



VOLSPRUIT MINE RESIDUE AND STORMWATER MANAGEMENT DETAIL DESIGN REPORT

SYLVANIA RESOURCES (PTY) LTD



ATTENTION: Mr Albert Jordaan

EMAIL: albert@sylvania.co.za

PROJECT NUMBER: 19-918-DDR (VOLSPRUIT MRD AND SWM DESIGN)

PROJECT: Volspruit Mining Project

DOCUMENT NUMBER: 19-918-DDR-RPT-001

VERSION: 0.0

AUTHOR: Natasjha de Beer

EMAIL: natasjhadb@ecoe.co.za

DATE: 2026/10/09



**DETAIL DESIGN REPORT
(WATER USE LICENCE APPLICATION)**

Document Review & Approval:

VERSION NO	VERSION HISTORY				REVIEW APPROVAL
AA – draft	2026/10/02	Natasjha de Beer		First draft for review/comments	Approved
BB – draft	2026/10/09	Waldo Kruger	 <small>Waldo Kruger Reviewed 19-918-RR-BB 2026-10-09 15:05 UTC-08:00 ID WK-20261009-01 Verify BDT-KFOX</small>	Technical Review	Approved
APPROVED FOR DISTRIBUTION					
0.0	2026/10/09	Natasjha de Beer, PrEng 202001336		Final Report	Approved

The final issue of this report (Version 0.0) and the associated drawings are certified and signed off by the Professional Engineer named above, who is registered with the Engineering Council of South Africa (ECSA) in terms of the Engineering Profession Act, 2000 (Act 46 of 2000). The certification is made in terms of regulation 3(2)(a) of GN R636.

Quality Control By:

NATURE OF SIGNOFF	RESPONSIBLE PERSON	ROLE / RESPONSIBILITY	QUALIFICATION / ENTITY
Author	Natasjha de Beer	Civil Engineer	BEng (Civil), PrEng 202001336
Technical Reviewer	Waldo Kruger	Principal Engineer – Tailings and Geotechnical	BEng (Civil), BEng (Hons) Geotechnical CPEng / PrEng 20170234

Legal Notice:

This report was prepared by Eco Elementum Engineering (Pty) Ltd (EcoE) for the account of Sylvania Resources (Pty) Ltd (SA). Report content reflects EcoE's best judgement based on the best information available at the time of preparation. Any use a third party makes of this report, or any reliance on or decisions made based on it is the responsibility of such third parties. Any reproductions of this report are uncontrolled and might not be the most recent revision.

During construction site conditions, or site development, may possibly dictate that amendments or adaptations be made to certain details of the design. Strict compliance with the project specifications is also essential. EcoE will therefore not accept liability for any losses, damages or consequential losses, suffered by any party resulting from the use or implementation of the contents of this report, if executed or amended without fulltime supervision by EcoE.

EXECUTIVE SUMMARY

Sylvania Resources (Pty) Ltd (SA) appointed Eco Elementum Engineering (Pty) Ltd to design the mine residue facilities and the stormwater management infrastructure for the Volspruit Mine near Mokopane in Limpopo. This report consolidates the detailed design of the TSF, the WRD, the RoM pad, the RWD and irrigation dam, the WRD, RoM and Plant PCDs, the silt traps, the clean and dirty water systems and the flood diversion berm on the Nyl River, in support of the water use licence application.

The waste classification update (Annexure A) assesses the wastes against GN R635 as amended in 2024. The tailings, the RoM ore and the water in the dams are Type 3 and need a Class C barrier, and the waste rock is Type 4 and is placed on a Class D foundation (Chapter 5).

The TSF is lined with a Class C equivalent inverted barrier: a 1.5 mm double textured HDPE geomembrane on a prepared receiving face, with the fine tailings slimed directly over it, a leachate drainage network above and a sub-surface drainage network below. The equivalence is motivated under regulation 3(2)(d) of GN R636 (Chapter 10 and Annexure H). The RWD, the irrigation dam and the WRD and RoM PCDs are lined with a 2 mm HDPE geomembrane under a soilcrete cover, and the Plant PCD, the silt traps and the RoM pad with water retaining reinforced concrete. Every barrier has a monitored sub-surface drainage system, and acceptable and action leakage rates are set for each facility (section 10.9).

