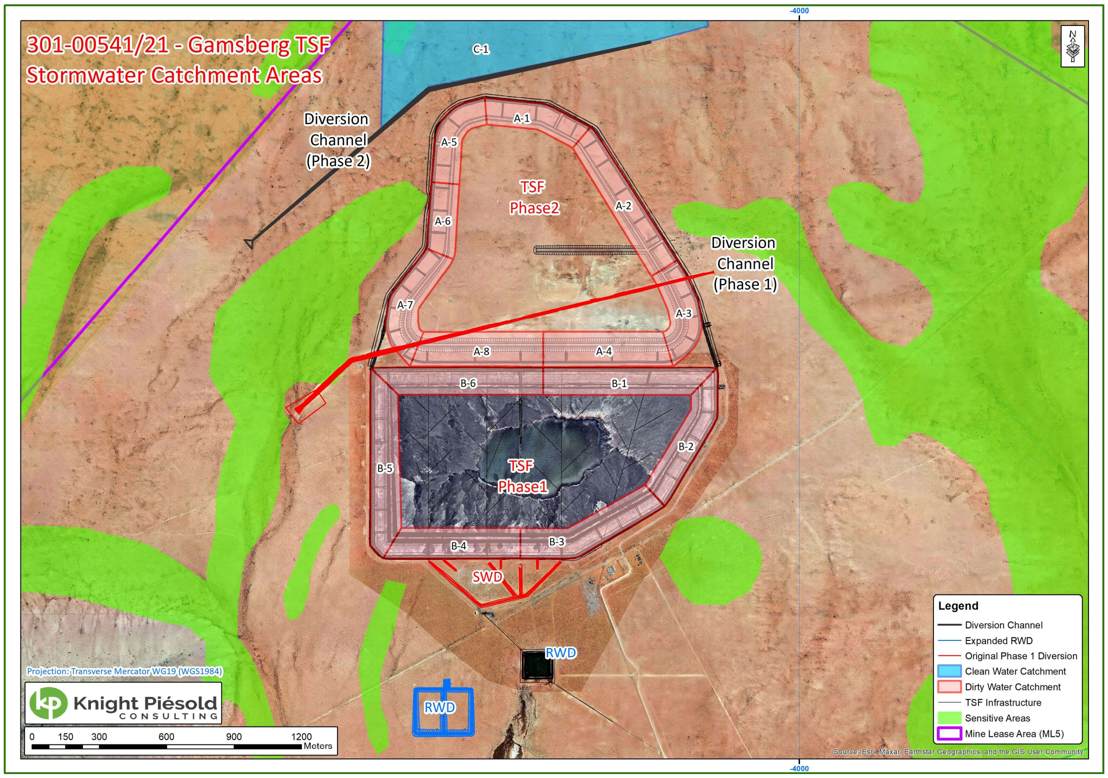


Figure 12-3 Contributing catchments and channel arrangement for both phases

The contribution catchments for Phase 2 were delineated for the East (A1, A2 and A3), West (A-5, A-6 and A-7) and South (A-4 and A-8) zones. It is important to mention that A-3 includes areas A-2 and A-1, A-2 includes A-1, A-7 includes areas A-5 and A-6, and A-6 includes A-5. Table 12-1 shows the salient catchment characteristics used in the Rational Method (RM) for TSF Phases 2. A catchment runoff coefficient of 0.6 was used.

Table 12-1 Catchment characteristics for the toe drains associated with TSF Phase 2

Catchment Name	A-1	A-2	A-3	A-4	A-5	A-6	A-7	A-8
Catchment Area (km²)	0.05	0.14	0.20	0.10	0.05	0.09	0.16	0.09
Length of longest watercourse (km)	0.50	1.20	1.75	0.58	0.50	0.80	1.41	0.59
Slope (-)	0.0026	0.0048	0.0057	0.0002	0.0062	0.0047	0.0070	0.0002
Time of concentration, t_c (h)	0.60	0.78	0.90	1.21	0.49	0.65	0.77	1.23
1:50-year RP 24-hr storm depths (mm)	108	108	108	108	108	108	108	108
1:50-year RP 24-hr storm depths (mm)	126	126	126	126	126	126	126	126
1:100-year RP Flood Peak (m³/s)	**0.23**	**0.52**	**0.67**	**0.26**	**0.25**	**0.37**	**0.58**	**0.23**
1:100-year RP Flood Peak (m³/s)	**0.32**	**0.72**	**0.92**	**0.36**	**0.34**	**0.51**	**0.81**	**0.32**

The contribution catchments for Phase 1 were delineated for the East (B1, B2 and B3) and West (B-4, B-5 and B-6) zones. Table 12-2 shows the salient catchment characteristics used in the Rational Method (RM) for TSF Phase 1. A catchment runoff coefficient of 0.6 was used.

Table 12-2 Catchment characteristics for the toe drains associated with TSF Phase 1

Catchment Name	**B-1**	**B-2**	**B-3**	**B-4**	**B-5**	**B-6**
Catchment Area (km²)	0.086623	0.070135	0.157343	0.085397	0.096058	0.078967
Length of longest watercourse (km)	0.774057	0.67786	1.38265	0.767782	0.861707	0.613477
Slope (-)	0.00072	0.00679	0.00574	0.00064	0.00990	0.00147
Time of concentration, tc (h)	0.99	0.55	0.80	1.02	0.56	0.75
1:50-year RP 24-hr storm depths (mm)	108	108	108	108	108	108
1:50-year RP 24-hr storm depths (mm)	126	126	126	126	126	126
1:100-year RP Flood Peak (m³/s)	**0.27**	**0.32**	**0.57**	**0.26**	**0.44**	**0.30**
1:100-year RP Flood Peak (m³/s)	**0.37**	**0.45**	**0.79**	**0.36**	**0.60**	**0.41**

Figure 12-4 shows the typical cross-section for the toe drains in Phase 2. The trapezoidal drain size has a bottom width of 1.2 m, with side slopes of 1:2 and a depth that varies from 0.2 – 1.8 m.

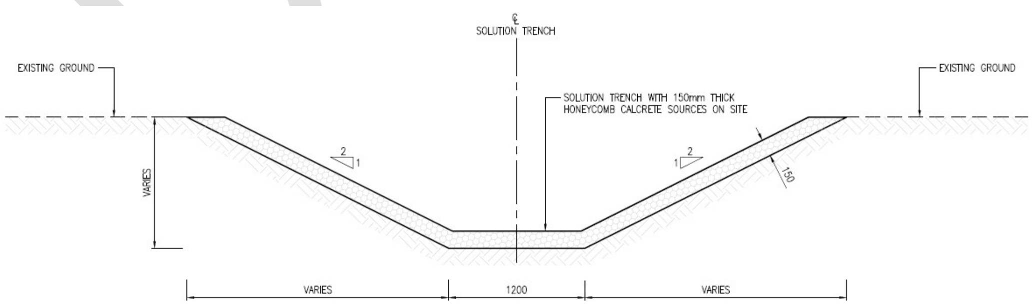


Figure 12-4 Typical cross-section of stormwater drains around TSF 2

The flows that would enter the existing toe drains for Phases 1 and 2 are summarised in Table 12-3.

Table 12-3 Flows for Phases 1 and 2

	Phase 1 Flows			Phase 2 Flows			Both Phases
	West and part South (B4+B5+B6)	East and part South (B1+B3)	Phase 1 Total	West and part South (A7+A8)	East and part South (A3+A4)	Phase 2 Total	Both Phases Total Flow
1:50-year Flows (m^3/s)	0.99	0.84	1.83	0.81	0.93	1.74	3.57
1:100-year Flows (m^3/s)	1.38	1.16	2.54	1.13	1.28	2.41	4.95

12.6 STORMWATER DAM

The Storm Water Dam (SWD) is located to the south of the Phase 1 TSF and has three compartments that are interconnected. The contributing catchment areas, the SWD location and the channel locations are shown Figure 12-3 in Section 12.5.

12.6.1 DESIGN METHODOLOGY

Using an Excel spreadsheet, a daily time step rainfall runoff model for the dirty stormwater catchments was coupled with a daily timestep water balance model for the Stormwater Dam (SWD). The rainfall runoff model is based on the United States Department of Agriculture's Soil Conservation Service's method (USSCS method) to estimate the portion of the rainfall which infiltrates or runs off each catchment type for each day of the simulation. The SWD water balance model considers stormwater inflows and direct rainfall against evaporation losses and a daily abstraction rate and estimates the volume of water in the SWD for each day of the simulation. The USSCS method has been widely used both internationally and locally in South Africa for the estimation of daily stormflow volumes. The USSCS method is an empirical unit hydrograph method.

12.6.2 DATA USED IN THE SWD ANALYSIS

12.6.2.1 CLIMATE DATA

Daily rainfall from the Aggeneys (POL) Rainfall Station (0246555 W) was used in sizing the SWD and is available in Section 3.2. Evaporation data from the D8E005 Pella Mission @ Pella Pump Station was used in sizing the SWD and the data is available in Section 3.3. The Aggeneys station referred to above, which is located within the Aggeneys town was misspelled in the database as Aggeneys, for clarity it has been corrected to Aggeneys within this report.

12.6.2.2 SWD CAPACITY DATA

The SWD area and volume was obtained from the latest survey data of the dams. This data is presented in Table 12-4. The area and storage capacity of the three different compartments were modelled as one unit due to the different compartments being connected by spillways.

Table 12-4 Stage-storage relationship for the SWD

Lower elevation (m)	Upper elevation (m)	Volume (m^3)	Plan area (m^2)
937.0	937.9	0.0	0.0
937.9	937.2	0.0	0.0
937.2	937.4	0.0	0.0
937.4	937.6	0.0	0.0
937.6	937.8	0.0	0.0
937.8	938.0	25.7	558.7
938.0	938.2	354.2	3215.2
938.2	938.4	1294.9	6148.1
938.4	938.6	2904.7	10187.0
938.6	938.8	5407.5	15085.8
938.8	939.0	9062.9	21465.8
939.0	939.2	14003.6	28199.4
939.2	939.4	20483.8	36985.7
939.4	939.6	28479.7	48644.8

12.6.2.3 CATCHMENT RUNOFF DATA

The areas considered to contribute to the SWD were the runoff from the side slopes of the Phase 1 and Phase 2 TSF side slopes. This information is shown in Figure 12-3 in Section 12.5. USSCS curve number (CN) used for the TSF Phase 1 and TSF Phase 2 contributing areas were 72. The contributing areas and the CN (for runoff calculations) that were used to size the SWD are presented in Table 12-5.

Table 12-5 Contributing catchment areas and their associated CN for runoff calculations

Catchment Name	Catchment Area (m^2)	Curve Number (CN)
Phase 1 TSF side slope area	504 400	72
Phase 2 TSF side slope area	538 700	72
Total Combined Area reporting to SWD	1 043 100	72

12.6.2.4 SWD SIZING DETAILS

The abstraction rate from the SWD will need to be pumped out at 4 800 m^3/day or 200 m^3/hour. A freeboard of 0.8 m should be provided above full supply level The below graph (Figure 12-5) shows the daily rainfall (mm), the maximum SWD volume (m^3), the modelled SWD volume (m^3) during the simulation and the modelled spill volume (m^3) during the simulation.

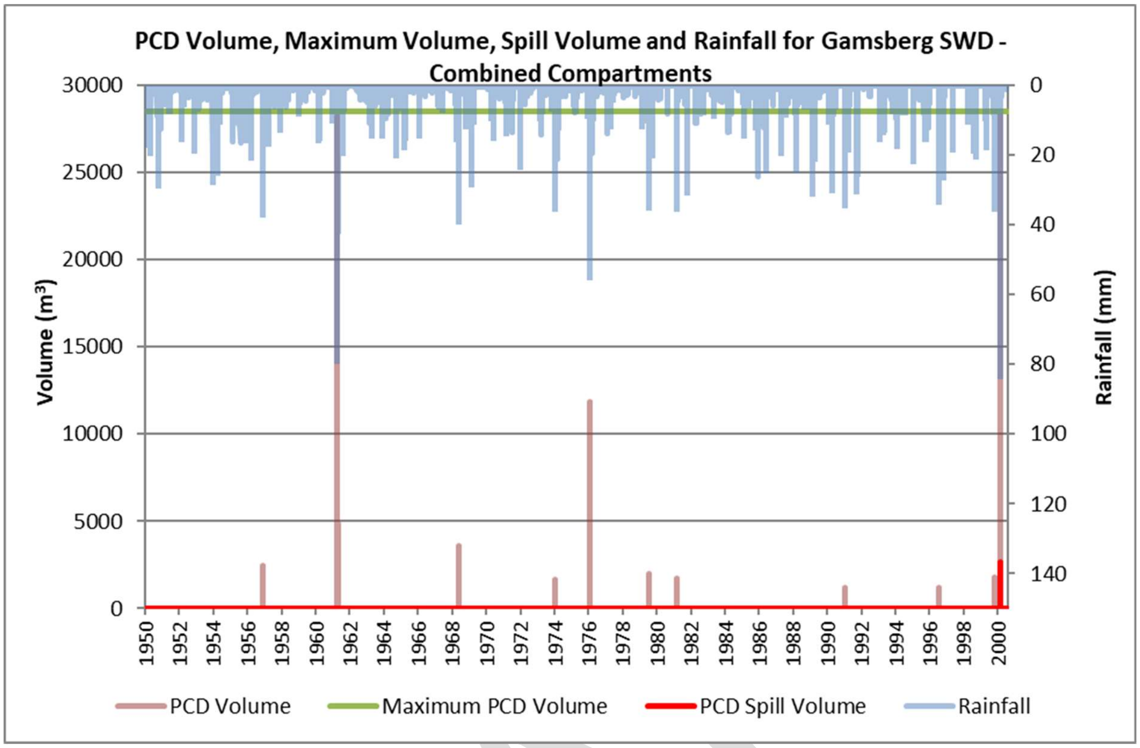


Figure 12-5 The SWD Volume, Maximum Volume, Spill Volume and the Rainfall

12.6.2.5 DISCUSSION

As can be seen from Figure 12-5, there is only one spill event (red) during the entire simulation giving the SWD a spill ratio of less than 2% and is therefore compliant with GN 704. The one spill event in the simulation record (using the above details) has a volume of 2 647 m^3. The spill event occurs on 19 February 2000 after a rainfall event of 83 mm, which equates to a Return Period rainfall event of more than the 1:200-year event (see Table 12-6).

Table 12-6 Return Period rainfall events

Return Period	LP3 (mm)
2	20
5	32.5
10	42
20	51
50	62
100	70
200	78
500	89
1 000	97
2 457	107.5
5 000	115.9
10 000	124

12.7 CLEAN WATER DIVERSION CHANNEL

In Phase 1 a clean water diversion channel was built to the north of the current TSF. In Phase 2, the second TSF is located to the north of the current TSF and thus a new clean water diversion channel is required in to divert clean water runoff coming from the north around the TSFs. Figure 12-6 shows the location of the new clean water diversion channel and the clean catchment draining from the north. The design flow for the channel was calculated from the Rational Method as 4.65 m³/s which gave a trapezoidal channel size of bottom width of 2 m, with side slopes of 1:2.5 and a depth of 1 m (Haarhoff & Cassa, 2009). Since the site has a general fall from north to south the channel was given a longitudinal slope of 1:250 and the channel flows from the east to the west before turning south to follow the natural ground. Due to the topography the channel starts at a depth of 1 m at the inlet of the channel and reaches a depth of 2.88 m at the bend where the channel turns south and is sloped in order to daylight to the environment. Figure 12-7 shows the typical cross-section of the channel while Figure 12-8 shows the outlet of the channel that fans at 45° and is lined with reno mattress to protect against erosion. There are sensitive areas that have been classified around the TSFs and the new clean water diversion drain bypasses this area and daylights adjacent to it.

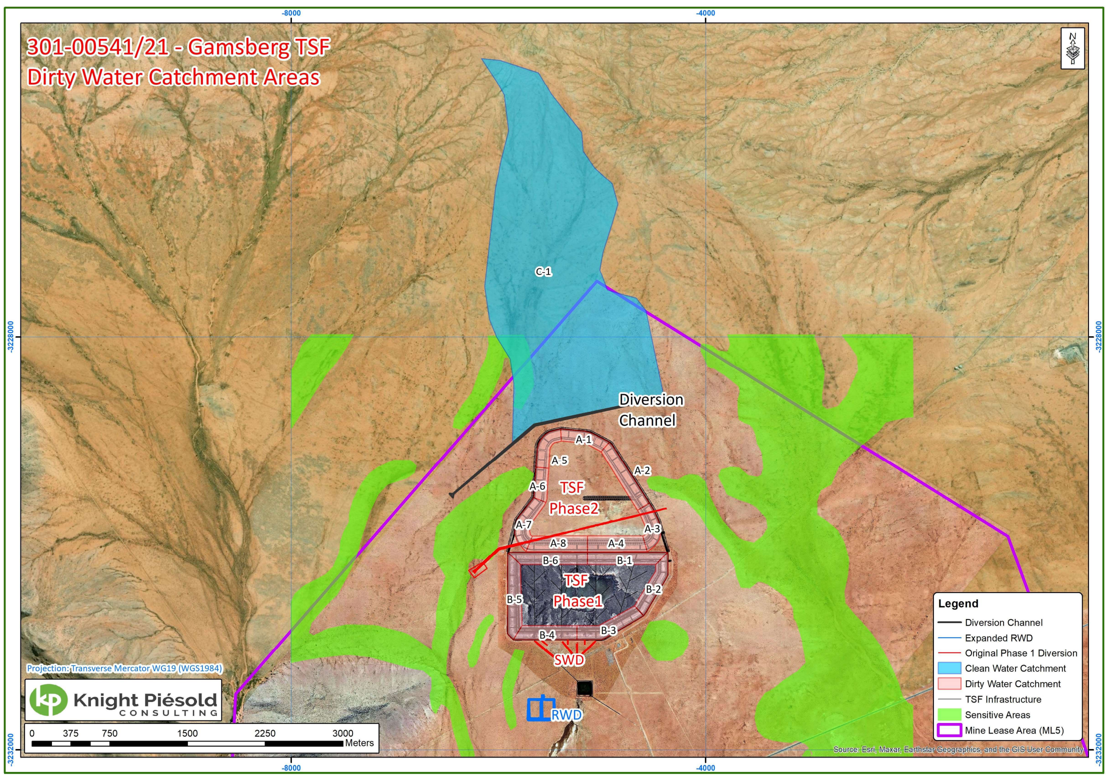


Figure 12-6 Location and catchments of new clean diversion channel

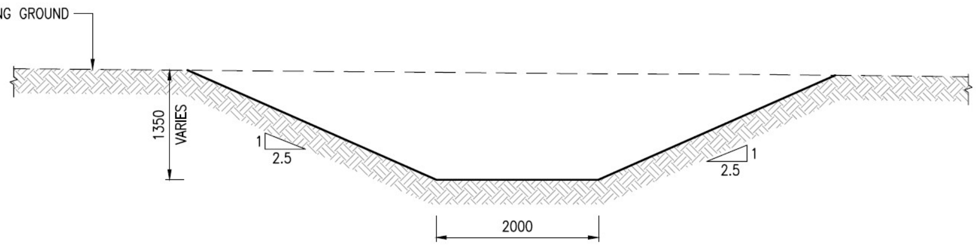


Figure 12-7 Typical cross section of clean water diversion channel

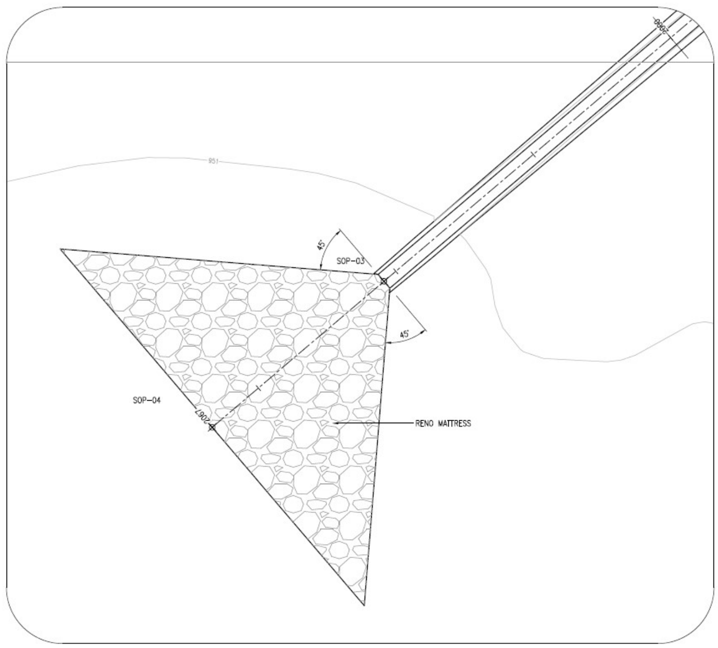


Figure 12-8 Outlet of clean water diversion channel

12.8 FLOODLINE DETERMINATION

As mentioned in Section 3.1 the Gamsberg Mine site sits in an endorheic basin where the water does not drain to an ocean. To the west of the TSFs there is a non-perennial river that flows southwards that required an updated flood line. This non perennial river is shown in Figure: 12-9.