Clean and dirty water are kept separate in accordance with GN 704. The channels, culverts and silt traps are sized for the 1:50-year flood peaks, and the dam capacities are based on the water balance, which is to be re-run on the updated production plan before construction to confirm the operating levels and storm storage of each dam (Chapter 6). The flood diversion berm keeps the 1:100-year flood of the Nyl River out of the North Pit (Chapter 18).

The stability of the TSF, the WRD, the dams and the berm is assessed against the minimum factors of safety in Chapter 9, with the analyses in Annexure G. A tailings dam breach analysis confirms the zone of influence and the safety classification of the TSF (Chapters 8 and 22) and forms the basis of the emergency response plan.

Construction is controlled by the CQA plan in Annexure I, with an independent electrical leak location survey of every geomembrane. The investigations, tests and approvals listed in Table 23-1 are completed before construction starts, and the design is reviewed against their results. Within the scope of this report, the design addresses the barrier, water management and safety requirements of GN R636 as amended, GN 704 and SANS 10286.

CONTENTS

EXECUTIVE SUMMARY	3
PART A — PROJECT AND SITE INFORMATION.....	14
1 KEY PROJECT INFORMATION	15
2 INTRODUCTION	17
2.1 Background	17
2.2 Scope of Work.....	17
2.3 Design Stage and Basis of Certification.....	18
2.4 Reliance Documents and Superseded Issues	18
2.5 Legislation and Guidance Documents	19
3 DESCRIPTION OF THE CURRENT ENVIRONMENT.....	22
3.1 Locality and Physical Characteristics.....	22
3.2 General Layout and Site Development Plan.....	22
3.3 Topography and Drainage: Quaternary Catchment A61E	23
3.4 Underground and Adjacent Mining	24
3.5 Geology.....	25
3.5.1 Regional Geology.....	25
3.5.2 Site Geology	25
3.6 Groundwater	25
3.7 Seismicity	26
3.8 Regional Climate.....	26
3.9 Rainfall, Evaporation and Storm Rainfall Depths.....	26
4 GEOTECHNICAL INVESTIGATION	29
4.1 Investigation Programme	29
4.2 Soil Profile and Founding Conditions.....	32
4.2.1 TSF and RWD Area	33
4.2.2 New WRD Area	34
4.2.3 RoM Area	35
4.2.4 Plant PCD Area	36
4.2.5 Groundwater	36
4.3 Laboratory Test Results	36
4.4 Problem Soils and Ground Improvement.....	38
4.5 Excavation Conditions.....	39
4.6 Material Suitability	39
4.7 Implications for the Design.....	39
5 MATERIALS WASTE CLASSIFICATION	41
5.1 Sampling and Analysis Methodology	41
5.2 Tailings.....	41
5.3 Waste Rock	42
5.4 Run-of-Mine Ore	42

5.5	Wastewater: Irrigation Dam	42
5.6	Waste Type Determination	43
5.7	Barrier Class Requirement by Facility	43
6	WATER BALANCE	45
6.1	Water Balance Model	45
6.2	Model Inputs	45
6.3	Water Storage Components	47
6.4	Results	47
6.5	Implications for the Design	49
7	HYDROLOGY AND SURFACE WATER ASSESSMENT	50
7.1	Catchment Characteristics	50
7.2	Flood Peaks and Volumes	51
7.3	Flood Lines	53
7.4	Clean and Dirty Water Catchment Delineation	54
8	SAFETY CLASSIFICATION	57
8.1	Zone of Influence	57
8.2	Classification Information	57
8.3	Classification per SANS 10286	59
8.4	Classification per the Global Industry Standard on Tailings Management Consequence Classification Matrix	60
	PART B — DESIGN BASIS	61
9	DESIGN CRITERIA	62
9.1	Design Criteria by Facility	62
9.2	Freeboard	63
9.3	Embankment Stability and Earthquake Criteria	63
9.4	Available Mineral Resources, Life of Mine and Polluting Period	65
9.5	Design Parameters and Tonnages	65
10	BARRIER DESIGN FRAMEWORK	66
10.1	Applicable Legislation	66
10.2	GN R636 Prescribed Barrier Systems	67
10.3	Barrier Class per Facility	68
10.4	Material Specifications	69
10.5	Geotextile Specification	70
10.6	Liner Longevity and Service Life of Geosynthetic Components	70
10.7	Electrical Leak Location Survey Specification	71
10.8	Inverted Barrier System, Basis and Motivation	71
10.9	Total Solute Seepage	72
10.10	Filter Compatibility	76
	PART C — FACILITY DESIGN	80
11	TAILINGS STORAGE FACILITY	81