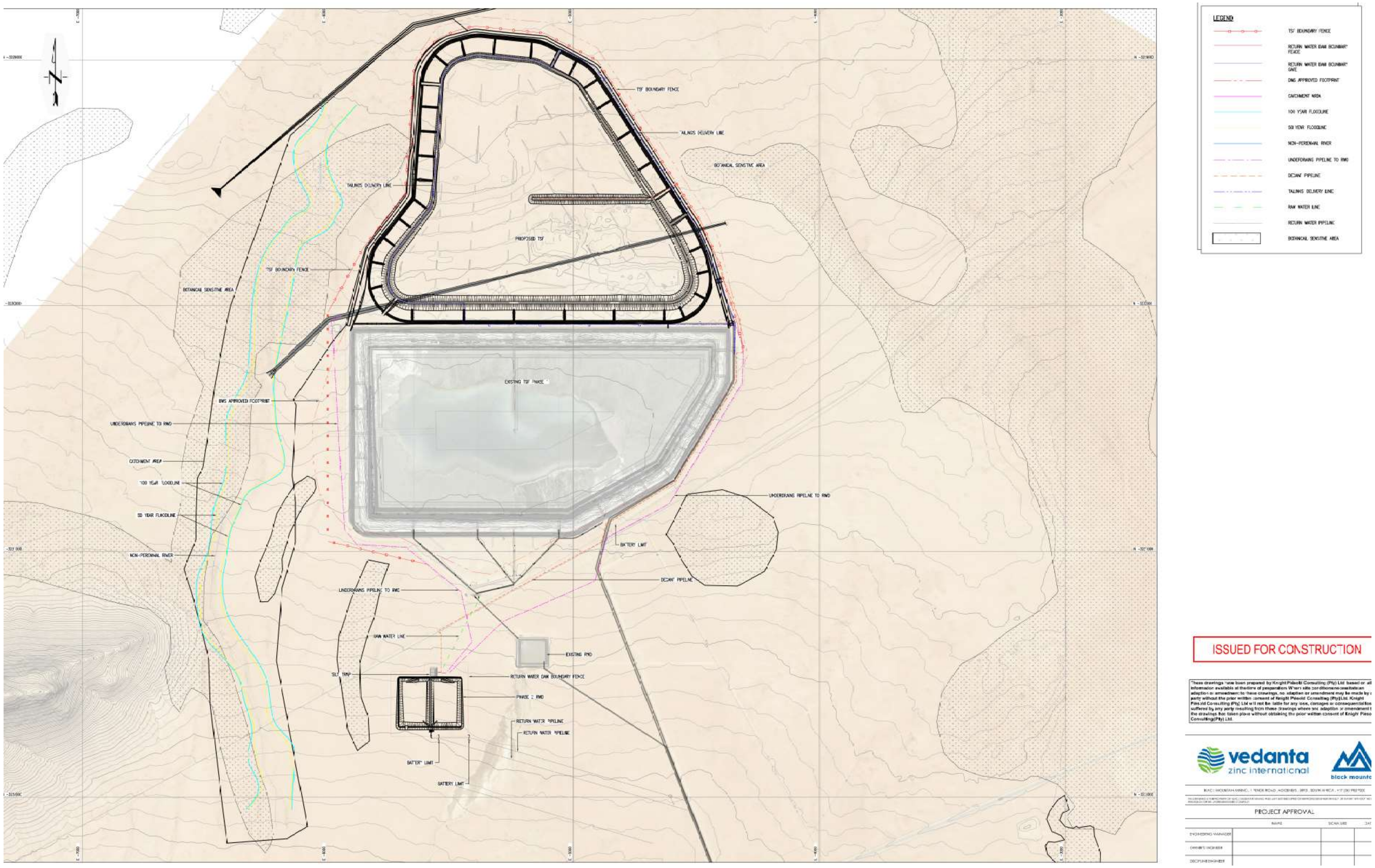


Figure: 12-9 Location and floodlines of non-perennial drainage line adjacent to the TSFs

The floodlines for the river were determined. The following method was used for the determination of the floodlines:

- The catchment area of the river was delineated based on the latest survey obtained from the client;
- A flood peak analysis was undertaken to determine the flood peaks for the different recurrence interval storms for the river using the Rational flood estimation method;
- The flood peaks and the survey data of the study area as supplied by the client were used as inputs to the HEC-RAS backwater programme to determine the surface water elevations for the 1: 50 and 1:100 year flood peaks;
- The floodlines were plotted on the available mapping; and
- Manning's n coefficients were estimated by comparing the vegetation and nature of the channel surfaces to published data (Webber, 1971).

12.8.1 SUBCATCHMENTS

The drainage area of the river was delineated as one catchment based on the topography of the area and taking into account the new clean diversion drain to the north of the Phase 2 TSF. The catchment boundary is shown in Figure: 12-9.

12.8.2 FLOOD PEAK CALCULATION

The various flood estimation methods are the Rational Method using Point Precipitation (RM-PP), the Rational Method using TR102 (RM-TR_1), the Standard Design Flood method (SDF) and the Empirical Flood Estimation method or the Regional Maximum Flood method (RMF) (Haarhoff & Cassa, 2009). Since the RM-TR102, SDF and RMF methods apply mainly to large catchments of 10 km² and greater only the rational method was used for this floodline as the catchment area is relatively small. The subcatchment characteristics used in applying this method are shown in Table 12-7 and the flood peaks for the 1 in 50 and 1 in 100 year flood are shown in Table 12-8.

Table 12-7 Subcatchment Characteristics used in the Flood Estimation

Subcatchment	Area (km²)	River Length	10-85 Slope	Time of concentration (h)
Catchment	1.25	3.15	0.009	1.754

Table 12-8 Computed 50 year and 100 year Flood Peak

Subcatchment	Peak Flow (m³/s)	
	1 in 50 year	1 in 100 year
Catchment	1.841	2.546

Figure: 12-9 shows the floodlines for the river adjacent to the TSFs. The Floodline Cross Sections and HEC-RAS Output results are given in Appendix E.

13.0 WATER BALANCE

The main purpose of the water balance is to provide a forecast simulation model which, can be used to run different probabilistic and hypothetical rainfall scenarios and in turn be used to make design, operation and management decisions around the water management of a TSF. This is to ensure that:

- Water is reclaimed back to the process plant for use as process water and optimises the recovery of water from the TSF;
- The TSF is not a safety hazard in respect of overtopping of the embankments;
- The tailings are deposited in a managed process which facilitates the efficient drying out of the tailings to ensure stability. The supernatant pond is maintained to manageable levels;
- Discharge of supernatant water directly to the environment is avoided or, at least minimised within discharge consent parameters thus minimising hydrological and environmental impacts;
- At life of mine (LoM) the TSF is left in the optimum condition for post-closure rehabilitation to proceed efficiently;
- Assess the adequacy of the associated infrastructure; and
- Check compliance of all facilities according to their respective regulatory requirements i.e. GN704 (National Water Act (Act 36 of 1998), 1999) and GISTM (Global Tailings Review, 2020) for both the TSFs and the RWDs.

13.1 FLOOD ROUTING, WATER CONTAINMENT AND FREEBOARD REQUIREMENTS

13.1.1 LEGAL REQUIREMENTS

All mining activities are required by law to adhere to various pieces of legislation. The legislation ranges from Health and Safety to Environmental aspects.

In South Africa, water management at mines is controlled by the National Water Act (Act No. 36 of 1998) (1998) and there is a regulation in place called GN704 (1999) which regulates the containment and freeboard requirements for dirty water systems (such as tailings dams and return water dams). In particular, the act makes provision for pollution prevention and control, dam safety and water use regulation.

For the purposes of developing the water balance for the Gamsberg Phase 2 TSF and related water management infrastructure, GN704 has been adopted. GN704 focuses on the control of dirty water to minimise the impact of mining activities on surrounding water sources whether it is ground or surface water. According to GN704 of the National Water Act, any mining operation is required to:

- Confine any unpolluted water to a clean water system, away from any dirty area (GN704 3.6a);
- Design, construct, maintain and operate any clean water system at the mine or activity so that it is not likely to spill into any dirty water system more than once in 50 years (GN704 3.6 b);
- Collect water arising within any dirty area, including water seeping from mining operations, outcrops or any other activity, into a dirty water system (GN704 3.6 c);
- Design, construct, maintain and operate any dirty water system at the mine or activity so that it is not likely to spill into any clean water system more than once in 50 years (GN704 3.6 d);

- Design, construct, maintain and operate any dam or tailings dam that forms part of a dirty water system to have a minimum freeboard of 800 mm above full supply level, unless otherwise specified in terms of Chapter 12 of the act (GN704 3.6 e); and
- Design, construct and maintain all water systems in such a manner as to guarantee the serviceability of such conveyance for flows up to and including those arising as a result of the maximum flood with an average period of recurrence of once in 50 years (GN704 3.6 f).

The dirty water containment and freeboard requirements referred above are highlighted in Figure 13-1.

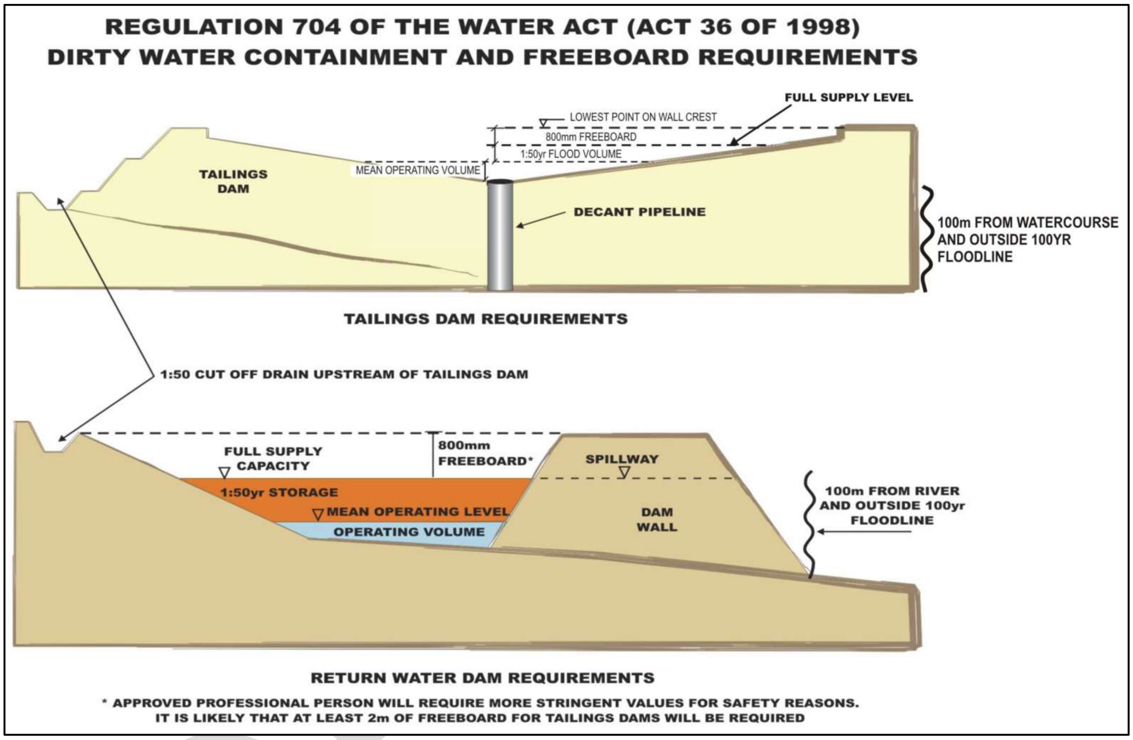


Figure 13-1 Dirty Water Containment and Freeboard Requirements as per GN704

13.1.2 GISTM GUIDELINES AND ICMM COMMITMENTS

In addition, the new Global Industry Standard on Tailings Management (GISTM) (Global Tailings Review, 2020) outlines, through 6 topics and 15 principles guidelines, how to effectively manage and operate tailings storage facilities.

All International Council on Mining and Metals (ICMM) members have committed to implement the Global Industry Standard on Tailings Management (GISTM) on their tailings dam operations. The commitment compliance date for facilities with 'Extreme' or 'Very high' potential consequences is 5 August 2023, and all other tailings facilities in operation must be in conformance with the Standard by 5 August 2025. Given how close these dates are and considering that there are as many as 77 auditable requirements, the implementation of the standard needs careful planning and prioritisation.

Although GISTM documentation provides stipulations for flood design criteria based on consequence classification, it doesn't specifically stipulate what jurisdictional regulatory framework should be followed as the basis for checking GISTM compliance and is also silent about the dry freeboard requirements. Furthermore, it states that:

> *"Alternative guidance exists, for example, by reputable national dam associations, which, in turn, form the basis of jurisdictional regulatory requirements. These alternative guidance's can be considered by the EOR, RTFE and ITRB or independent technical reviewer and adopted, if appropriate and approved by the Accountable Executive."*

Although there may be varying interpretations to the GISTM guidelines, KP's approach to checking GISTM compliance for Southern African projects, where GN704 was previously adopted, is as follows:

The TSF is categorised into one of two cases:

- **Case 1:** A TSF where storm inflow into the TSF is limited to stormflow from the TSF area (i.e., a ring dyke type facility); and
- **Case 2:** A TSF where there is storm inflow into the TSF from an external catchment.

For a Case 1 TSF: The regulatory guidance to follow, as the basis for checking GISTM compliance, remains GN704. This implies that the flood design criteria remain the 1:50 year 24-hour rainfall storm event and further implies that this storm needs to be accommodated on top of the Mean Operating Volume (or Normal Operating Water Level) with a nominal/dry freeboard of 800 mm (freeboard requirement as per GN704).

For a Case 2 TSF: The regulatory guidance to follow, as the basis for checking GISTM compliance, is to apply a GISTM Inflow Design Flood (IDF) (as determined from GISTM Flood design criteria for the specific consequence classification) on top of the Mean Operating Volume (or Normal Operating Water Level) with a nominal/dry freeboard of 800 mm (freeboard requirement as per GN704).

This approach is depicted in Figure 13-2.

Where a mine house is a member of ICMM, Case 1 TSFs are checked for both regulatory guidance requirements as listed under Case 1 and Case 2.

For ease of reporting the two compliance checks performed for the Gamsberg Phase 2 TSF, it will be referred in short as

- GN704 compliance (referring to Case 1 compliance); and
- GISTM IDF compliance (referring to Case 2 compliance).

For the Gamsberg Phase 2 TSF, the consequence classification of the facility is listed as significant and thus according to the GISTM standards the flood criteria – annual exceedance probability for operations and closure will be the 1 in 1 000-year storm event.

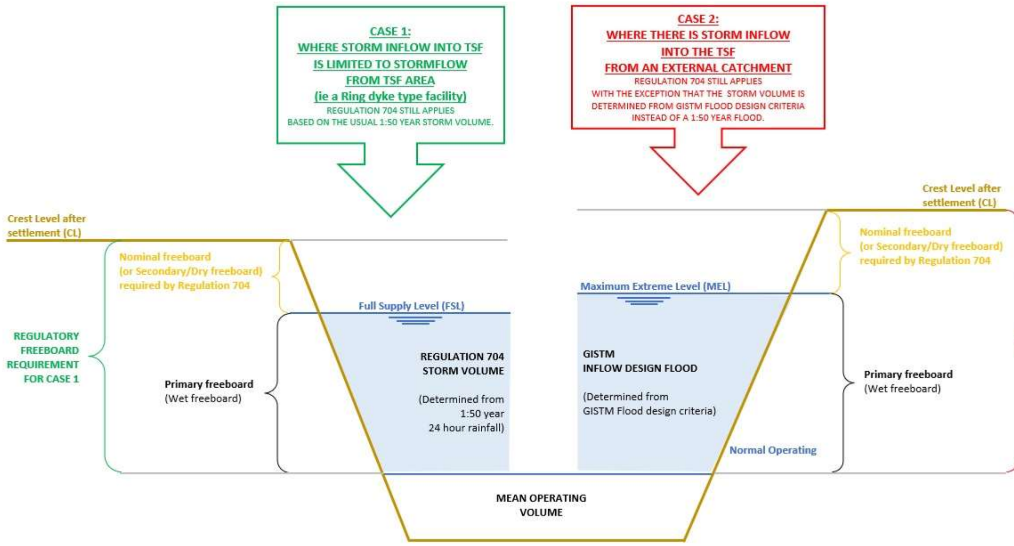


Figure 13-2 Dirty Water Containment and Freeboard Requirements as per GISTM

13.2 WATER SCHEMATIC

The water management system at Gamsberg Zinc Mine currently consists of a plant, a tailings storage facility (TSF) and a return water dam (RWD1). Phase 2 of the project will be adding an additional TSF, RWD and plant to the system. The water circuit for the TSF system consists of the following:

- **Plant 1:** The plant discharges tailings to TSF1. It receives return water from RWD1 and the plant make-up water is supplied from various sources. The water balance model will be able to simulate the dynamics of the interactions between the tailings storage facility and its associated RWD. Since the modelling of the plants are not part of this scope, they will be modelled as a “black box” where it will be assumed that all the water returned from the TSF can be accommodated at the plant;
- **Tailings Storage Facility 1 (TSF1):** The on-site management at TSF1 is by FAT. This TSF receives tailings from Plant 1 and is incorporated into the latest water balance update. The tailings decant and under drain flow is sent to RWD1;
- **Return Water Dam 1 (RWD1):** This dam receives the tailings decant and water from TSF1 and under drains from the TSF1. RWD1 return water is pumped to Plant 1. The capacity of RWD1 is 29 672 m^3;
- **Plant 2:** The plant discharges tailings to TSF2. It receives return water from RWD2 and the plant make-up water is supplied from various sources. As mentioned above, this plant will be modelled as a “black box” where it will be assumed that all the water returned from the TSF can be accommodated at the plant;
- **Tailings Storage Facility 2 (TSF2):** The on-site management at TSF2 is by FAT. This TSF receives tailings from Plant 2 and is incorporated into the latest water balance update. The tailings decant and under drain flow is sent to RWD2;
- **Return Water Dam 2 (RWD2):** This dam receives the tailings decant water and under drains from TSF2. RWD2 return water will be designed to be pumped to Plant 2. The capacity of RWD2 is 131 701 m^3;

The schematic of the Gamsberg Phase 2 TSF water balance is shown in Figure 13-3 clearly outlining the inflows, outflows, and storage variation.

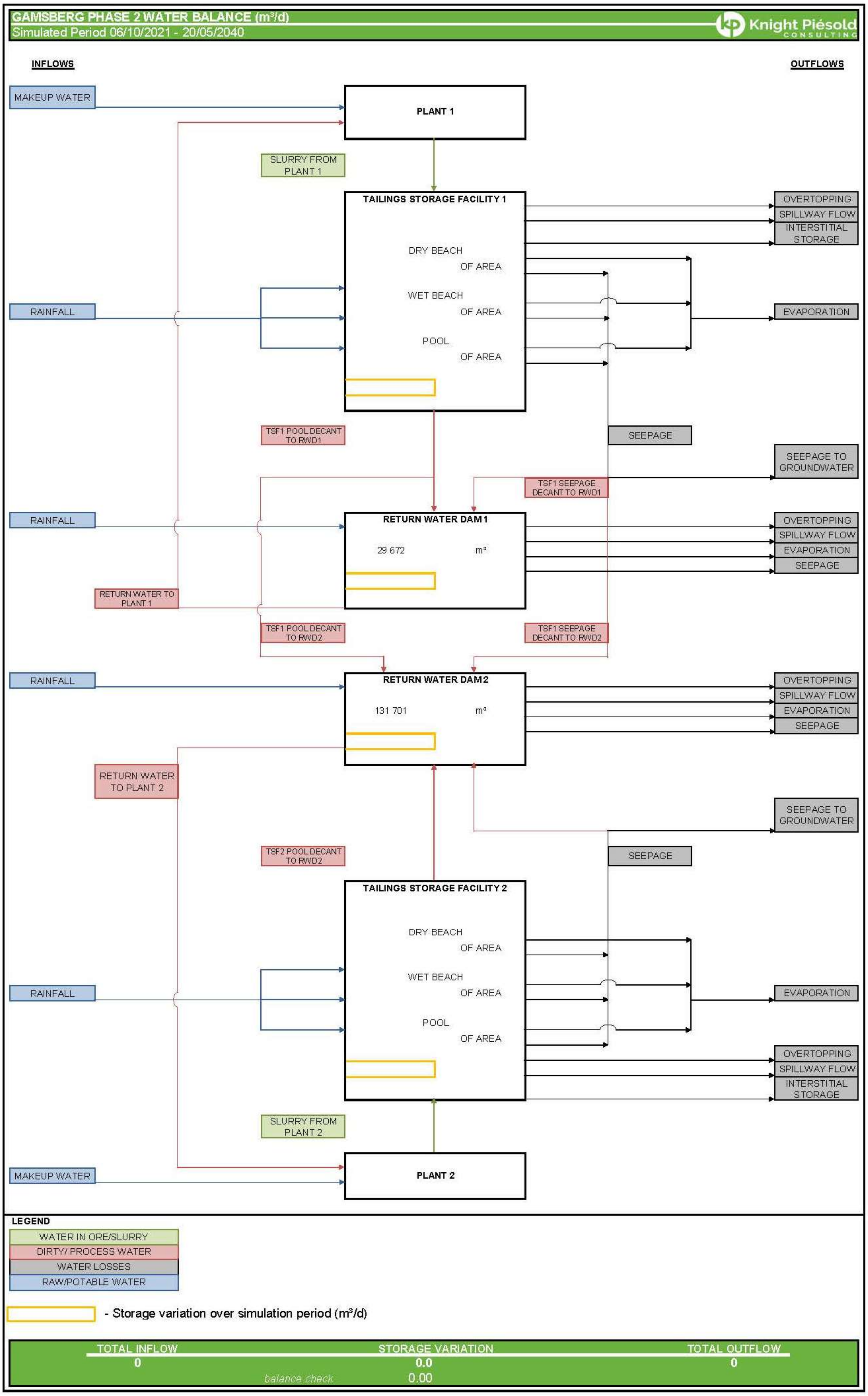


Figure 13-3 Gamsberg Phase 2 Water Balance Schematic

13.3 MODELLING THE WATER BALANCE

13.3.1 THE NEED FOR A DAILY PROBABILISTIC WATER BALANCE MODEL

Mine water management is a fundamental issue that affects most plant sites worldwide. The risk of surface water and groundwater contamination caused by mining and processing activities and any subsequent environmental consequences results, in the need for careful planning and operation of plant water infrastructure. The need to provide an adequate approximation of process water supply, as well as the assurance of compliance of discharge water quantity and quality with local environmental legislation and best practises is becoming equally important. Typically, mine water balances are focused around the tailings area, plant processes and pit area. The classic water balance approach usually involves building a deterministic model in which respective elements of the water cycle are usually represented by using averaged values of system variables over modelled time steps. Many of these variables are well known in advance and can be fairly precisely defined. However, hydrological processes and plant processes are often probabilistic by nature and using their average values (such as a fixed runoff percentage as a runoff estimate) limits the accuracy and usefulness of the model outputs. Furthermore, a deterministic model does not usually account for extreme meteorological events, which may have direct implications for water supply and disposal, and quite often determine important design requirements during the design and approval processes.

A continuous probabilistic model can provide a picture of the various possible outcomes of different scenarios, and assess the impact of risk, allowing for better decision making under uncertainty. This is of particular interest for hydrological events where future rainfall sequences cannot be predicted. The daily probabilistic model allows the user to simulate different rainfall sequences, and to explore how these can affect the water balance (see Figure 13-4). The magnitude of extreme meteorological events is furthermore as important as the timing at which it occurs. While a proper monitoring system provides a good knowledge of what has happened in the past, it cannot predict what will happen in the future. The model can test and simulate what could happen in the future, as well as the associated risk.

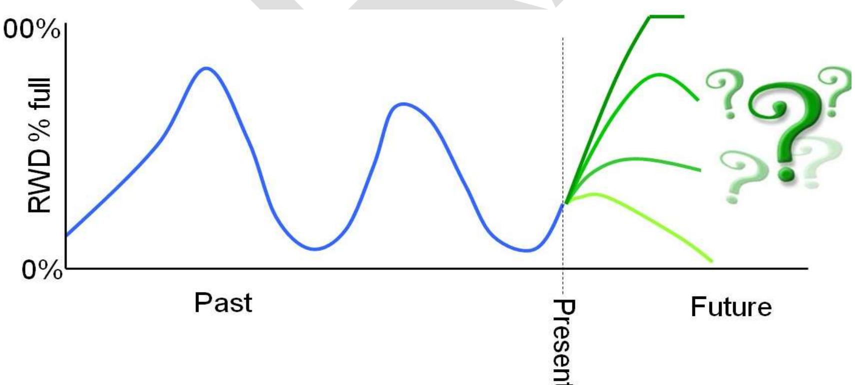


Figure 13-4 Illustration of simulation of future events

Building a daily probabilistic water balance model, with appropriate assumptions, allows the user to build in the necessary subtleties required to address and understand complex systems.

A daily probabilistic model offers the user the following benefits:

- A better understanding of the dynamics of the various circuits;

- While flows and water levels can be measured quantitatively, the estimation of dynamic processes such as runoff and seepage can provide a complete vision of the overall water balance;
- The use of the results of the continuous daily modelling to estimate average daily flows between all the components that make up the site water circuits;
- Incorporation of dynamic operating rules that react to various situations as they occur. The effects of the operating rules can then be assessed, and the operating rules optimised; and
- An assessment and better understanding of the probability distributions of variables. These include dam levels, spillage frequencies and spillage volumes. This provides a better understanding of the risks associated with unpredictable events such as rainfall.

13.3.2 GOLDSIM MODELLING ENVIRONMENT

The water balance model described in this report was built using GoldSim (GoldSim Technology Group LLC, 2022), a dynamic system modelling package, which is a graphical object-oriented modelling environment with an in-built capacity to carry out dynamic probabilistic simulations.

GoldSim uses equations and rules to simulate a system quantitatively in order to identify and understand the factors that control a system or to predict the future behaviour of the system. GoldSim was developed to be flexible so that it may be applied to a variety of systems. It can therefore be applied to ecosystems, environmental systems, engineered systems and strategic planning systems amongst others.

The model was coded using a top-down approach, i.e., beginning with a broad overview of the water balance, and then exploring the next level of detail if available and if required. The approach is particularly appropriate for tailings dams and plant water balance applications, as there are often uncertainties within these systems. Where additional detail is required and data is available, the modelling can be more detailed.

The tailings dams water balance is dynamic and depends on many factors including rainfall; the mine plan and mine water requirements. The operating rules and connectivity govern how the water streams produced from the different elements are linked together. The connectivity and operating rules are programmed into the model and have to be changed within GoldSim.

The time step of the model is dependent on the objective of the model. An annual timestep can be used to give an indication of the average water volumes that may need to be managed and an overall indication of the plant water balance. Such a long timestep does not however address the seasonal or daily variations and therefore cannot be used to size storage facilities. A monthly time step accounts for the seasonal variations and can be used to provide indicative sizing of storage facilities such as pollution control dams. The monthly timestep can also be used to provide an indication of the capacities of the pumping pipeline infrastructure needed to convey the water between storage elements. A daily timestep model allows for a more accurate determination of the return water dam sizes and pump/pipeline capacities. A daily timestep was used for the Gamsberg Phase 2 model.

13.4 OPERATING PHILOSOPHY

From Phase 1 it was found that the capacity of Return Water Dam 1 may be undersized. This prompted an investigation to mitigate any spillage from Return Water Dam 1 in Phase 2 of the project. This was done by developing an overall water management strategy. This strategy includes the dynamic integration of Return Water Dam 1 and Return Water Dam 2. The operating philosophy of Phase 2 water balance is as follows:

- When Return Water Dam 1 (RWD1) reaches a capacity that is greater or equal to the capacity at the spillway (20 370 m³), all the inflows from TSF1 are rerouted to Return Water Dam 2 (RWD2) excluding the rainfall and runoff that continue to flow into RWD1;
- RWD1 will at this point only take the rainfall and runoff of its own area and all outflows from TSF1 will be flowing to RWD2;
- At the moment RWD1 falls to 75% of its spillway capacity all the inflows will then be redirected back to RWD1;
- RWD2 will limit the inflows that it receives from TSF1 by not taking any inflows after it reaches a capacity of 80 000 m^3;
- At this capacity RWD2 will only be allowed to receive inflows from TSF2;
- When the inflows are both limited, capacity for backing up the pool on TSF1 is allowed until either of the Return Water Dams are capable of receiving an inflow; and
- The reasoning behind limiting these inflows is due to the sluice gates being operated by a spindle and requires manual operation.

The capacity of Return Water Dam 2 is assessed to not only accommodate the decant water from TSF2 but also provide the flexibility to receive excess water from Return Water Dam 1. This integration optimises water resource utilisation by redistributing surplus water to Return Water Dam 2 without posing any risk to its structural stability.

13.5 RESULTS

From Figure 13-5 the water balance model shows that over the simulation period of 6 801 days and calculating for 200 realizations the mean annual average water balance does spill at the Return Water Dam 2 facility. To assess the spill frequency of each facility, the Reservoir Elements within the model for both the TSF and the RWD were setup to report spill statistics. This reported the number of years that the element spilled which was calculated by checking if there was any overflow from the reservoir element in every year that is simulated. The number of years spilled was then used to calculate the spill frequency.

Table 13-1 shows the summary of the spillage for the entire rainfall record run with the stochastic rainfall generator for the Gamsberg Phase 2 model.

Table 13-1: Summary of spillage frequency for the stochastic water balance

Name	No of years spilled	Simulation duration (days)	No. of realisations	Simulation duration (years)	Total Spillage over period (years)	Spillage frequency	Return period (years)
RWD 2	57	6 801	200	3 724.02	57	0.0153	65.33

From Table 13-1, it shows RWD2 is well within GN704 (National Water Act, 1999) spill criteria of 1:50 year recurrence interval.

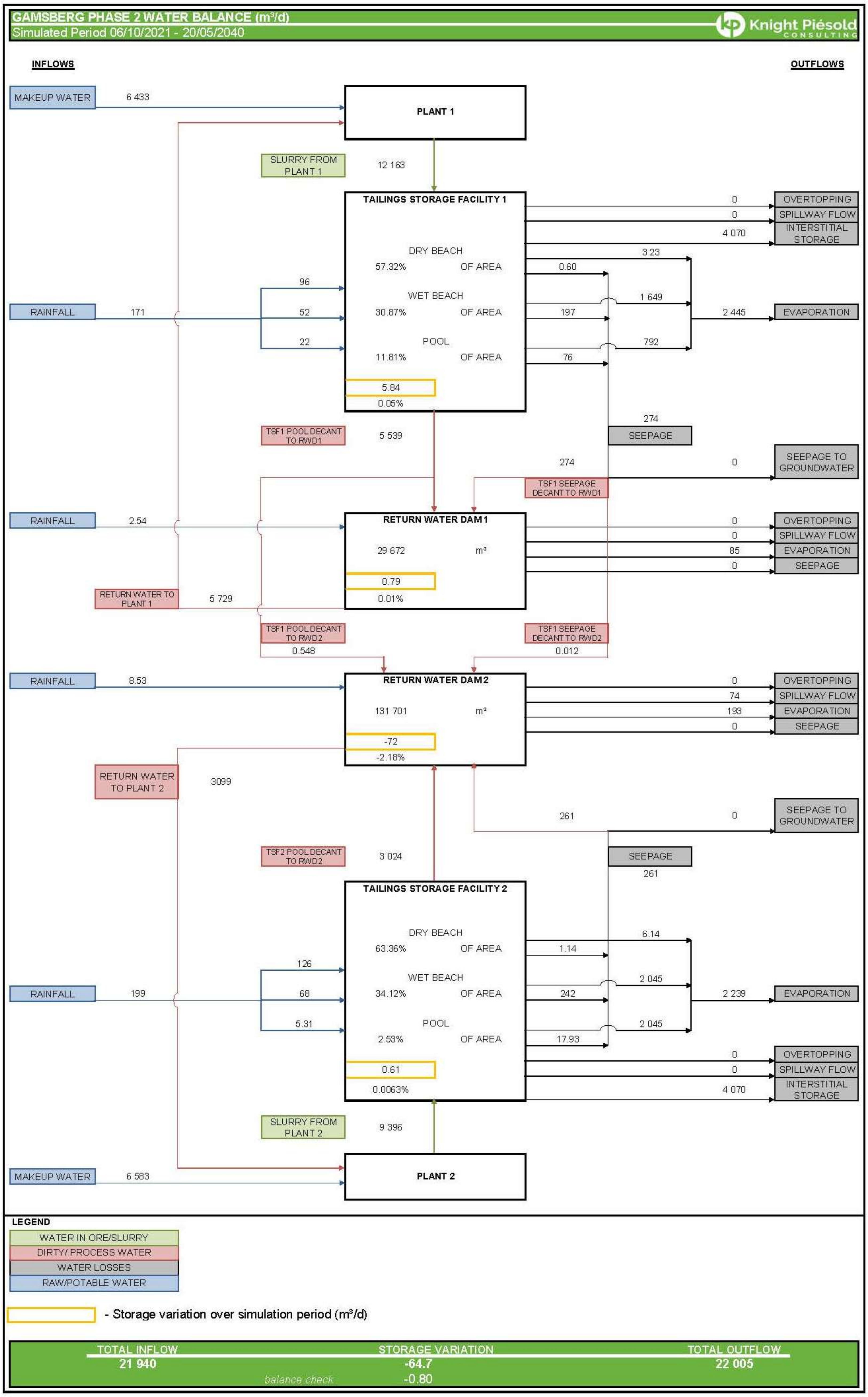


Figure 13-5 Gamsberg Phase 2 Water Balance

Figure 13-6 and Figure 13-7 show the probabilistic plot for the TSF1 and TSF2 pool operating volume respectively. This shows the fluctuations of the pool operating volume over the simulation duration.

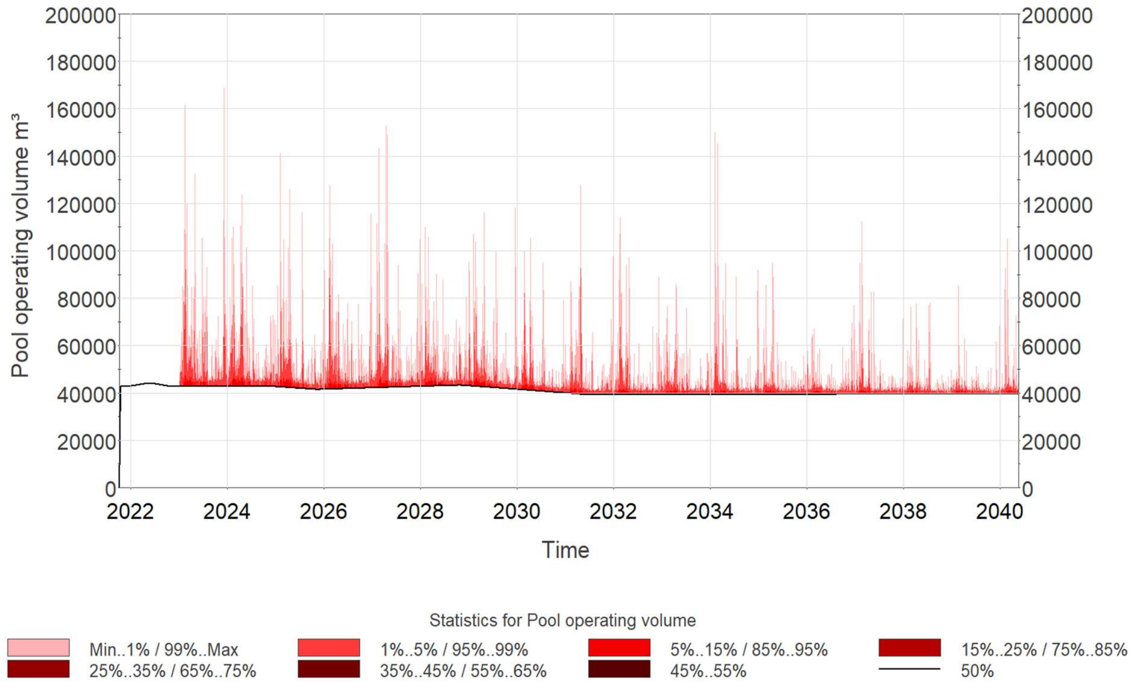


Figure 13-6 TSF1 pool operating volume

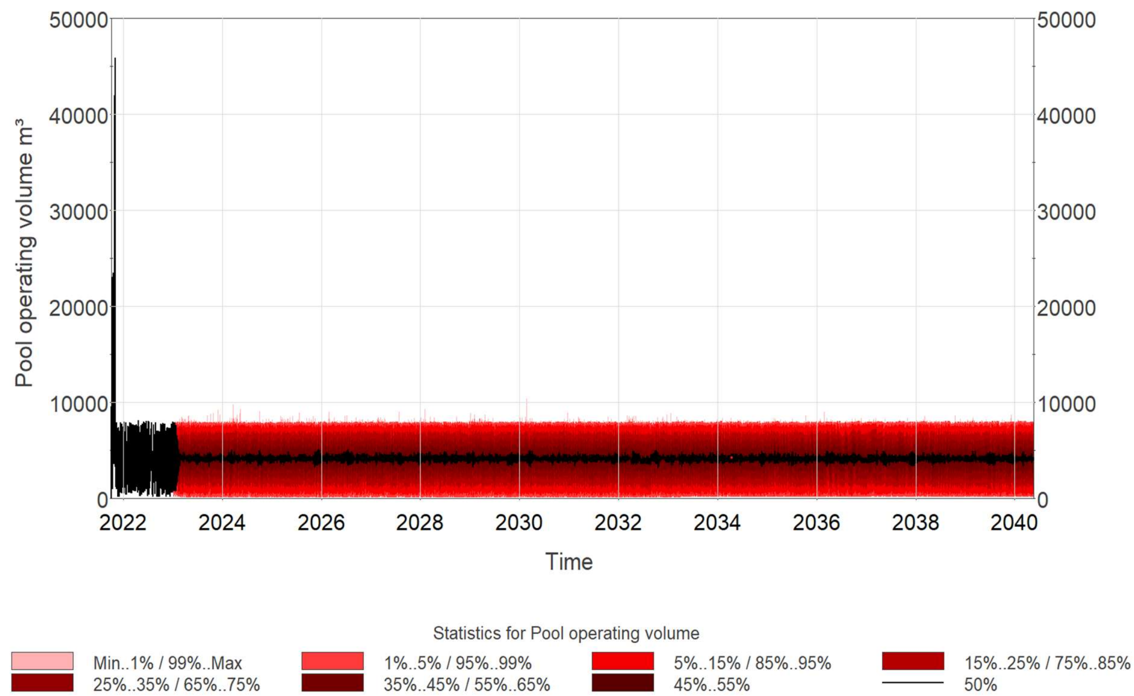


Figure 13-7 TSF2 pool operating volume

Figure 13-8 and Figure 13-9 show the probabilistic plot for the RWD1 and RWD2 pool operating volume respectively. This shows the fluctuations of the pool operating volume over the simulation duration.

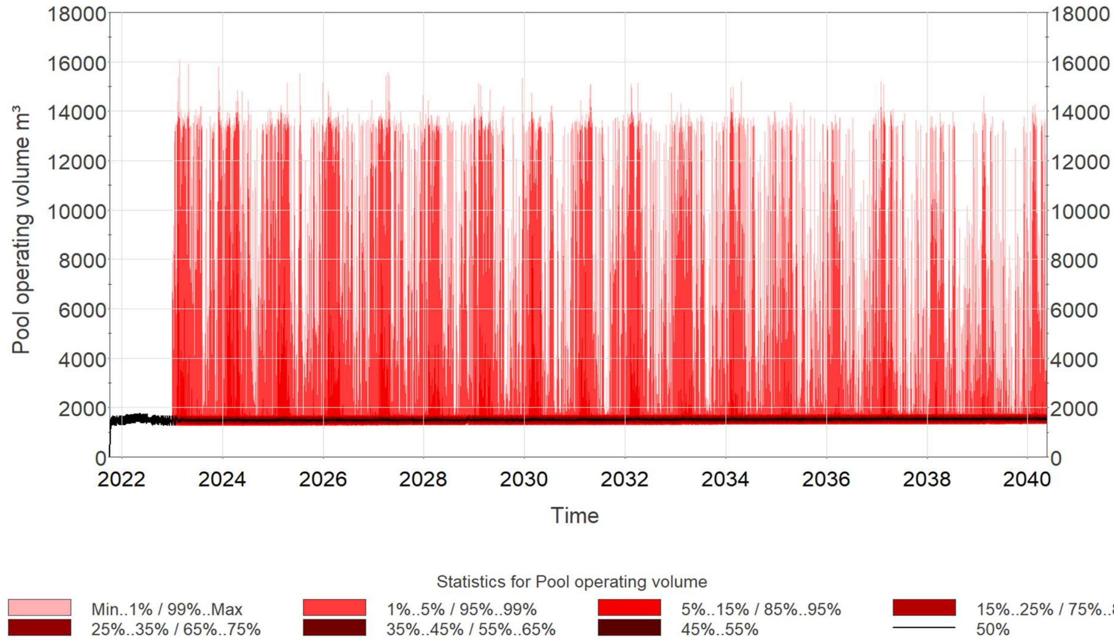


Figure 13-8 RWD1 pool operating volume

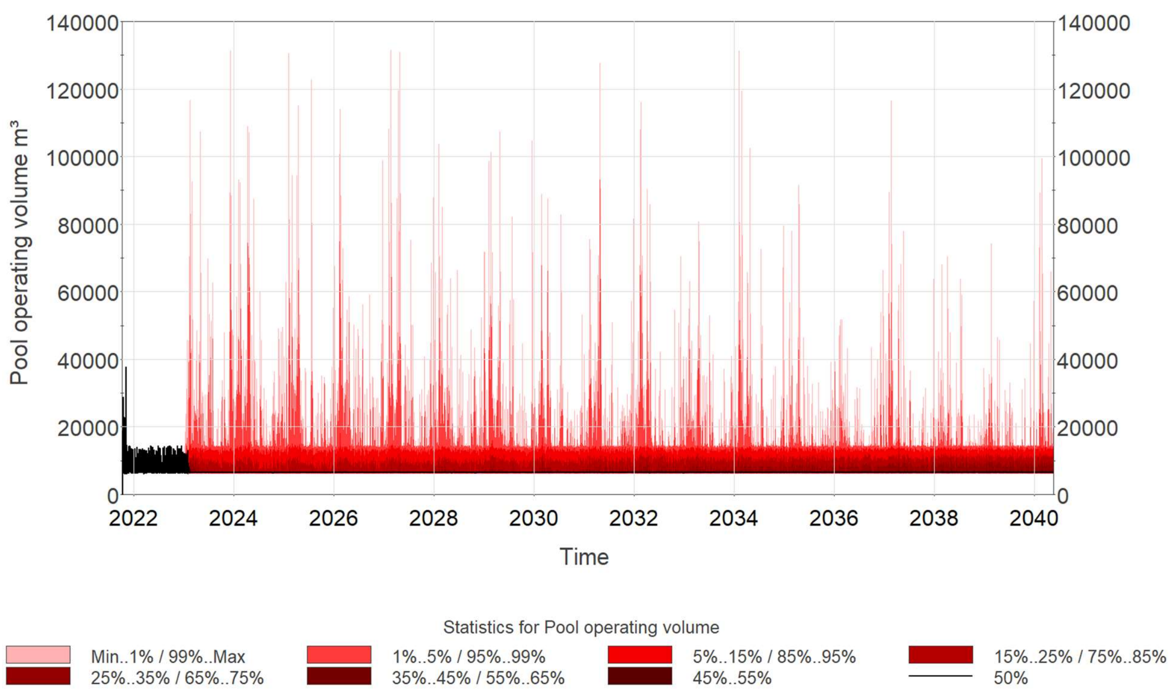


Figure 13-9 RWD2 pool operating volumes

13.5.1 GN704 COMPLIANCE

Figure 13-10 and Figure 13-11 shows the state of TSF1 and TSF2 storage volumes over the simulation period of 18.6 years. In this case, both TSF1 and TSF2 complies with the GN704 standards.