11.1	Overview and Design Objectives.....	81
11.2	Dam Safety Registration and Classification	81
11.3	Layout, Capacity and Compartments	82
11.4	Staged Development and Rate of Rise	85
11.5	Foundation Preparation	85
11.6	Barrier System	86
11.7	Barrier Performance Verification	88
11.8	Starter, Toe and Heel Walls.....	88
11.9	Sub-surface and Internal Drainage.....	89
11.10	Perimeter Solution Trenches	92
11.11	Penstock and Decant System.....	92
11.12	Penstock Hydraulic Capacity	94
11.13	Access, Fencing and Ancillary Features	95
11.14	Seepage and Slope Stability.....	95
11.14.1	Ground Model and Material Parameters.....	95
11.14.2	Analysis Method.....	97
11.15	Operation and Deposition Strategy	97
11.16	Instrumentation and Monitoring	98
11.17	Closure and Cover.....	98
12	WASTE ROCK DUMP	100
12.1	Overview	100
12.2	Layout, Capacity and Development.....	100
12.3	Foundation Preparation	100
12.4	Barrier System	101
12.5	Barrier Performance Verification	102
12.6	Drainage and Solution Collection	102
12.7	Slope Stability	102
12.7.1	Ground Model.....	102
12.7.2	Waste Rock Strength	103
12.7.3	Loading Conditions.....	103
12.8	Closure and Cover	103
13	RUN-OF-MINE PAD	104
13.1	Overview	104
13.2	Layout and Layerworks.....	104
13.3	Barrier System	105
13.4	Drainage and Solution Collection	105
13.5	Operation and Closure.....	106
14	RETURN WATER, POLLUTION CONTROL AND IRRIGATION DAMS	107
14.1	Overview	107
14.2	Dam Safety Registration and Classification	107

14.3	Embankment Design	109
14.4	HDPE Barrier System	109
14.5	Plant PCD Concrete Lining	111
14.6	Veneer Stability	113
14.7	Spillways	115
14.8	Stability	115
14.9	Operation and Monitoring	117
15	SILT TRAPS	118
15.1	Overview	118
15.2	Silt Trap Design	118
15.3	Cleaning and Maintenance	120
16	DIRTY WATER COLLECTION SYSTEM	121
16.1	Overview	121
16.2	Channel Design and Linings	122
16.3	Culvert Crossings	123
17	CLEAN WATER DIVERSION SYSTEM	125
17.1	Overview	125
17.2	Channel Design	125
17.3	Erosion Protection	126
17.4	Crossings and Discharge Structures	126
18	FLOOD DIVERSION BERM	128
18.1	Overview	128
18.2	Geometry and Freeboard	128
18.3	Erosion Protection	129
18.4	Stability	129
PART D — IMPLEMENTATION AND ASSURANCE		130
19	CONSTRUCTION MATERIALS AND SPECIFICATIONS	131
19.1	Material Sources and Availability	131
19.2	Earthworks and Layerworks	132
19.3	Geosynthetics	132
19.4	Concrete and Pipework	132
20	CONSTRUCTION QUALITY ASSURANCE	133
21	OPERATION, MAINTENANCE AND MONITORING REQUIREMENTS	134
21.1	Responsibilities	134
21.2	Tailings Storage Facility	134
21.3	Dams	135
21.4	Stormwater Infrastructure	135
21.5	Waste Rock Dump and Run-of-Mine Pad	136
21.6	Water Quality and Groundwater Monitoring	136
21.7	Reporting and Review	136