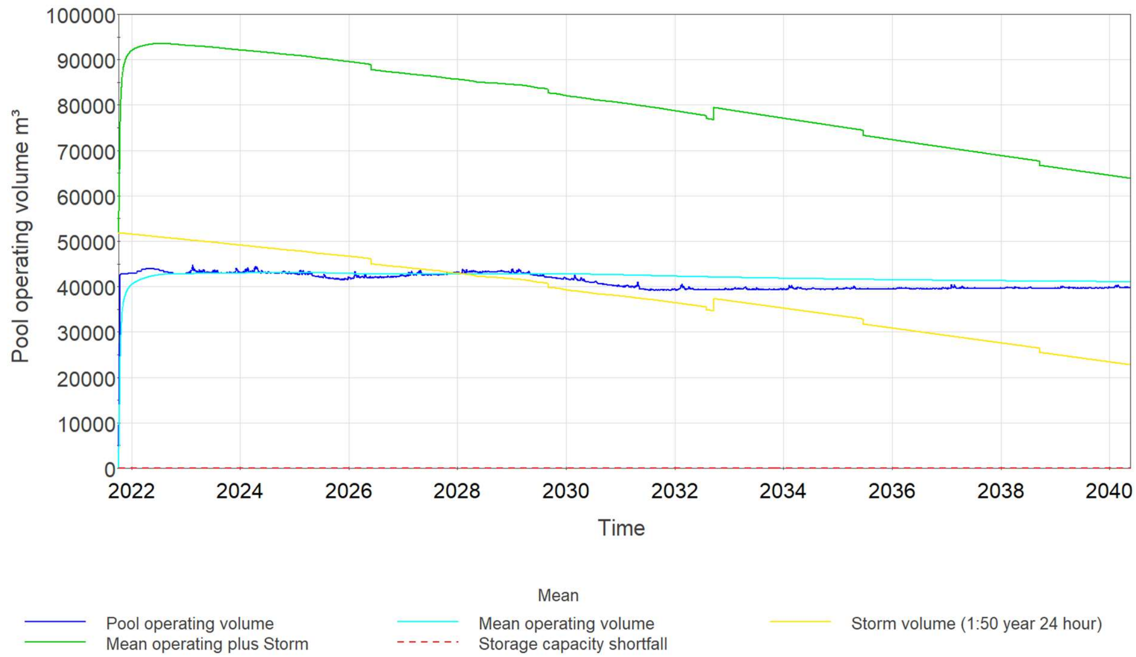


Figure 13-10 TSF1 Pool Storage GN704

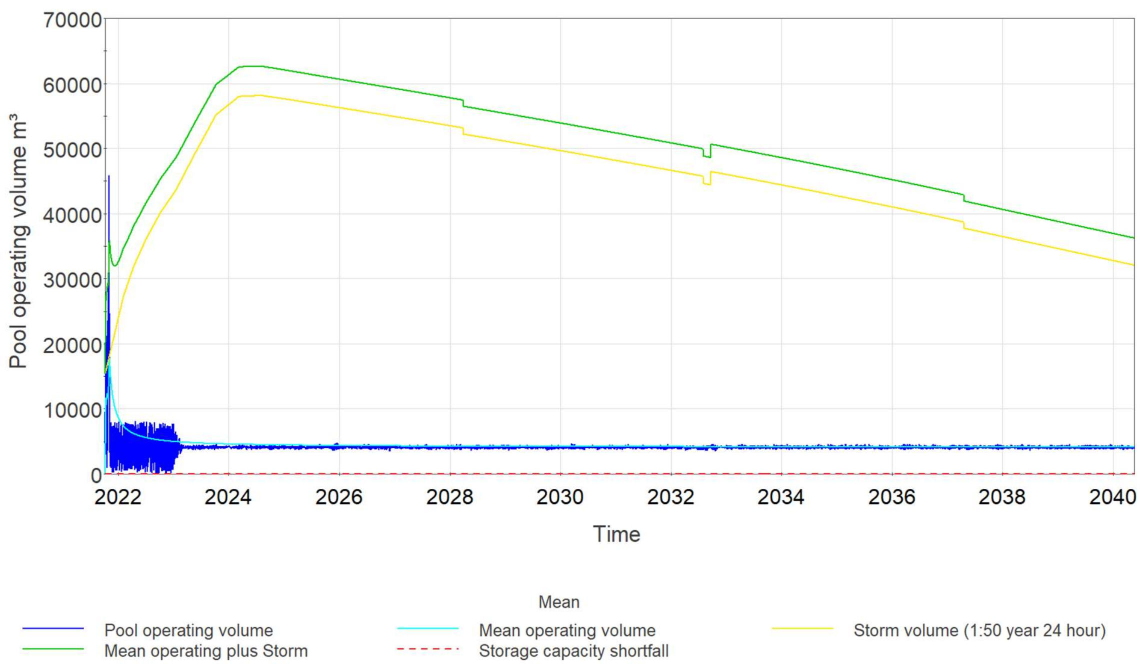


Figure 13-11 TSF2 Pool Storage GN704

Figure 13-12 and Figure 13-13 shows the state of RWD1 and RWD2 storage volumes over the simulation period of 18.6 years. In this case, both RWD1 and RWD2 complies with the GN704 standards.

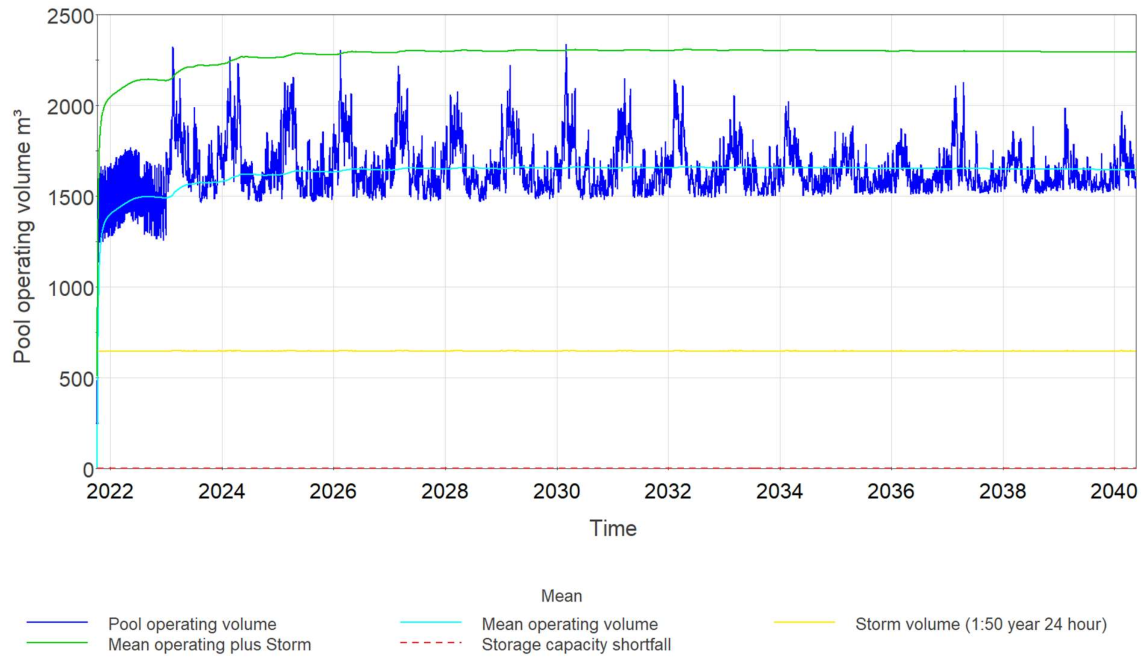


Figure 13-12 RWD1 Pool Storage GN704

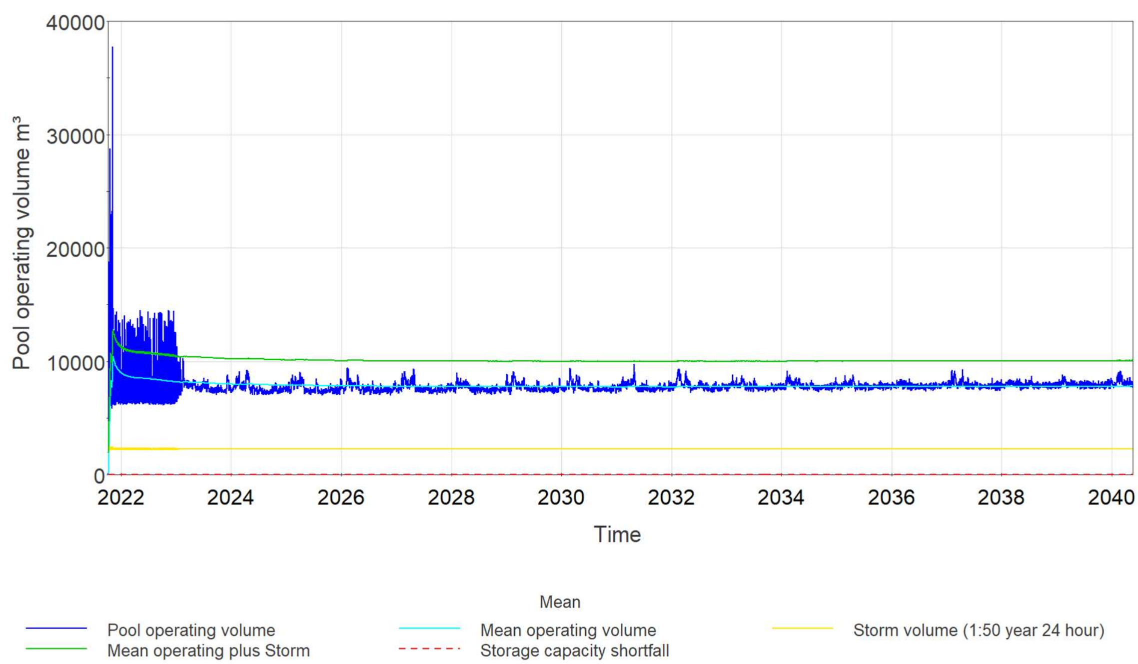


Figure 13-13 RWD2 Pool Storage GN704

13.5.2 GISTM COMPLIANCE

Figure 13-14 and Figure 13-15 shows the state of TSF1 and TSF2 storage volumes over the simulation period of 18.6 years. In this case, both TSF1 and TSF2 complies with the GISTM standards.

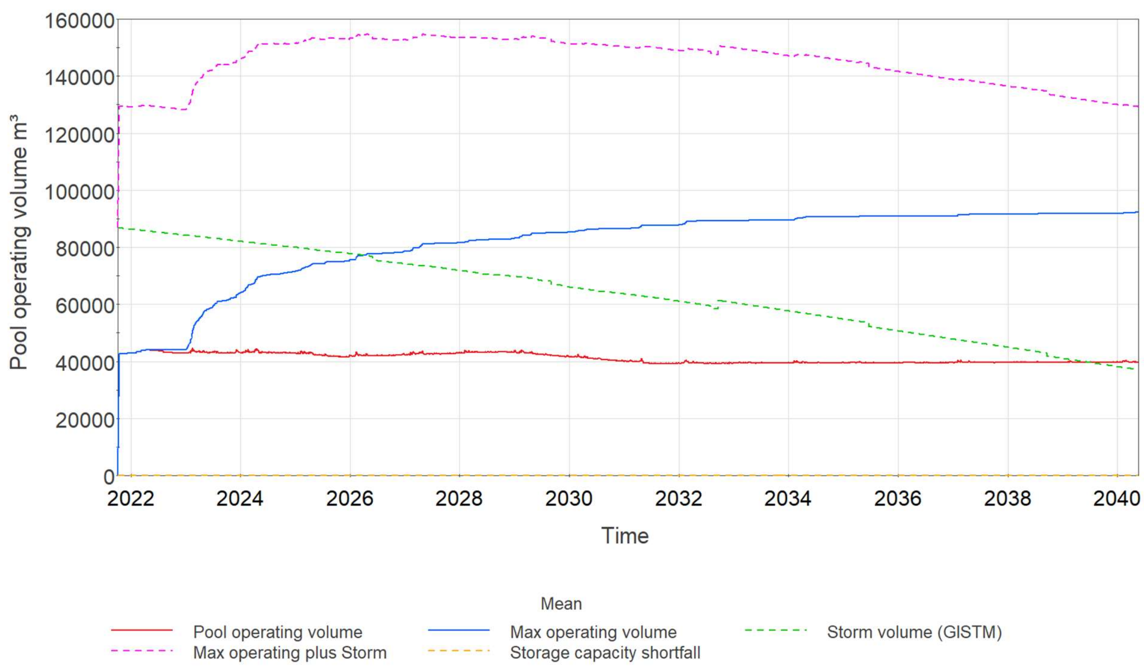


Figure 13-14 TSF1 Pool Storage GISTM

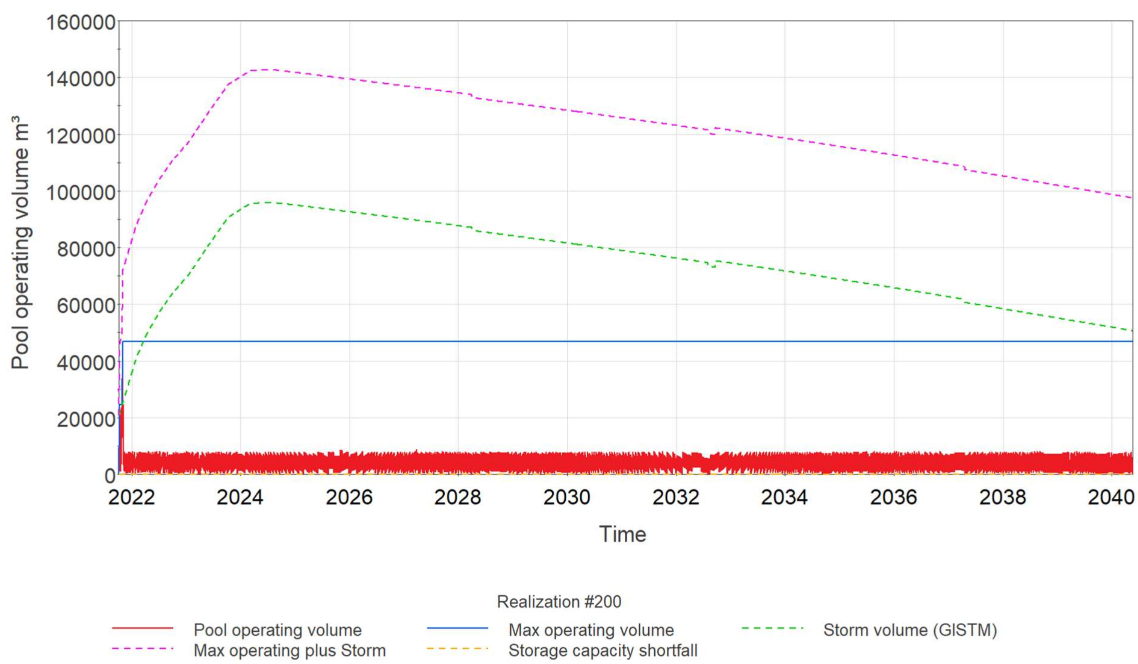


Figure 13-15 TSF2 Pool Storage GISTM

Figure 13-16 and Figure 13-17 shows the state of RWD1 and RWD2 storage volumes over the simulation period of 18.6 years. In this case, both RWD1 and RWD2 complies with the GISTM standards.

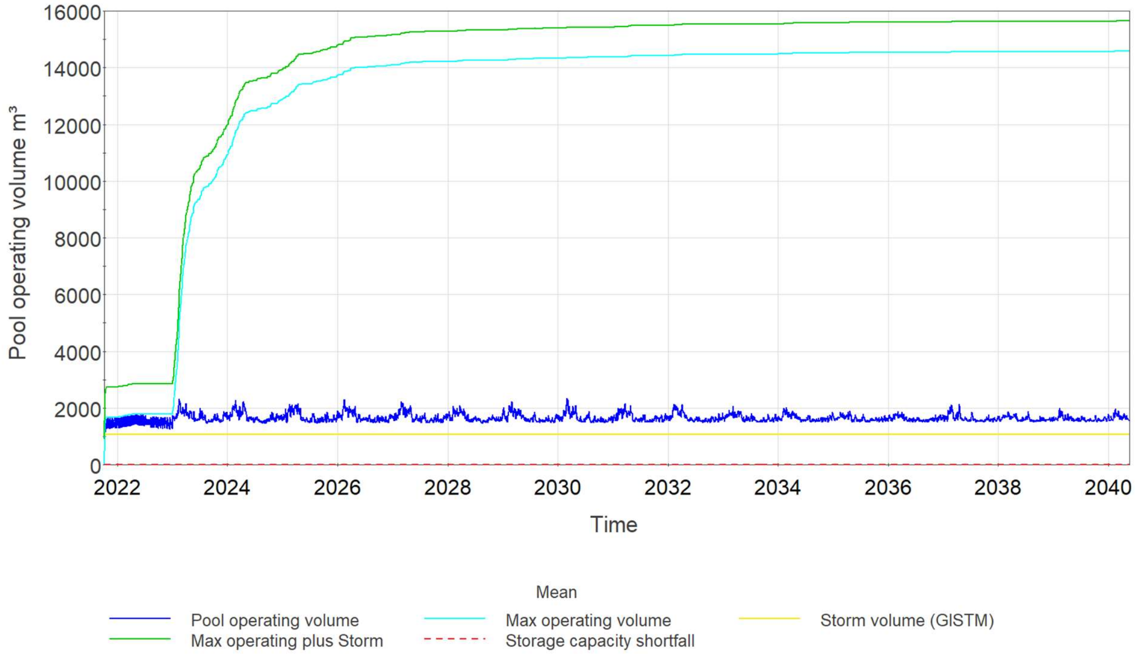


Figure 13-16 RWD1 Pool Storage GISTM

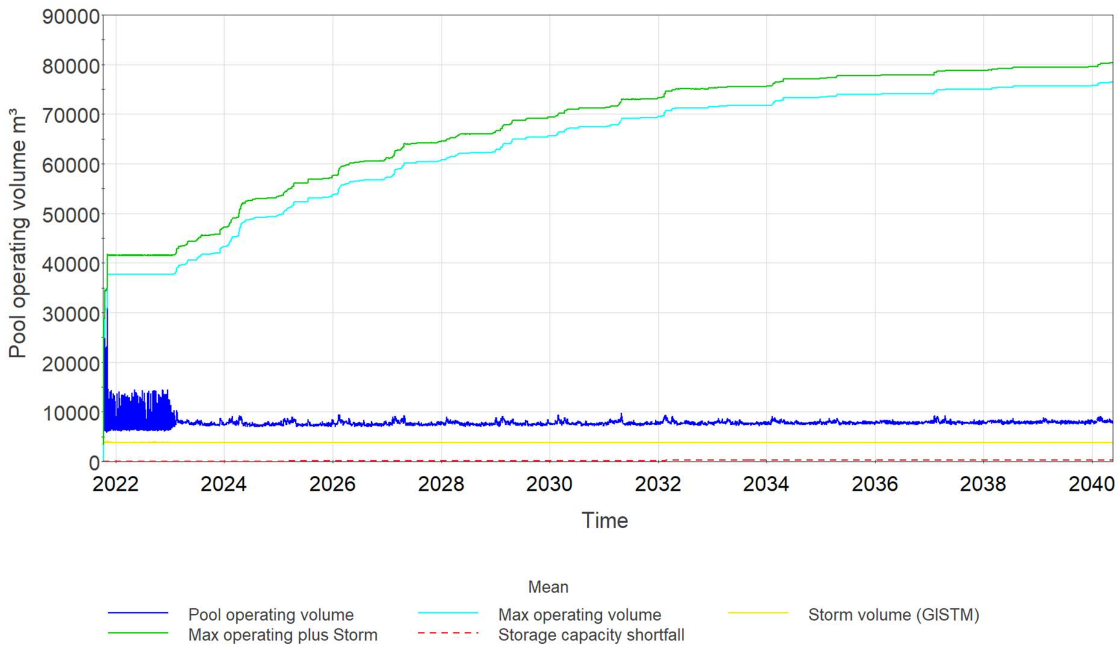


Figure 13-17 RWD2 Pool Storage GISTM

14.0 TAILINGS DAM BREAK ANALYSIS

The tailings dam breach analysis study for the Gamsberg Phase 2 Tailings Storage Facility (TSF), was completed based on a configuration corresponding to a final design crest elevation of 1 001 m.

Key characteristics of the TSF and the downstream watercourse, dam breach and downstream flood wave analyses, and flood inundation mapping that was performed as part of this updated TDBA study are described. The information presented in this document can be used by Gamsberg Zinc to support the development of an emergency preparedness plan (EPP) and for dam classification.

Six failure scenarios were considered at the Gamsberg Phase 2 TSF as part of this Tailings Dam Breach Analysis (TDBA) study, comprising an evaluation of both sunny- and rainy-day failure modes at three respective breach positions. The breach positions were selected at locations where potential impacts will be maximised, to determine the maximum inundated extent downstream. The study is based on a TSF configuration corresponding to the final designed crest elevation of 1 001 mamsl.

14.1 FREEBOARD

A TSF freeboard analysis indicated that the 1 in 10 000-year storm event can be contained without overtopping the TSF, assuming that the siphon decant is operational. The vertical freeboard under this case is projected to be 1.6 m (0.6m beach freeboard, 111 m dry beach length), implying that the dam would not overtop.

14.2 FAILURE SCENARIO DEVELOPMENT

Slope failure in the upper portion of the wall leading to loss of freeboard, followed by an erosional failure due to the release of pond water and eroded / liquefied tailings was selected as the failure mode for all rainy-day failure scenarios. Similarly, large-scale, deep seated slope failure leading to loss of confinement and the subsequent release of liquefiable tailings was selected as the failure mode for all sunny-day failure scenarios. The TSF is assumed to be susceptible to static liquefaction following loss of confinement. Outflow of liquefied tailings during a breach event was therefore included in the analysis.

The rainy-day failure scenarios consider two interrelated discharge mechanisms that will occur:

- Initial Flood Wave: The initial discharge of supernatant pond carries tailings and (eroded or liquefied) dam fill material; and
- Mudflow / Flow Slide: Flowable tailings is discharged due to liquified tailings outflow (failure) of unsupported tailings.

The sunny-day failure scenarios only consider one discharge mechanism, viz. the release of flowable liquefied tailings.

14.3 BREACH POSITIONS

Three breach positions were identified. The recommended breach positions are indicated in Figure 14-1. The breach positions were selected at locations where potential impacts will be maximised.

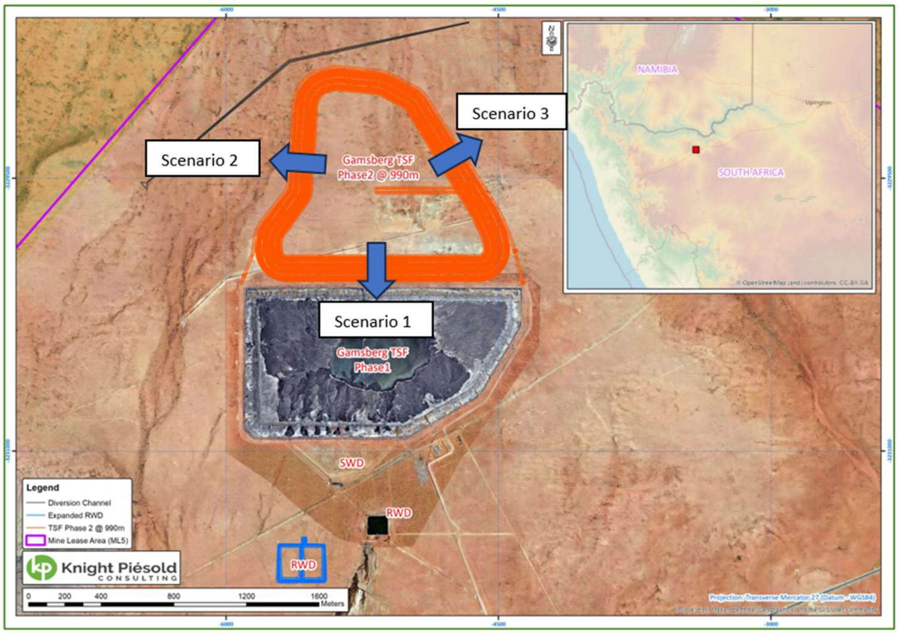
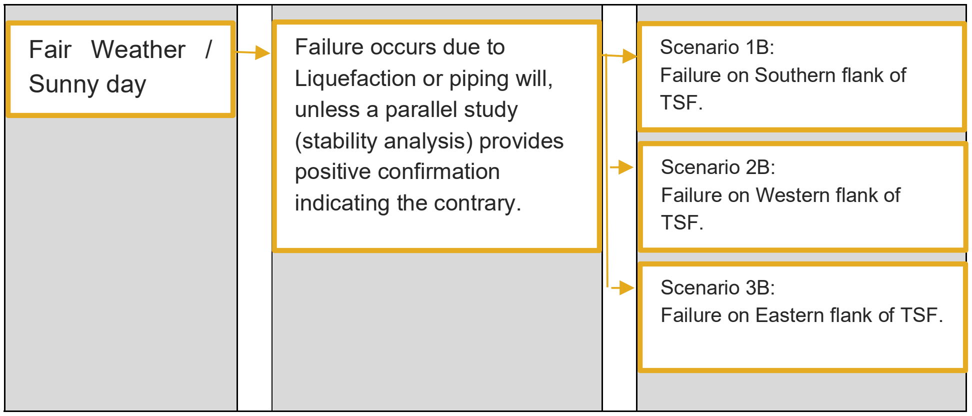


Figure 14-1: Schematic indicating recommended breach positions.

Only the most critical scenarios were evaluated (largest potential impact). The breach scenarios evaluated in this study is summarised in Table 14-1.

Table 14-1: Analysed breach scenarios for analysis.

Hydrologic Condition	Failure Mechanism	Scenario
Flood Induced / Rainy day	Failure occurs due to overtopping of the wall or piping due to proximity of the pool edge to the wall resulting in a raised phreatic surface and hydraulic gradient through the wall).	Scenario 1A: Failure on Southern flank of TSF.
		Scenario 2A: Failure on Western flank of TSF.
		Scenario 3A: Failure on Eastern flank of TSF.



14.4 BREACH OUTFLOW HYDROGRAPHS

The breach outflow hydrographs developed in HEC-HMS for the rainy- and sunny-day failure scenarios are presented in Figure 14-2 and Figure 14-3. As can be seen, the breach hydrographs that develop are characterised by two components / flood peaks:

- Initial Flood Wave: The initial discharge of supernatant pond carries tailings and (eroded or liquefied) dam fill material; and
- Mudflow / Flow Slide: Discharge of flowable tailings due to liquefaction

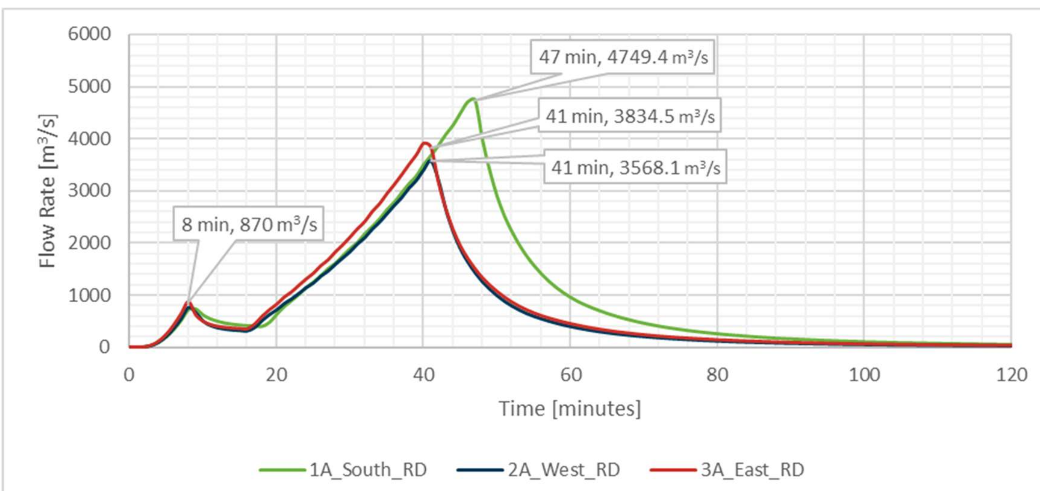


Figure 14-2: Breach outflow hydrographs for Rainy-Day failure scenarios

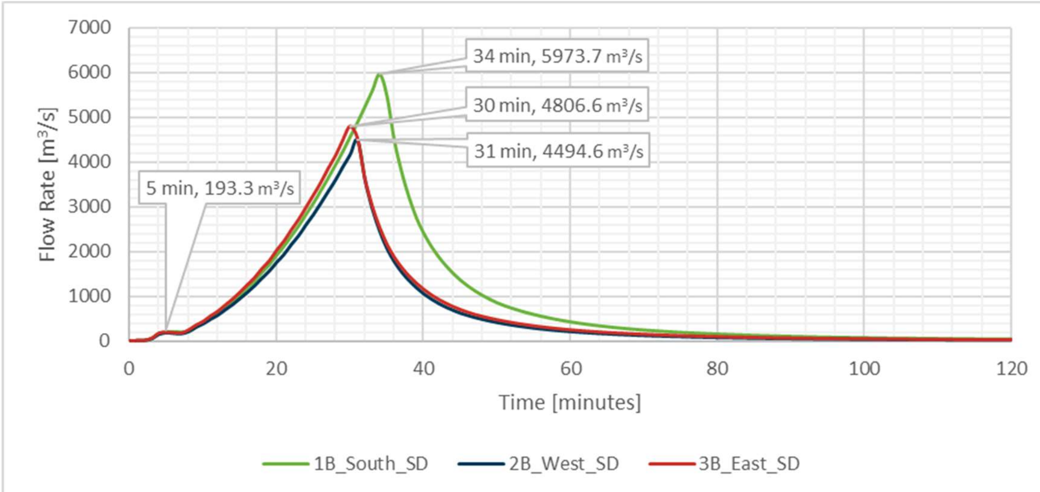


Figure 14-3: Breach outflow hydrographs for Sunny-Day failure scenarios

14.5 DOWNSTREAM FLOOD WAVE ANALYSIS

A set of inundation maps for each scenario will be included as an Appendix to the TDBA report. Each set of maps will comprise individual maps representing the inundation area, maximum depth of flow, maximum flow velocity and flood hazard rating. These maps can be used to aid the development / updating of the Emergency Preparedness and Response Plan (EPRP) of the TSF.

The results of the runout analyses are summarised per scenario below. Figure 14-4 and Figure 14-5 present a consolidated view of the inundation boundaries associated with the rainy-day and sunny-day failure scenarios for the respective TSF configurations.

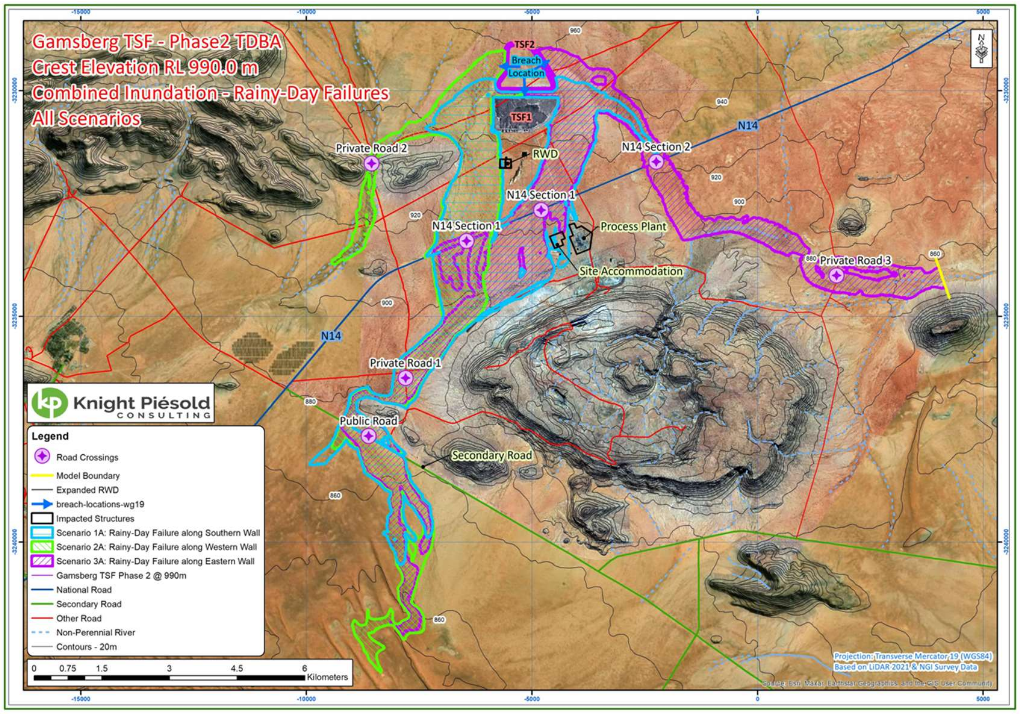


Figure 14-4: Combined Inundation Map for Rainy Day Scenario

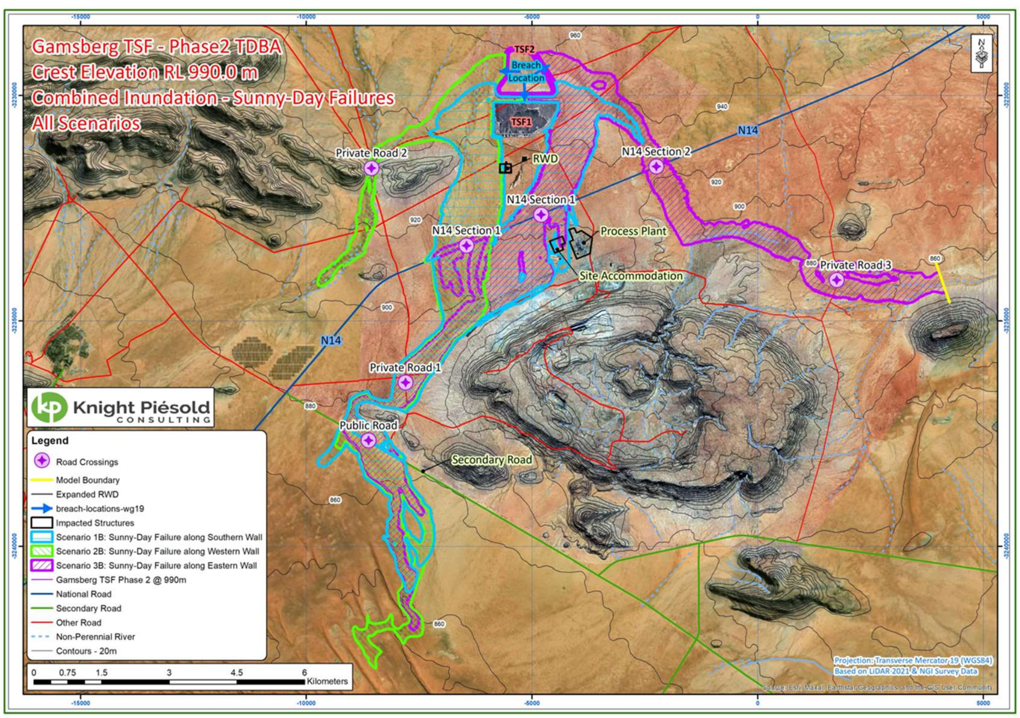


Figure 14-5: Combined Inundation Map for Sunny Day Scenario

No permanent Potential Population at Risk (PAR) were identified for any of the failure scenarios. However, transportation routes downstream including the N14 highway, the gravel road networks east and west of the Gamsberg plant and the TSF facility and the public gravel road south-east of the Aggeneys solar power plant will be inundated following a breach event at any of the evaluated positions. Loss of life following a breach event is therefore not expected but cannot be excluded.

14.6 FACILITY CLASSIFICATION

Table 14-2 summarises the results per scenario, at the respective TSF configurations, in terms of the GISTM classification matrix.

Table 14-2: GISTM (2020) Consequence Classification Matrix

Failure Scenario	Dam Failure Consequence Classification				
	Potential Population at Risk	Potential Loss of life	Environment	Health, Social and Cultural	Infrastructure and Economics
1A: South – Rainy Day	Significant	Low	Significant	Low	Low
1B: South – Sunny Day	Significant	Low	Significant	Low	Low
2A: West – Rainy Day	Significant	Low	Significant	Low	Low
2B: West – Sunny Day	Significant	Low	Significant	Low	Low
3A: East – Rainy Day	Significant	Low	Significant	Low	Low
3B: East – Sunny Day	Significant	Low	Significant	Low	Low

A “**Significant**” consequence classification is recommended for the TSF at a final crest elevation of 1 001 mamsl in terms of the GISTM Classification Matrix. This recommendation is based on the potential environmental risk and potential population at risk identified for all identified failure scenario’s. Environmental impacts were only evaluated at scoping level and an environmental impact specialist study is recommended to confirm this classification.

14.7 SENSITIVITY ANALYSIS

Sensitivity analyses were completed to evaluate the influence of key assumptions on the most critical failure scenarios. The main findings of the sensitivity analyses are as follow:

- Rheology: The identified impacts of both sensitivity scenarios (increased and decreased viscosity) indicate significant differences in inundation areas to that of the main scenario. This indicates that the derived Consequence Classification is sensitive to assumptions made with regards to the rheological properties of the tailings.

14.8 CONCLUSION AND RECOMMENDATIONS

It is recommended that the following measures be implemented/installed/maintained at critical locations as part of the Emergency Preparedness and Response Plan (EPRP):

- Effective early warning systems and evacuation procedures, including sirens and emergency lighting.
- Measures to block access downstream of the TSF, should a breach event occur.
- Moratorium on further development within potentially impacted areas immediately downstream of the TSF, in consultation with local authorities.
- The derived Consequence Classification for the facility is not sensitive to the assumptions made with respect to the shape of the failure shape.
- The breach development time does have a significant impact on the overall inundated extents, considering a sunny-day failure, with an increased inundation resulting from sudden, brittle slope failure. Indicating that the estimated area of inundation is sensitive to the adopted failure mode. It is recommended that an investigation be carried out to confirm if brittle slope failure is credible. If confirmed, then this TDBA study must be updated to include detailed inundation mapping and reporting for this type of failure.

The failure scenarios evaluated as part of the TDBA were developed based on the latest survey information. The study would need to be updated prior to closure, and during operations if:

- Actual average operating pond volumes exceed the pond volumes considered herein.
- If the specific gravity or in-situ density of the tailing's density differs significantly from the values assumed.
- Future CPTu data indicates a significant change to the Peak and Residual shear strength ratios assumed in this study.

The TSF is raised beyond the level evaluated as part of this TDBA.

15.0 SANS 10286 DAM SAFETY CLASSIFICATION

Safety classification of the tailing's facility, in accordance with the criteria in SANS 10286:1998 "Code of practice, Mine residue" (The South African Bureau of Standards (SABS), 1998), is dependent upon the zone of influence of the facility. This is the area around the dam in which a failure would have the effect of causing loss of life, damage to property and pollution of the environment. The code prescribes the aims, principles and minimum requirements that apply to the classification procedure and the classification in turn gives rise to minimum requirements for investigation, design, construction, operation and decommissioning.

The boundary of the zone of influence (ZOI) is determined as follows (where h is the height of the facility at the point under consideration):

a) Upstream of any point on the perimeter, the lesser of a distance of 5h from the toe; and the distance to the point where the ground level exceeds h/2 above the elevation of the toe at the point on the perimeter.

b) On the sides parallel to the ground slope – a distance of 10h from the toe.

c) Downstream of the lowest point on the perimeter – a distance of 100h up to a maximum of 6km.

The ZOI as per SANS10286 is presented in Figure 15-1.

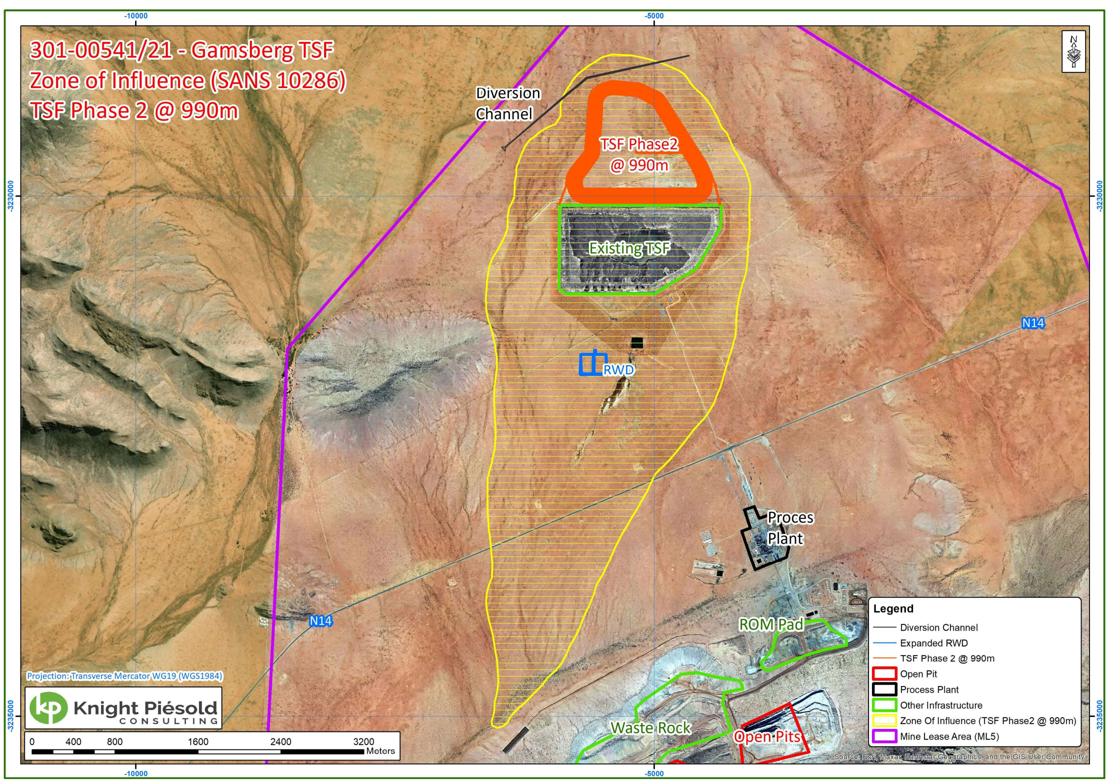


Figure 15-1: SANS 10286 ZOI

The ZOI as per SANS 10286 encompasses the site office and crosses the national highway. The TDBA should be considered as more accurate with the use of rheology, topography and flow.

Based on the ZOI the facility is classified as medium hazard facility. This is due to:

- There are no residents who live in the ZOI. The ZOI affects the national highway, the highway has low traffic volumes.
- To the south, downstream of the TSF is the site office, which is within the ZOI, and it is assumed that greater than 11 but less then 100 workers will be present in this area. Additionally some limited infrastructure will be in the ZOI for the Phase 2 plant.
- There is no third-party property.
- No underground working in ZOI (Mine is opencast)

The table below shows the SANS10286 hazard classification method with highlighted designations. As no third-party property exists no escalation of the SANS 10286 third property values is required.

Table 15-1: SANS10286 hazard classification

Classification	Number of residents in zone of influence	Number of workers in zone of influence[1]	Value of third-party property in zone of influence[2]	Depth to underground mine workings[3]
High Hazard	>10	> 100	> R20million	< 50m
Medium Hazard	1 - 10	11 – 100	R2million – R20million	50m – 200m
Low Hazard	0	<10	R0million – R2million	> 200m

[1] Not including workers employed solely for the purposes of operating the deposit.
[2] Values are as per SANS 10286 1998
[3] The potential for collapse of the residue deposit into the underground workings effectively extends the zone of influence to below ground level.

In conclusion, SANS10286 determines the facility to be **medium hazard.**

16.0 TAILINGS SLURRY RINGFEED SYSTEM

The detailed design of the Tailings slurry ringfeed system was performed following the design criteria document (Knight Piésold (Pty) Ltd., 2023) developed that outlines the following key aspects:

- Summary descriptions of the Tailings and Return decant pipeline system (with schematics and General arrangement drawings)
- Scope of work and Battery limits
- Mechanical and Civil Design Criteria
- Key Infrastructure Information
- Electrical and Control and Instrumentation Design Criteria

16.1 SLURRY PIPE SIZING

The slurry operating range and flow behaviour are, among other things, dependent on the material properties of the slurry, such as particle size distribution, specific gravity of solids, solids concentration and other rheological parameters.

The **Design operating envelope** (based on project required throughput and density range) was evaluated against the **Acceptable system operating envelope** (based on system limits) for the following pipe sizes as finalised for the slurry pipeline system:

- **External ringfeed pipeline – DN300 STD Schedule Steel epoxy coated pipe (with 10 mm HDPE lining)**
- **Offtake riser pipe from the external ringfeed – DN400 PE100 PN25**
- **Internal ringfeed pipeline – DN315 PE100 PN16**
- **11 number of Offtake pipes to cyclones – DN125 PE100 PN10**
- **8 number of Offtake pipes to cyclones – DN125 PE100 PN10**

The operating envelopes for each of the above pipe sizes are shown with the slurry operating range figures and evaluations included under section 16.1.1 to 16.1.5. The following legend applies to the figures shown under these sections.

- Red diagonal lines: Represents the slurry density concentrations for the considered pipe size.
- Thin black horizontal lines: Represents the instantaneous minimum, normal and maximum solid throughputs within the safe operating system limits.
- Grey filled area: Acceptable system operating envelope (based on the safe operating system limits).
- Orange line: Theoretically estimated deposition velocities as per Wilson and Judge (mixed regime) equation for the considered pipe size.
- Light blue dashed vertical line: Recommended minimum velocity calculated as the maximum theoretically estimated deposition velocities plus a safety margin of 0.25 m/s.

16.1.1 EXTERNAL RINGFEED PIPELINE – DN300 STD SCHEDULE STEEL EPOXY COATED PIPE (WITH 10 MM HDPE LINING)

The operating envelopes for the **DN300 STD SCHEDULE STEEL EPOXY COATED PIPE (WITH 10 MM HDPE LINING)** pipe segment are shown with the figure below:

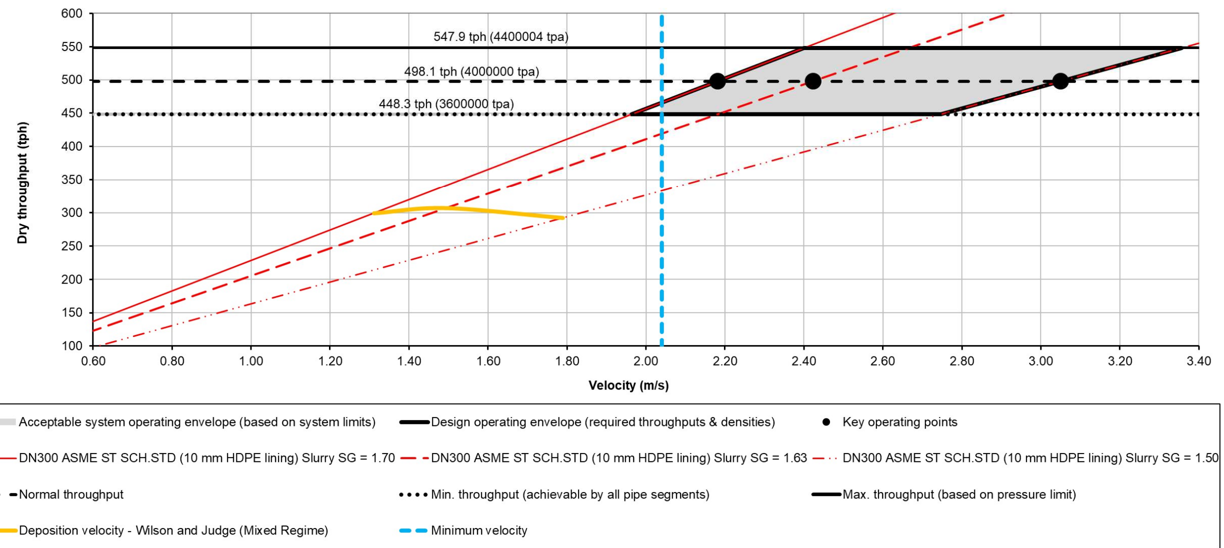


Figure 16-1: Slurry operating envelopes for the DN300 STD SCHEDULE STEEL EPOXY COATED PIPE (WITH 10 MM HDPE LINING)

The following conclusions are made from the figure:

- Key operating duty points at normal throughput (498.1 tph)
  - The operating velocities at these duty points *[2.18 m/s, 2.42 m/s and 3.05 m/s (at 1.7 t/m^3, 1.63 t/m^3 and 1.5 t/m^3]* have sufficient margin of safety above the corresponding calculated stationary deposition velocities *[1.31 m/s, 1.49 m/s and 1.79 m/s]* and they are well beyond the minimum velocity of 2.04 m/s.
- Design operating envelope (448.3 tph to 547.9 tph)
  - Operation within the *Design operating envelope* is mostly within the *Acceptable system operating envelope* and considered acceptable.

16.1.2 OFFTAKE RISER PIPE FROM THE EXTERNAL RINGFEED – DN400 PE100 PN25

The operating envelopes for the **DN400 PE100 PN25** are shown with the figure below:

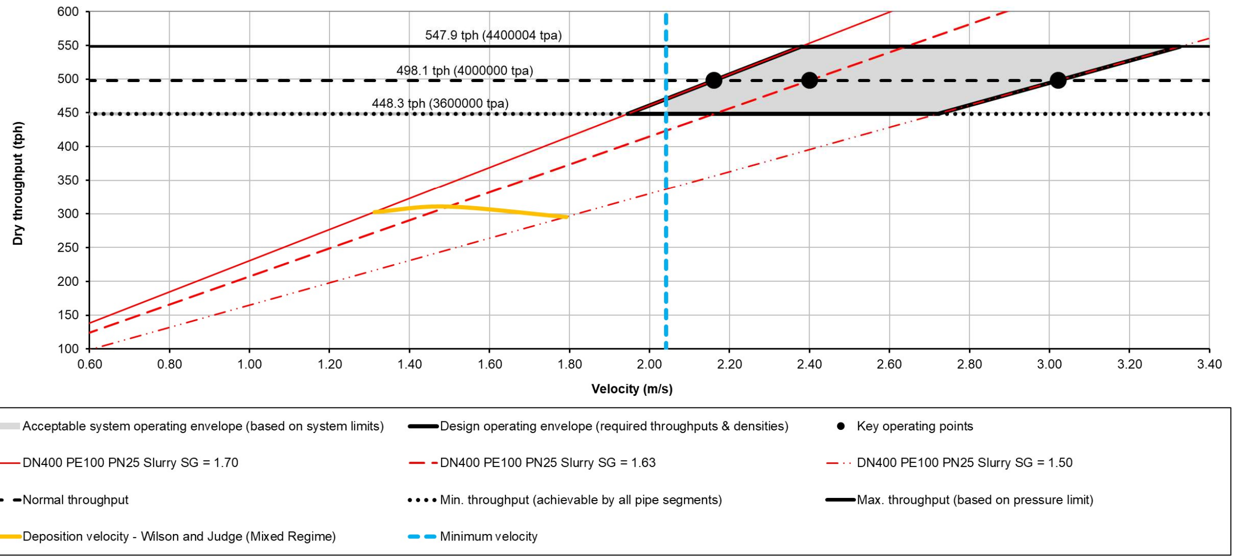


Figure 16-2: Slurry operating envelopes for the DN400 PE100 PN25

The following conclusions are made from the figure:

- Key operating duty points at normal throughput (498.1 tph)
  - The operating velocities at these duty points *[2.16 m/s, 2.40 m/s and 3.02 m/s (at 1.7 t/m^3, 1.63 t/m^3 and 1.5 t/m^3]* have sufficient margin of safety above the corresponding calculated stationary deposition velocities *[1.31 m/s, 1.5 m/s and 1.79 m/s]* and they are well beyond the minimum velocity of 2.04 m/s.
- Design operating envelope (448.3 tph to 547.9 tph)
  - Operation within the *Design operating envelope* is mostly within the *Acceptable system operating envelope* and considered acceptable.

16.1.3 INTERNAL RINGFEED PIPELINE – DN355 PE100 PN16

The operating envelopes for the **DN355 PE100 PN16** are shown with the figure below:

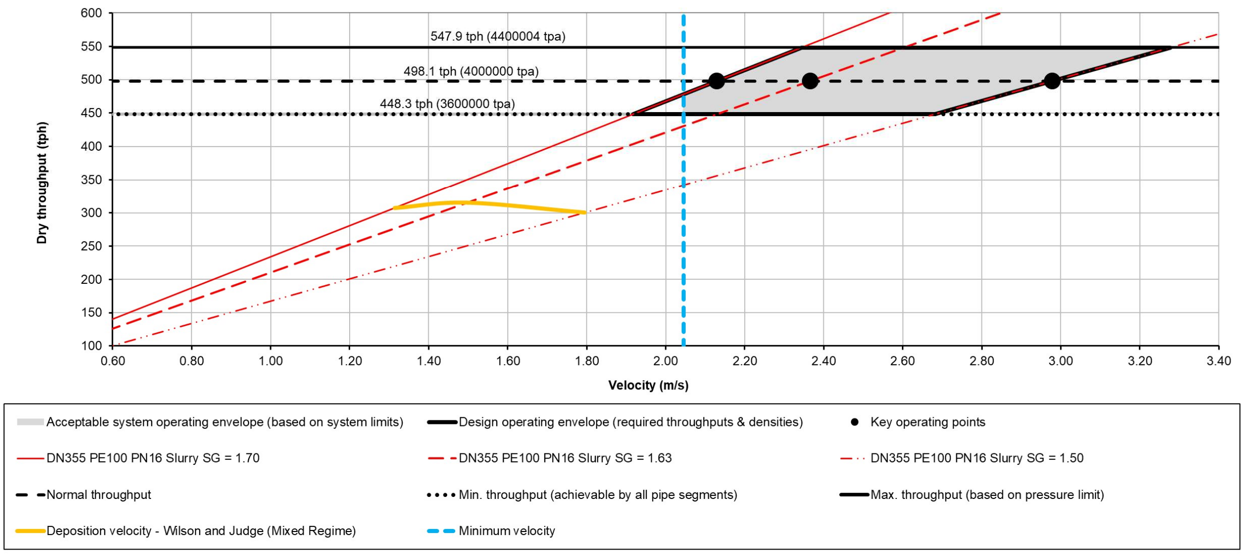


Figure 16-3: Slurry operating envelopes for the DN355 PE100 PN16

The following conclusions are made from the figure:

- Key operating duty points at normal throughput (498.1 tph)
    - The operating velocities at these duty points *[2.13 m/s, 2.37 m/s and 2.98 m/s (at 1.7 t/m^3, 1.63 t/m^3 and 1.5 t/m^3]* have sufficient margin of safety above the corresponding calculated stationary deposition velocities *[1.31 m/s, 1.5 m/s and 1.8 m/s]* and they are well beyond the minimum velocity of 2.04 m/s.
- Design operating envelope (448.3 tph to 547.9 tph)
    - Operation within the *Design operating envelope* is mostly within the *Acceptable system operating envelope* and considered acceptable

16.1.4 11 NUMBER OF OFFTAKE PIPES TO CYCLONES – DN125 PE100 PN10

The operating envelopes for the **DN125 PE100 PN10** (**with 11 number of offtake pipes to cyclones)** are shown with the figure below:

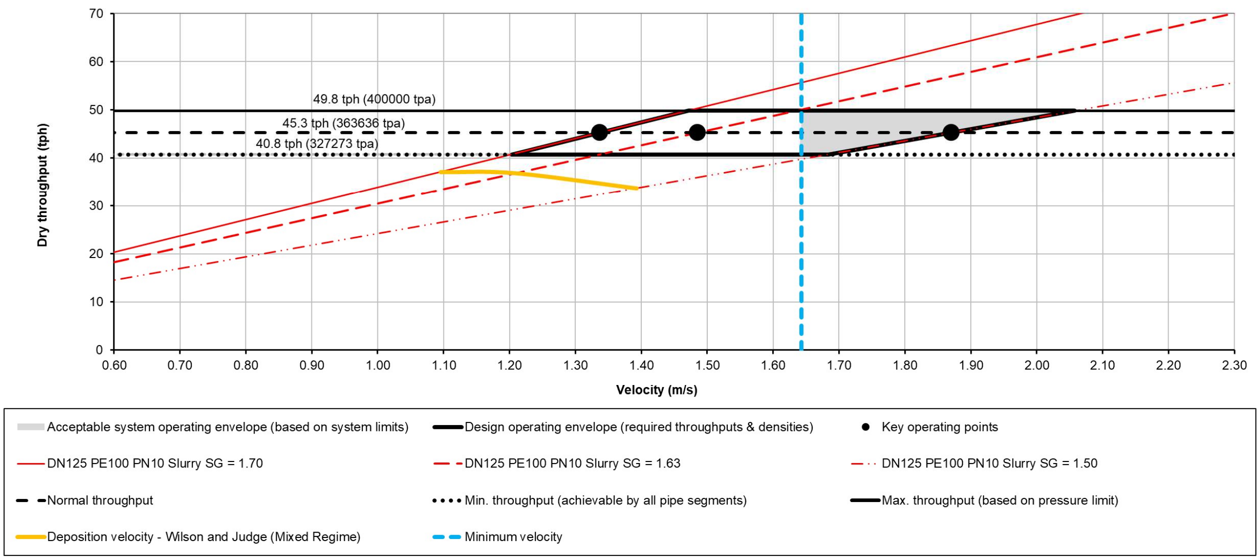


Figure 16-4: Slurry operating envelopes for the DN125 PE100 PN10 (with 11 number of offtake pipes to cyclones)

The following conclusions are made from the figure:

- Key operating duty points at normal throughput per pipe (45.3 tph)
  - The operating velocities at these duty points *[1.34 m/s, 1.49 m/s and 1.87 m/s (at 1.7 t/m^3, 1.63 t/m^3 and 1.5 t/m^3]* have reasonable margin of safety above the corresponding calculated stationary deposition velocities *[1.1 m/s, 1.21 m/s and 1.39 m/s]* although not above the minimum velocity of 1.643 m/s for the first two duty points.
- Design operating envelope per pipe (40.8 tph to 49.8 tph)
  - Operation within the *Design operating envelope* is within the *Acceptable system operating envelope,* considering that 11 number of cyclones will be the maximum number of cyclones ever operated and that less cyclones can per operated should the system run into deposition issues.

16.1.5 8 NUMBER OF OFFTAKE PIPES TO CYCLONES – DN125 PE100 PN10

The operating envelopes for the **DN125 PE100 PN10 size (with 8 number of Offtake pipes to cyclones** are shown with the figure below:

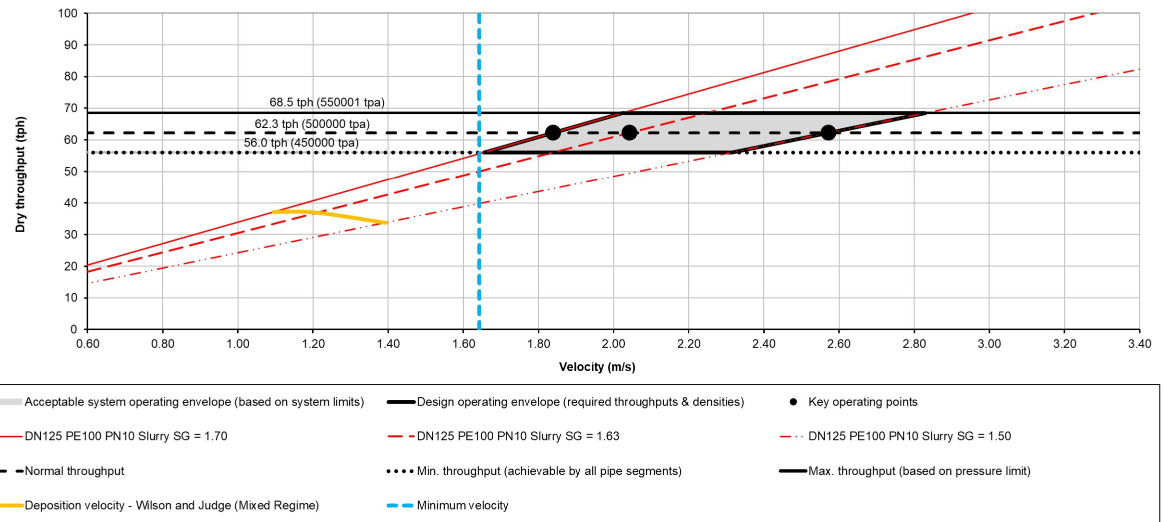


Figure 16-5: Slurry operating envelopes for the DN125 PE100 PN10 (with 8 number of Offtake pipes to cyclones)

The following conclusions are made from the figure:

- Key operating duty points at normal throughput per pipe (62.3 tph)
  - The operating velocities at these duty points *[1.84 m/s, 2.04 m/s and 2.57 m/s (at 1.7 t/m^3, 1.63 t/m^3 and 1.5 t/m^3]* have reasonable margin of safety above the corresponding calculated stationary deposition velocities *[1.1 m/s, 1.21 m/s and 1.39 m/s]* and they are well beyond the minimum velocity of 1.643 m/s.
- Design operating envelope per pipe (56 tph to 68.5 tph)
  - Operation within the Design operating envelope is within the Acceptable system operating envelope.

16.2 TAILINGS PIPELINE SEGMENT DETAILS AND RESULTING PUMPING HEADS

The pipeline segment details and resulting head losses and pumping heads for the proposed Tailings slurry delivery system at final height are outlined within the table below. The hydraulics were calculated from chainage 0 (as specified within KP's long section drawing).

Table 16.1: Pipeline segment details and resulting head losses and pumping heads – Proposed Tailings slurry ringfeed system at Final height (from chainage 0 on KP long section drawing to the furthest deposition point on the TSF along the Western branch)

Description	Unit	Max. throughput @ max. slurry density	Maximum throughput @ min. slurry density	Normal throughput @ normal slurry density	Min. throughput @ min. slurry density
Tailings dry throughput per year	**dry tpa**			**4 000 000**	
Tailings dry throughput per month	**dry tpm**	**366 667**	**366 667**	**333 333**	**300 000**
Solids density	t/m³	3.386	3.386	3.386	3.386
Slurry density	t/m³	1.700	1.500	1.630	1.500
Density of water	t/m³	0.9982	0.9982	0.9982	0.9982
Specific weight of slurry	kN/m³	16.67	14.71	15.98	14.71
Volumetric concentration of slurry	%v	29.39	21.02	26.46	21.02
Mass concentration of slurry	%m	58.54	47.44	54.96	47.44
Monthly slurry tonnage	**tpm**	**626 352**	**772 936**	**606 454**	**632 401**
Monthly slurry volume	**m³ / month**	**368 442**	**515 291**	**372 058**	**421 601**
Monthly slurry : Volume of solids	m³ / month	108 289	108 289	98 445	88 600
Monthly slurry : Volume of water	m³ / month	260 153	407 001	273 613	333 001
hours per annum	%	8030	8030	8030	8030
Instantaneous slurry/mixture volume	**m³ / h**	**551**	**770**	**556**	**630**
	l/s	153	214	154	175
Instantaneous solids throughput	tph	548	548	498	448
1st Pipe Segment (External ringfeed pipeline)		**A**			
Pipe Details		**DN300 ASME ST SCH.STD (10 mm HDPE lining)**			
No. of Pipe	No.	1			
Pipe length	m	1 233			
Pipe Inside Diameter (ID)	m	0.28486			
Flow velocity	m/s	2.40	3.36	2.42	2.75
Pipeline friction pressure gradient	Pa/m	322	523	309	353
Pipeline friction hydraulic gradient	m/m	0.019	0.036	0.019	0.024
2nd Pipe Segment (Offtake riser pipe from the external ringfeed)		**B**			
Pipe Details		**DN400 PE100 PN25**			
No. of Pipe	No.	1			
Pipe length	m	155			
Pipe Inside Diameter (ID)	m	0.2862			
Flow velocity	m/s	2.38	3.32	2.40	2.72
Pipeline friction pressure gradient	Pa/m	315	510	301	345
Pipeline friction hydraulic gradient	m/m	0.019	0.035	0.019	0.023
3rd Pipe Segment (Internal ringfeed pipeline)		**C**			
Pipe Details		**DN355 PE100 PN16**			
No. of Pipe	No.	1			
Pipe length	m	899			
Pipe Inside Diameter (ID)	m	0.2883			

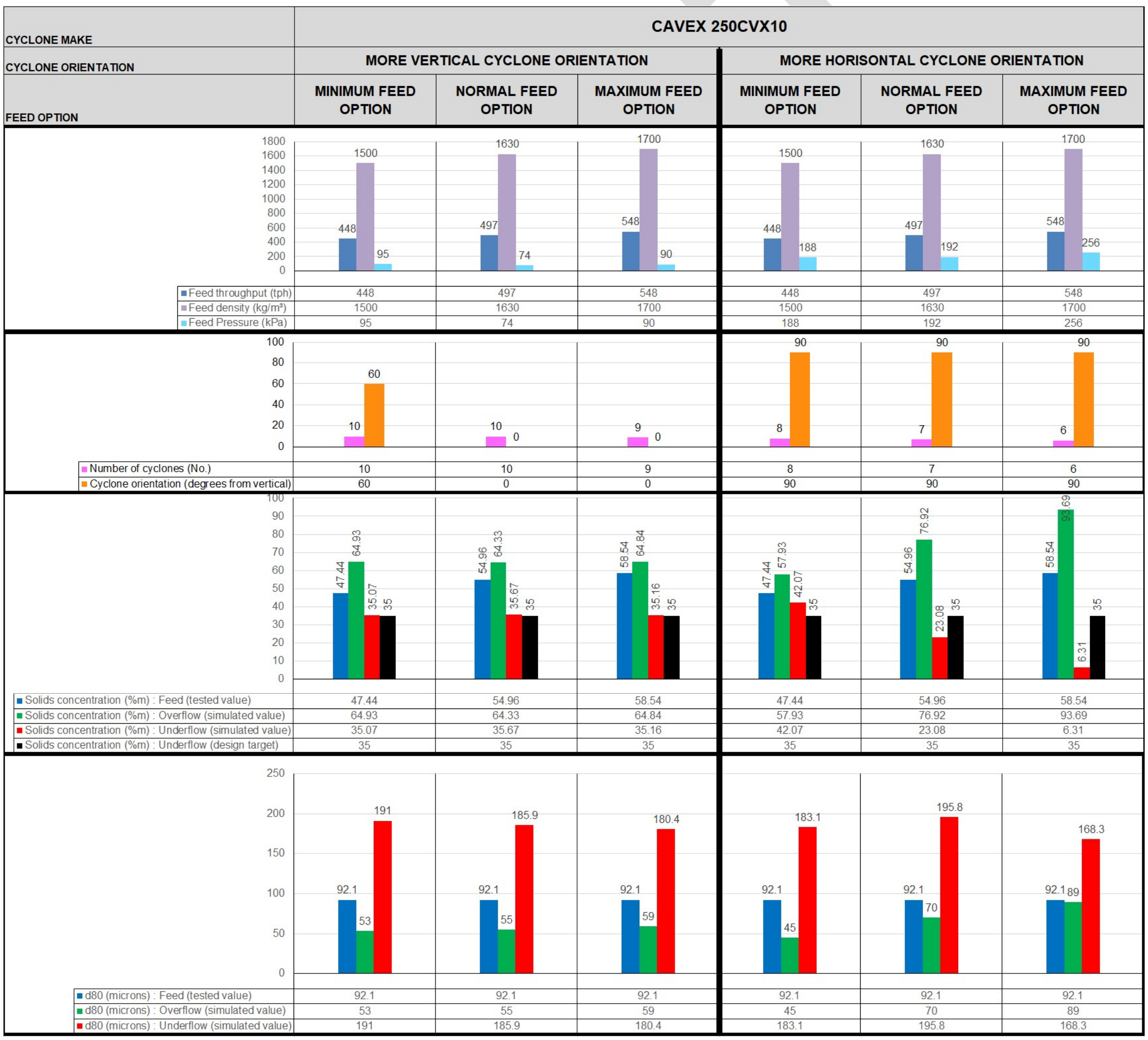
Description	Unit	Max. throughput @ max. slurry density	Maximum throughput @ min. slurry density	Normal throughput @ normal slurry density	Min. throughput @ min. slurry density
Flow velocity	m/s	2.34	3.28	2.37	2.68
Pipeline friction pressure gradient	Pa/m	303	491	290	332
Pipeline friction hydraulic gradient	m/m	0.018	0.033	0.018	0.023
4th Pipe Segment (Offtake pipe to cyclone, operating 6 cyclones – worst case consideration)		**D**			
Pipe Details		**DN125 PE100 PN10**			
No. of Pipe	No.	6			
Pipe length	m	20			
Pipe Inside Diameter (ID)	m	0.12285			
Flow velocity	m/s	2.15	3.01	2.17	2.46
Pipeline friction pressure gradient	Pa/m	1 319	2 145	1 265	1 446
Pipeline friction hydraulic gradient	m/m	0.079	0.146	0.079	0.098
5th Pipe Segment (Overflow pipe from cyclone, operating 6 cyclones – worst case consideration)		**E**			
Pipe Details		**DN160 PE100 PN10**			
No. of Pipe	No.	6			
Pipe length	m	20			
Pipe Inside Diameter (ID)	m	0.1404			
Flow velocity	m/s	1.65	2.30	1.66	1.88
Pipeline friction pressure gradient	Pa/m	377	600	359	407
Pipeline friction hydraulic gradient	m/m	0.023	0.041	0.022	0.028
Summary					
Total combined length	**m**	**2 327**			
Total combined friction loss (hf)	**m**	**45.15**	**82.95**	**45.11**	**56.08**
Total combined friction loss	Pa	753	1 220	721	825
Maximum slurry elevation (at start)		947			
Minimum slurry elevation (at start)		947			
Maximum slurry elevation (at end point)		990			
Minimum slurry elevation (at end point)		990			
Static head difference (Hs)	**m**	**43.02**	**43.02**	**43.02**	**43.02**
		(Hs max.)	(Hs max.)	(Hs Ave.)	(Hs Min.)
Maximum static head difference in slurry pressure	kPa	717	633	688	633
Minor head losses	**m**	**15.59**	**15.59**	**14.49**	**14.98**
Minor pressure losses	kPa	260	229	232	220
Total slurry hydraulic head (excluding de-rating factors)	**m**	**103.76**	**141.56**	**102.62**	**114.08**
Total slurry pressure	kPa	1 730	2 082	1 640	1 678

The worst-case pressure scenario (Maximum throughput @ min. slurry density) has been highlighted in light red above and is shown within the hydraulic gradient under 16.4 (line in red).

16.3 SUMMARY OF CYCLONE SIMULATION RESULTS

A graphical summary of the cyclone simulation results (from Weir minerals) are shown below for the CAVEX 250CVX10 cyclone and the following two key simulation approaches followed namely, more vertical cyclone orientation and more horizontal cyclone orientation. As can be seen, for a fully horizontal orientation

- and the maximum feed option: the design target underflow by mass of 35%m is not achieved (only 6.3%m is achieve) which will prompt the need to orientate the cyclone closer to a more vertical orientation.
- and the normal feed option: an underflow by mass of 23%m is achieved, which is considered reasonable considering that configuring the cyclone closer to vertical will move the recovery closer to the design target of 35%m.



CYCLONE MAKE	CAVEX 250CVX10					
CYCLONE ORIENTATION	MORE VERTICAL CYCLONE ORIENTATION			MORE HORISONTAL CYCLONE ORIENTATION		
FEED OPTION	MINIMUM FEED OPTION	NORMAL FEED OPTION	MAXIMUM FEED OPTION	MINIMUM FEED OPTION	NORMAL FEED OPTION	MAXIMUM FEED OPTION
Feed throughput (tph)	448	497	548	448	497	548
Feed density (kg/m³)	1500	1630	1700	1500	1630	1700
Feed Pressure (kPa)	95	74	90	188	192	256
Number of cyclones (No.)	10	10	9	8	7	6
Cyclone orientation (degrees from vertical)	60	0	0	90	90	90
Solids concentration (%m) : Feed (tested value)	47.44	54.96	58.54	47.44	54.96	58.54
Solids concentration (%m) : Overflow (simulated value)	64.93	64.33	64.84	57.93	76.92	93.69
Solids concentration (%m) : Underflow (simulated value)	35.07	35.67	35.16	42.07	23.08	6.31
Solids concentration (%m) : Underflow (design target)	35	35	35	35	35	35
d80 (microns) : Feed (tested value)	92.1	92.1	92.1	92.1	92.1	92.1
d80 (microns) : Overflow (simulated value)	53	55	59	45	70	89
d80 (microns) : Underflow (simulated value)	191	185.9	180.4	183.1	195.8	168.3

Figure 16.6: Graphical summary of cyclone simulation results (from Weir minerals)

16.4 TAILINGS HYDRAULIC GRADIENT

The maximum and minimum tailings hydraulic gradient profiles for the proposed tailings ringfeed pipeline system at are shown in the figure below. The hydraulic gradients and results incorporate a cyclone feed pressure of 188kPa operating 8 number of cyclones at a time (as a worst pressure case modelling scenario).

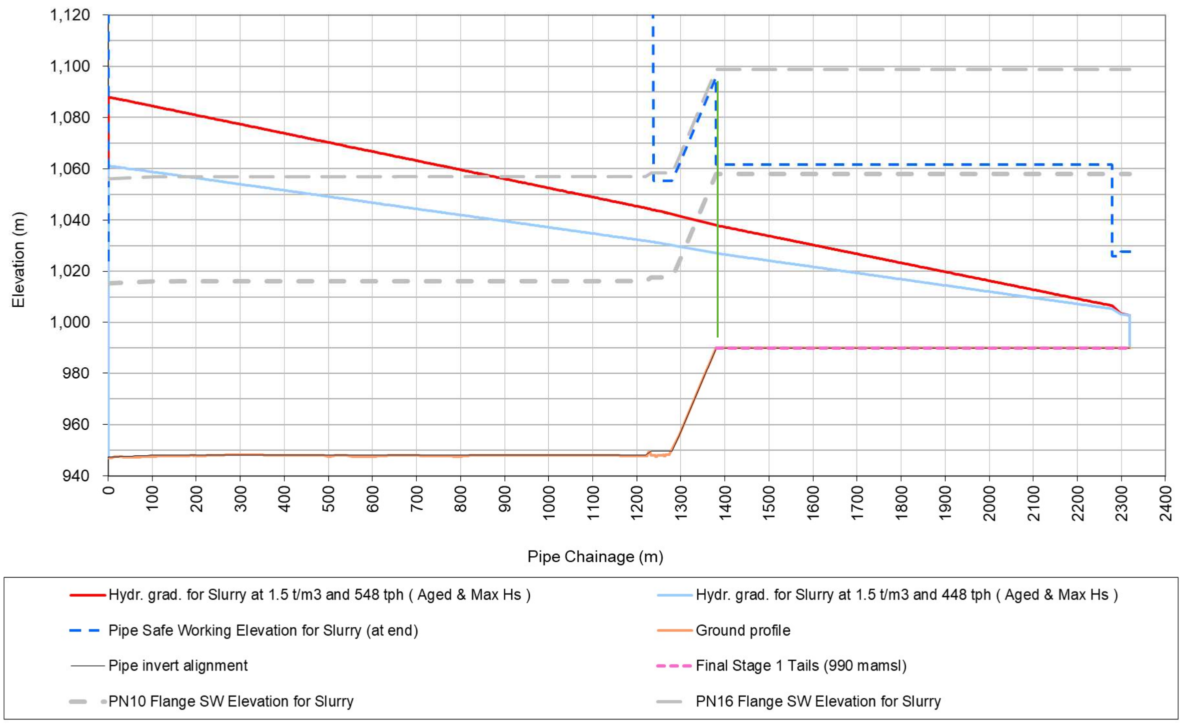


Table 16.2: Hydraulic gradient– Proposed Tailings slurry ringfeed system (from chainage 0 on KP long section drawing to the furthest deposition point on the TSF along the Western branch)

According to pipe safe working pressure calculation the above steady state hydraulic gradient shows that it does not exceed the design safe working pressure capacity at any point along the pipe route. This confirms the appropriateness of the pipe size and pressure class for the proposed pipeline system.

17.0 CONSTRUCTION QUALITY ASSURANCE

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

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

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

18.0 DRAWINGS

A set of drawings has been provided within Appendix I of this report. In summary a list of drawings is provided in Table 18-1: Gamsberg Mine Phase 2 TSF Drawing List. A more complete set of drawings is being prepared for the construction, the drawings list below is presented for the purpose of water use license application.

Table 18-1: Gamsberg Mine Phase 2 TSF Drawing List

Drawing Number	Description
ZI-GAM02-U5910-ENG-CL-0000-SH01	GENERAL-DRAWING LIST-
ZI-GAM02-U5910-LAY-CL-0001-SH01	GENERAL-SITE LOCATION-PLAN
ZI-GAM02-U5910-LAY-CL-0002-SH01	GENERAL-PLANT AND TSF-LAYOUT
ZI-GAM02-U5910-LAY-CL-0003-SH01	GENERAL-TAILINGS STORAGE FACILITY-LAYOUT
ZI-GAM02-U5910-SLP-CL-0004-SH01	GENERAL-LIFE CYCLE-PLAN
ZI-GAM02-U5910-SET-CL-0005-SH01	GENERAL-SETTING OUT POINTS-LAYOUT
ZI-GAM02-U5910-LAY-CL-0010-SH01	GENERAL-GEOTECHNICAL TEST PIT LOCATIONS-LAYOUT
ZI-GAM02-U5910-LAY-CL-0011-SH01	GENERAL-BORROW PIT AREAS-LAYOUT
ZI-GAM02-U5910-LAY-CL-0012-SH01	GENERAL-CUT & FILL TAILINGS STORAGE FACILITY LAYOUT-LAYOUT
ZI-GAM02-U5910-LAY-CL-0020-SH01	GENERAL-BATTERY LIMITS-LAYOUT
ZI-GAM02-U5910-LAY-CL-0110-SH01	TSF-STARTER WALLS-LAYOUT
ZI-GAM02-U5910-SEC-CL-0112-SH01	TSF-STARTER WALLS-SECTIONS
ZI-GAM02-U5910-SEC-CL-0113-SH01	TSF-STARTER WALLS-LONG SECTION
ZI-GAM02-U5910-SEC-CL-0115-SH01	TSF-EMBANKMENT-SECTIONS
ZI-GAM02-U5910-LAY-CL-0120-SH01	TSF-LINER SYSTEM-LAYOUT
ZI-GAM02-U5910-LAY-CL-0210-SH01	DECANT SYSTEM-MAIN DECANT-LAYOUT
ZI-GAM02-U5910-SEC-CL-0211-SH01	DECANT SYSTEM-MAIN DECANT-SECTIONS
ZI-GAM02-U5910-CLR-CL-0260-SH01	DECANT SYSTEM-SILT TRAP-LAYOUT AND SECTIONS
ZI-GAM02-U5910-DET-CL-0261-SH01	DECANT SYSTEM-SILT TRAP-DETAILS AND SECTIONS
ZI-GAM02-U5910-LAY-CL-0310-SH01	RETURN WATER DAM-RWD WALLS-LAYOUT
ZI-GAM02-U5910-SEC-CL-0311-SH01	RETURN WATER DAM-RWD WALLS-SECTIONS
ZI-GAM02-U5910-SEC-CL-0330-SH01	RETURN WATER DAM-SPILLWAY-SECTIONS
ZI-GAM02-U5910-LAY-CL-0340-SH01	STORM WATER DAM-SWD-LAYOUT

ZI-GAM02-U5910-DRA-CL-0415-SH01	DRAINAGE SYSTEM-UNDERDRAINAGE-LAYOUT
ZI-GAM02-U5910-DRA-CL-0416-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN EAST 1 & 2
ZI-GAM02-U5910-DRA-CL-0417-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN WEST 1 & 2
ZI-GAM02-U5910-DRA-CL-0418-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN INT WEST A & 1
ZI-GAM02-U5910-DRA-CL-0419-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN INT WEST 2 & 3
ZI-GAM02-U5910-DRA-CL-0420-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN INT EAST 1 & 2
ZI-GAM02-U5910-DRA-CL-0421-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN INT EAST 3 & 4
ZI-GAM02-U5910-DRA-CL-0422-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN INT SOUTH 1, DRAIN SOUTHWEST 1, 1A, 1B, 2, 2A & 2B
ZI-GAM02-U5910-DRA-CL-0423-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN INT SOUTHEAST 1, 1A, 1B, 2, 2A & 2B
ZI-GAM02-U5910-DRA-CL-0424-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN SOUTH EXTERNAL
ZI-GAM02-U5910-DRA-CL-0425-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN EXTERNAL WEST
ZI-GAM02-U5910-DRA-CL-0426-SH01	DRAINS-LONG SECTIONS WITH PIPE LIST-DRAIN EXTERNAL EAST
ZI-GAM02-U5910-DRA-CL-0427-SH01	DRAINAGE SYSTEM-UNDERDRAINAGE-LAYOUT AND SECTIONS
ZI-GAM02-U5910-DRA-CL-0428-SH01	DRAINAGE-LEAKAGE DETECTION UNDERDRAINAGE-LAYOUT
ZI-GAM02-U5910-DRA-CL-0429-SH01	GENERAL DRAINAGE SYSTEM-DRAINS LONG SECTIONS WITH PIPE LIST-DRAINAGE SECTIONS
ZI-GAM02-U5910-DET-CL-0430-SH01	DRAINAGE SYSTEM-DRAINAGE-DETAILS
ZI-GAM02-U5910-DET-CL-0432-SH01	DRAINAGE SYSTEM-TOE DRAINS-LAYOUT AND SECTIONS
ZI-GAM02-U5910-DET-CL-0440-SH01	DRAINAGE SYSTEM-SUMPS-DETAILS
ZI-GAM02-U5910-LAY-CL-0610-SH01	STORMWATER MANAGEMENT-DIVERSION CHANNEL-LAYOUT
ZI-GAM02-U5910-SEC-CL-0620-SH01	STORMWATER MANAGEMENT-DIVERSION CHANNEL-LONG SECTION
ZI-GAM02-U5910-DET-CL-0816-SH01	MISCELLANEOUS-FENCING-DETAILS
ZI-GAM02-U5910-LAY-CL-0820-SH01	MISCELLANEOUS-ROADS-LAYOUT
ZI-GAM02-U5910-SEC-CL-0821-SH01	MISCELLANEOUS-ROADS-SECTIONS

19.0 BILL OF QUANTITIES

The estimated capital costs for the Phase 2 TSF construction are summarised in Table 19-1. This includes an allowance for Contractors Preliminary and General and excludes the cost of pipes, pumps and instrumentation associated with the slurry delivery line and return water dam.

Table 19-1: Summary of Bill of Quantities

Section Summary	AMOUNT (ZAR)
P&Gs	**118 765 632,00**
General	118 765 632,00
TSF	**463 858 807,66**
Access Roads	598 064,29
Basin	243 136 354,55
Liner	220 124 388,82
RWD	**38 755 441,29**
Basin	24 102 060,36
Liner	14 653 380,93
Instrumentation	**TBD**
Basin	TBD
Embankment	TBD
Pumps and Pipes	**TBD**
Basin	TBD
Grand Total	**621 379 880,95**

20.0 OPERATION

20.1 AIMS AND CONSTRAINTS

Operation of the TSF must comply with the following standards and acts. In the case of conflicting standards, the more onerous standard applies unless otherwise agreed:

- South African, SANS 10286 'Code of practice (The South African Bureau of Standards (SABS), 1998)
- Global Industry Standard for Tailings Management (GISTM,2020)

The complete operation guidance is given with the operation manual for the Gamsberg TSF complex. The method described within this report highlights some key SANS 10286 requirements.

20.2 OPERATION METHOD

The TSF will use cyclones to deposit tailings and perform raises using a combination of downstream, centreline and upstream wall building techniques. For the full operation manual which includes a comprehensive description of the operation method, please refer to Appendix N.

20.3 MONITORING

As per SANS 10286 only suitably qualified personnel are to operate the tailings dam complex. In addition, a suitably qualified and certified engineer named the Engineer of Record (EoR) should inspect the facility at the required interval as per the hazard rating on the SANS 10286 or the CCS type for GISTM compliance based. The monitoring procedures are reiterated within the operation manual in Appendix N.

20.3.1 DAILY LOGBOOK

A daily logbook should always be kept on site. The contents of this book are summarised below:

- the date.
- the weather conditions.
- any performance monitoring data recorded during the day
- the day's activities, e.g., slurry deposition times, location, decant times, quality of water, equipment.
- materials changes and decommissioning activities.
- visitors to the TSF.
- notes of conversations, instructions given and received.
- materials required or requested.
- labour and equipment used
- incidents, i.e., accidents and spillages and notes of the actions taken; and
- notes of inspections conducted and by whom.

20.3.2 INSPECTIONS

Inspections serve as performance monitoring and quality assurance system. Mine management of sufficient seniority should conduct a monthly inspection of the TSF, while quarterly inspections are carried out by the responsible engineer in conjunction with a review meeting. A copy of the previous inspection should be referred to check on the execution of corrective actions. Details of the inspection checklist will be provided in the operations manual. The inspection checklist should contain the following as per SANS 10286:

- the name of the mine.
- the name/number of facilities.
- the date/time of inspection.
- the name(s) and signatures of persons conducting the inspection.
- an item for every aspect of the facility (e.g., roads, fences, pipelines, benches, environmental aspects, etc.).
- a quantitative or qualitative performance measurement criterion for each item (i.e., an actual measured value or a qualitative rating of, say, 1 to 5 or excellent to poor, and, preferably, comments. A tick or cross against an item is meaningless).
- a sketch plan of the facility with marked reference beacons to allow geographical distinction of different qualities of the same item; and
- a system of identifying items that require action, priorities, and responsibilities

20.3.3 QUARTERLY REVIEW MEETING

SANS 10286 recommends that a quarterly review meeting attended by all responsible personal be held. These review meetings are a formal assessment of the development and performance of the TSF. The meeting agenda should include:

- the name of the mine.
- the name or the number of facilities.
- the date.
- the names of those present, apologies and a distribution list.
- previous minutes, which should be referred to.
- an item for every aspect of the facility (refer to checklist); each item should be dealt with at each meeting and either a "satisfactory" state should be minuted or some other condition should be described, and remedial action noted (a "no comment" minute is meaningless);
- action plans with priorities and responsibilities; and
- records of performance measurement data, which should be referred to in the minutes, where appropriate, as a means of acknowledgement and the records should be attached, if necessary

These minutes as well as the inspection checklists and monitoring data will form the record of the TSF.

20.3.4 ANNUAL DAM REPORT

An annual dam safety inspection is required. It should be completed by EoR and to include results and interpretation of findings in routine inspections and quarterly reports. Shall include interpretation and impact of monitoring data and trends.

20.3.5 INDEPENDENT TAILINGS REVIEW BOARD

Independent Tailings Review Board (ITRB) is a term for a group of senior engineers who perform an external audit of the tailing's facilities.

The appointment and selection of the ITRB must be performed by the mine.

20.4 PERFORMANCE MONITORING PARAMETERS

The parameters that will need to be measured are presented in the following sub sections. Please note that the frequency of measurements presented below serves as a minimum requirement. Should there be any visible distress, or any other reason stated in the operation manual, measurements should be increased accordingly. The list may not be exhaustive and the EoR may dictate other monitoring data to be provided.

20.4.1 PIEZOMETERS

Vibrating wire piezometers must be installed to measure the phreatic surface around the TSF. This data must be reported on monthly. The piezometers must be located at a depth below the phreatic surface. There will be 16 sections on the TSF as per drawings contained within Appendix I. More Piezometers may be installed at the discretion of the EoR based on the TSF condition.

20.4.2 CLIMATOLOGY

The TSF will have a rain gauge and evaporation pan installed on or near the TSF to monitor the evaporation and rainfall. This data should be recorded daily and reported monthly. If a large rainfall event occurs the EoR should be notified.

20.4.3 FREEBOARD

The Freeboard requirement for the TSF is 2 m. Georeferenced freeboard poles must be installed around the perimeter of the walls and the beach. An additional pole within the pond of the TSF should be installed to measure pond level. The spacing around the TSF perimeter walls should be 100 metres. These will be monitored monthly. Alternatively, regular surveys may be used to determine freeboard as well as deposition planning. This, however, does not allow the operator to instantaneously determine freeboard on an ad hoc basis, so it is preferable that freeboard poles should be installed, and regular surveys should be performed.

20.4.4 DECANT SYSTEM

The decant system should be operated daily. The decant hours should be recorded daily as well as the clarity of the decanted water by use of a turbidity wedge or similar approved procedure on the outlet side by the energy dissipater. This data must be data captured and reported on monthly.

20.4.5 UNDERDRAINS FLOW

The flow through the underdrains should be measured and reported monthly. Upon initial operation before the drains are covered, these flows will be monitored daily to check for any flow of solids through the system. During commissioning, initial deposition over the drains must be closely monitored by experienced personal to prevent fines from blinding the filter and to prevent erosion of the filter drain material into the basin.

20.4.6 TAILINGS PROPERTIES AND QUANTITY

The particle size distribution should initially be monitored monthly for a period of six months and if the material is consistent the frequency of testing may then be reduced. This will be determined by the EoR. The solids content of the slurry should be measured daily by means of a Marcy scale at the deposition point. This frequency may also be reviewed by the EoR.

The quantity of tailings deposited on the TSF should be recorded monthly as tonnages. In-situ densities should be calculated at each survey, at least annually. An appropriate on-site in-situ density test such as a sand replacement test should also be conducted yearly. The location of these tests should be dictated by the EoR

20.4.7 DUST

Various dust control methods exist for use during deposition, three are listed below:

- Application of materials which encapsulate the tailings by forming a hard crust,
- Dust nets which are strategically placed to reduce the wind velocities close to the TSF surface and to catch tailings particles which do lift off it,
- Water irrigation systems which consistently wet the surface.

The more permanent solution is either to vegetate or cap the TSF with adequate material which can be vegetated.

Dust buckets should be detailed and installed by suitably qualified personnel. The frequency at which the dust bucket result data should be recorded must also be determined by the relevant personnel.

20.4.8 WATER QUALITY

Water quality testing of the effluent generated from the TSF as well as ground water around the TSF must be performed. The monitoring frequency should be as per the water use license requirements.

20.4.9 SURFACE MONUMENTS

Four surface monuments will be installed at equal spacing on each new bench around the TSF. These should be measured at each survey. The minimum frequency is annually.

20.4.10 LINER TEMPERATURE

Temperature probes have been provided to measure the liner temperature to ensure that the design variables and intent will be met throughout the facility. These should be checked at least monthly as part of routine monitoring but preferably automatically logged and tracked with trigger temperatures set at 27°.

21.0 CLOSURE

The proposed TSF raise will be constructed and operated with final closure in mind. Cladding or vegetation must be established on all the outer side slopes and on the top surface of the TSF. Trials and activities related to rock cladding and/or establishment of a vegetation cover and associated irrigation systems must form part of the TSF operating cost allowances.

The closure provisions must comply with the EMPr report. The objective of establishing the closure requirements during the design phase is to reduce the capital cost of closure and maintenance on cessation of operations. This can be achieved by constructing outer slopes that will support vegetation and/or rock cladding and by concurrent rehabilitation of the outer slopes during operation. For Gamsberg rock cladding has been selected due its locality and low rainfall.

The TSF is an upstream constructed facility, which has the advantage over either centreline or downstream construction methods of enabling concurrent rehabilitation to take place during operation. The cost of concurrent rehabilitation should be included in the operating cost of the TSF.

It is proposed that tests be carried out on sections of the lower slopes of the TSF during the initial phases of post-recommissioning operation, to establish whether rock cladding will work as preferred solution for rehabilitation.

The objective for closure is to reduce post-closure maintenance and monitoring to a low or even to a negligible level. The following principles must be considered for closure.

a) The side slopes must be maintained so that they can be stabilised by cladding or vegetation.
b) The berms and/or benches must be maintained so that rainfall does not cause erosion, which will then lead to concentrated flows down the side slopes causing additional side slope erosion.
c) All pipework and other infrastructure associated with deposition and operation must be uplifted.
d) The phreatic surface is likely to decrease after closure and will eventually stabilize within the TSF. This will affect the drain maintenance requirements on the TSF.
e) The upper surface of the tailings dam should be shaped to avoid excessive ponding, and to promote evaporation of accumulated rainwater. Water management is a critical consideration.
f) The upper surface should be vegetated or clad in a similar manner to the outer slopes.
g) The solution trench must be maintained until the vegetation and/or rock cladding is stable to the extent that siltation of the solution trench ceases.

At closure, a closure report will need to be prepared by a Professional Engineer and according to SANS 10286:1998 must include, but not be limited to:

a) Closure objectives and criteria
b) Closure techniques
c) Post closure monitoring
d) Monitoring and performance measures
e) State and stability of the outer slopes
f) Physical and chemical stability of the TSF
g) Hydrological consideration with respect to the top of the TSF and run-off from the side of the dam
h) State of penstocks and penstock outfall conduit. (Not applicable for pumped system, the pump system should be removed, or adequacy evaluated if monitoring continues)
i) State of the filter drain outlets and solution trenches
j) Risk that the dam poses to the environment and safety.

21.1 PRE-ABANDONMENT PERIOD

The focus of the closure/post-closure strategy is to minimize erosion and promote landform stability. Concurrent with operations, the outer side slopes the facility will be built up with deposited underflow tailings to the required slopes and covered by rock fill armour to reduce the exposed tailings surface area to erosion by wind and water.

Being a cyclone constructed facility, concurrent rehabilitation work will be able to be carried out on the outer slopes of the facility. The overall outer slope will be 1:3 (V:H) with intermediate slopes between benches at 1:2.5 (V:H).

It is expected that decommissioning, rehabilitation, and closure of the facility at the cessation of operations will include the following:

- Removal of the slurry delivery and return water pipelines and all associated works,
- Sealing the gravity decant outlet,
- Removal of the synthetic liners from and landscaping of the return water dam,
- Upgrading the toe drains around the facility to ensure the containment of surface water runoff,
- Grading the top of the tailings surface to direct all runoff from the surface of the facility into perimeter water management structures. This process will involve the creation of drainage swales and interconnected depressions where intermittent run-off from precipitation events will be directed off the tailings surface.
- The surface will be graded so that runoff is directed towards low point(s) and, if required, an overflow rundown off the TSF will be included. Concurrent reclamation of the outer slopes of the TSF will begin the first year of operations. Formation of closure spillway will obviate future ponding on the surface of the TSF. See Drawing no. XXXXXXXXX for the Layout and Sections

21.2 LONG-TERM MAINTENANCE PERIOD

Aftercare and maintenance of the site is expected to comprise the repair of localised erosion gulley's and the maintenance of vegetation for a period of 3 to 5 years after completion of the rehabilitation and closure works described above. Monitoring of surface and groundwater quality in the area is likely to be required to continue for at least 5 years after closure.

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23.0 CERTIFICATION

This report was prepared and reviewed by the undersigned.

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APPENDIX A

24 hour Storm Rainfall Depths Statistical Analysis

Table A-1 shows the data used in the Reg Flood program (Alexander, et al., 2003) to produce the 24 hour rainfall depths for the 1 in 2, 1 in 10, 1 in 20, 1 in 50, 1 in 100 and 1 in 200 , 1 in 2 457, 1 in 5 000 and 1 in 10 000 recurrence intervals at the 0246555 W Rainfall Station.

Table A-1 Daily recorded maximum's for every year for the two rainfall stations used

Year	Maximum daily rainfall recorded (mm)
1950	19.2
1951	28.3
1952	15.1
1953	18.4
1954	27.4
1955	16.4
1956	20.6
1957	36.8
1958	12.3
1959	7.8
1960	15.5
1961	78.8
1962	4.1
1963	14.1
1964	19.7
1965	17.6
1966	14.2
1967	6.7
1968	38.6
1969	28.1
1970	14.8
1971	13.6
1972	23.2
1973	14.2
1974	35
1975	7.9
1976	54.6
1977	13.8
1978	3.6
1979	34.8
1980	19.9
1981	35
1982	30.5
1983	8.4
1984	13.6
1985	14
1986	26.2
1987	19
1988	24.2
1989	30.7
1990	29.7
1991	34
1992	25
1993	15

1994	17
1995	21.5
1996	33
1997	26
1998	20
1999	17.5
2000	83

The best fit for the 0246555 W Aggeneys (POL) Rainfall Station was the Log Pearson 3 distribution curve. Figure A-1 shows the Log Pearson 3 distribution curve.

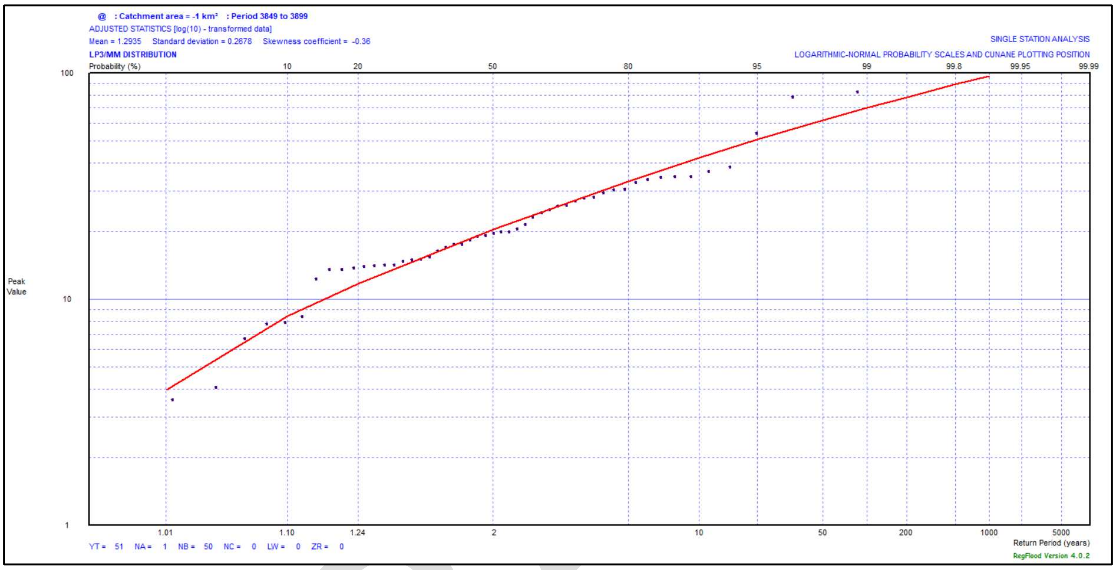

Figure A-1 Log Pearson 3 distribution curve for 0246555 W Aggeneys (POL) Rainfall Station

APPENDIX B

Geotechnical Investigation Interpretive Report

APPENDIX C

Soil Laboratory Test Results

APPENDIX D

Waste Classification Assessment

APPENDIX D1

Round 1 Sampling: September 2022

APPENDIX D2

Round 2 of Sampling: January 2023

APPENDIX D3

Round 1 Sampling: September 2022

APPENDIX D4

Round 2 Sampling: January 2023

APPENDIX E

Floodline Cross Sections and Hec-Ras Output

Figure E-1 Location and numbering of cross-sections for the non-perennial River

NON-PERENNIAL RIVER PROFILE OUTPUT TABLE:

River Station	Profile	Q Total	Minimum Channel Elevation	Water Surface Elevation	Critical Water Surface	Energy Gradient Elevation	Energy Gradient Slope	Velocity in Channel	Flow Area	Top Width	Froude No. in Channel
		(m³/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m²)	(m)	
28.16	1 in 50	1.84	954.88	954.95	954.93	954.96	0.004427	0.3	6.16	124.37	0.43
28.16	1 in 100	2.55	954.88	954.96	954.94	954.97	0.004296	0.33	7.67	129.55	0.44
124.36	1 in 50	1.84	954	954.11	954.11	954.15	0.021713	0.79	2.32	35.73	0.99
124.36	1 in 100	2.55	954	954.13	954.13	954.17	0.022035	0.86	2.95	40.47	1.02
209	1 in 50	1.84	953.35	953.46	953.42	953.47	0.004074	0.37	5	69.44	0.44
209	1 in 100	2.55	953.35	953.48	953.43	953.49	0.00405	0.39	6.52	82.69	0.44
375.74	1 in 50	1.84	952	952.08	952.08	952.11	0.023503	0.78	2.36	39.52	1.02
375.74	1 in 100	2.55	952	952.1	952.1	952.13	0.023656	0.85	2.99	44.01	1.05
432.37	1 in 50	1.84	951.5	951.6	951.57	951.6	0.004482	0.31	5.9	112.81	0.44
432.37	1 in 100	2.55	951.5	951.61	951.58	951.62	0.004354	0.35	7.37	118.42	0.44
478	1 in 50	1.84	951.09	951.18	951.18	951.2	0.023745	0.63	2.95	69.37	0.97
478	1 in 100	2.55	951.09	951.19	951.19	951.21	0.02503	0.7	3.64	75.14	1.02
535	1 in 50	1.84	950.6	950.7	950.68	950.71	0.003783	0.27	6.8	141.91	0.39
535	1 in 100	2.55	950.6	950.72	950.68	950.72	0.003627	0.3	8.54	149.28	0.4
630.42	1 in 50	1.84	949.9	949.99	949.98	950	0.020508	0.57	3.25	79.33	0.89
630.42	1 in 100	2.55	949.9	949.99	949.99	950.02	0.022589	0.65	3.9	82.86	0.96

River Station	Profile	Q Total (m³/s)	Minimum Channel Elevation (m)	Water Surface Elevation (m)	Critical Water Surface (m)	Energy Gradient Elevation (m)	Energy Gradient Slope (m/m)	Velocity in Channel (m/s)	Flow Area (m²)	Top Width (m)	Froude No. in Channel
683.24	1 in 50	1.84	949.46	949.56	949.53	949.57	0.00436	0.31	6	115.27	0.43
683.24	1 in 100	2.55	949.46	949.56	949.54	949.57	0.006966	0.4	6.36	116.64	0.55
840	1 in 50	1.84	948.03	948.12	948.12	948.14	0.029558	0.63	2.94	81.08	1.05
840	1 in 100	2.55	948.03	948.14		948.15	0.012261	0.51	5.04	99.32	0.72
1214.96	1 in 50	1.84	944.52	944.76		944.79	0.005809	0.69	2.66	18.68	0.59
1214.96	1 in 100	2.55	944.52	944.79	944.74	944.83	0.006723	0.79	3.24	21.02	0.64
1310	1 in 50	1.84	943.99	944.08	944.06	944.09	0.009562	0.41	4.5	101.16	0.62
1310	1 in 100	2.55	943.99	944.09		944.1	0.00851	0.44	5.8	107.7	0.6
1369.27	1 in 50	1.84	943.46	943.55	943.54	943.56	0.008275	0.35	5.2	130.05	0.57
1369.27	1 in 100	2.55	943.46	943.56	943.55	943.57	0.009394	0.41	6.16	134.61	0.62
1433	1 in 50	1.84	942.74	942.83	942.82	942.84	0.016269	0.56	3.28	68.52	0.82
1433	1 in 100	2.55	942.74	942.85	942.83	942.86	0.013365	0.54	4.72	90.25	0.75
1571	1 in 50	1.84	941.5	941.64	941.61	941.65	0.005373	0.37	4.97	84.12	0.49
1571	1 in 100	2.55	941.5	941.65	941.62	941.66	0.006108	0.43	5.93	88.76	0.53
1699	1 in 50	1.84	940.43	940.51	940.5	940.53	0.016674	0.62	2.95	53.54	0.85
1699	1 in 100	2.55	940.43	940.53	940.52	940.55	0.013325	0.6	4.21	67.61	0.77
1790	1 in 50	1.84	939.47	939.56		939.57	0.007257	0.38	4.81	97.12	0.55
1790	1 in 100	2.55	939.47	939.57		939.58	0.008669	0.46	5.58	99.1	0.61

River Station	Profile	Q Total (m^3/s)	Minimum Channel Elevation (m)	Water Surface Elevation (m)	Critical Water Surface (m)	Energy Gradient Elevation (m)	Energy Gradient Slope (m/m)	Velocity in Channel (m/s)	Flow Area (m^2)	Top Width (m)	Froude No. in Channel
1877.9	1 in 50	1.84	938.5	938.6	938.59	938.63	0.017059	0.72	2.55	37.63	0.89
1877.9	1 in 100	2.55	938.5	938.62	938.61	938.65	0.013034	0.72	3.54	42.97	0.8
1985.49	1 in 50	1.84	937.48	937.57		937.58	0.006215	0.44	4.18	61.28	0.54
1985.49	1 in 100	2.55	937.48	937.58		937.6	0.007559	0.53	4.84	64.01	0.6
2077.61	1 in 50	1.84	936.59	936.7	936.7	936.72	0.015814	0.57	3.24	64.92	0.81
2077.61	1 in 100	2.55	936.59	936.72		936.74	0.01181	0.56	4.6	87.25	0.73
2203	1 in 50	1.84	935.48	935.56		935.56	0.005928	0.36	5.17	98.05	0.5
2203	1 in 100	2.55	935.48	935.57		935.58	0.007359	0.44	5.94	100.59	0.57
2298	1 in 50	1.84	934.57	934.66		934.67	0.017055	0.5	3.8	105.25	0.81
2298	1 in 100	2.55	934.57	934.68	934.67	934.69	0.012269	0.48	5.46	122.57	0.71
2497.5	1 in 50	1.84	932.67	932.75	932.73	932.76	0.006166	0.38	4.8	85.39	0.52
2497.5	1 in 100	2.55	932.67	932.76	932.74	932.77	0.007804	0.46	5.52	88.87	0.59
2597.95	1 in 50	1.84	931.58	931.64	931.64	931.66	0.024436	0.59	3.13	82.76	0.96
2597.95	1 in 100	2.55	931.58	931.66	931.65	931.67	0.016322	0.58	4.42	88.71	0.82
2732	1 in 50	1.84	930.1	930.25	930.21	930.26	0.005763	0.5	3.68	41.89	0.54
2732	1 in 100	2.55	930.1	930.26	930.23	930.28	0.007216	0.59	4.3	45.1	0.61
2799.00	1 in 50	1.84	929.48	929.54	929.54	929.55	0.025251	0.47	3.92	148.57	0.92

River Station	Profile	Q Total	Minimum Channel Elevation	Water Surface Elevation	Critical Water Surface	Energy Gradient Elevation	Energy Gradient Slope	Velocity in Channel	Flow Area	Top Width	Froude No. in Channel
		(m^3/s)	(m)	(m)	(m)	(m)	(m/m)	(m/s)	(m^2)	(m)	
2799.00	1 in 100	2.55	929.48	929.55	929.55	929.56	0.017176	0.47	5.47	157.44	0.8
2870.60	1 in 50	1.84	928.68	928.74	928.74	928.76	0.023041	0.47	3.95	141.41	0.89
2870.60	1 in 100	2.55	928.68	928.77	928.75	928.77	0.007678	0.34	7.51	189.72	0.54
3011.00	1 in 50	1.84	927.29	927.38	927.36	927.39	0.005315	0.38	4.83	77.88	0.49
3011.00	1 in 100	2.55	927.29	927.38	927.36	927.4	0.012944	0.57	4.44	75.49	0.76
3087.02	1 in 50	1.84	926.48	926.53	926.53	926.55	0.035607	0.52	3.55	150.02	1.08
3087.02	1 in 100	2.55	926.48	926.55	926.54	926.56	0.009316	0.37	6.82	172.56	0.6

APPENDIX F

Seepage and Stability Sections

APPENDIX G

Technical Specification

APPENDIX H

Construction Quality Assurance

APPENDIX I

Drawings

APPENDIX J

Bill of Quantities

APPENDIX K

Pump and Pipeline Design Report

Rev B
15 April 2024

APPENDIX L

Hydrogeology Study

APPENDIX M

DWS Checklist

APPENDIX N

Operation Manual