22	DAM BREAK ANALYSIS	137
22.1	Basis of the Analysis.....	137
22.2	Analysis Limitations	138
22.3	Inundation Zones	138
22.4	Flow Velocities and Arrival Times.....	138
22.5	Sensitivity to the Outflow Volume	138
22.6	Implications for the Design and Operation	138
23	CONCLUSION	140
24	REFERENCES.....	142
	ANNEXURES.....	146
	ANNEXURE A: MATERIALS WASTE CLASSIFICATION.....	147
	ANNEXURE B: WATER BALANCE REPORT	148
	ANNEXURE C: GEOTECHNICAL INVESTIGATION REPORT	149
	ANNEXURE D: HYDROLOGY AND CALCULATIONS	150
	ANNEXURE E: FLOOD LINE STUDY	151
	ANNEXURE F: GEOHYDROLOGICAL STUDY	152
	ANNEXURE G: SEEPAGE AND SLOPE STABILITY ANALYSIS RESULTS	153
	ANNEXURE H: ADDITIONAL INFORMATION FOR INVERTED BARRIER SYSTEMS	154
	ANNEXURE I: CONSTRUCTION QUALITY ASSURANCE PLAN	155
	ANNEXURE J: DETAIL DESIGN DRAWINGS	156
	ANNEXURE K: TAILINGS DAM BREACH ANALYSIS REPORT	157

LIST OF TABLES

Table 1-1: Information Pertaining to Site	15
Table 1-2: Project Information.....	15
Table 1-3: DWS Checklist Cross-Reference.....	16
Table 2-1: Reliance Documents.....	18
Table 2-2: Superseded Issues	19
Table 3-1: Rainfall and S-Pan Evaporation Data	27
Table 3-2: Adopted Design Rainfall	28
Table 4-1: Investigation Coverage by Area.....	29
Table 4-2: Generalised Ground Profile	32
Table 4-3: Summary of Test Profiles, TSF and RWD Area	33
Table 4-4: Summary of Test Profiles, New WRD Area	35
Table 4-5: Summary of Test Profiles, RoM Area	35
Table 4-6: Summary of Laboratory Results by Layer	37
Table 4-7: Double Oedometer Results	38
Table 5-1: Waste Classification Samples	41
Table 5-2: Modelled North Pit Sump Water Quality Against the Leachable Concentration Thresholds	42

Table 5-3: Waste Type per Material.....	43
Table 5-4: Barrier Class Required per Facility	44
Table 6-1: Water Balance Model Inputs.....	46
Table 6-2: Water Balance Storages and Design Capacities.....	47
Table 6-3: Average Monthly Water Sources and Losses	48
Table 6-4: Scaled Check of the Production Rate.....	48
Table 7-1: Clean Water Catchment Characteristics and Flood Peaks	52
Table 7-2: Dirty Water Catchment Characteristics and Flood Peaks	53
Table 7-3: Nyl River Design Flood Peaks	53
Table 8-1: Safety Classification Information	57
Table 8-2: SANS 10286 Safety Classification Criteria	59
Table 8-3: Safety Classification Using the Zone of Influence	59
Table 8-4: GISTM Consequence Classification Matrix for the TSF	60
Table 9-1: Design Criteria by Facility	62
Table 9-2: TSF Freeboard Criteria	63
Table 9-3: TSF Loading Conditions and Minimum Factors of Safety	63
Table 9-4: Dam and Flood Diversion Berm Loading Conditions and Minimum Factors of Safety (CDA, 2013).....	64
Table 9-5: Life of Mine and Service Life Considerations	65
Table 9-6: Design Parameters and Tonnages	65
Table 10-1: GN R636 Regulation 3(2) Compliance	66
Table 10-2: Barrier Class and Barrier System per Facility.....	68
Table 10-3: Barrier Material Specifications	69
Table 10-4: Geotextile Applications	70
Table 10-5: Service Life Considerations	71
Table 10-6: TSF Total Solute Seepage and Class C Comparison	74
Table 10-7: HDPE Lined Dam Total Solute Seepage and Class C Comparison	75
Table 10-8: Filter Compatibility Interfaces	77
Table 10-9: Interim Filter Compatibility Checks	78
Table 11-1: TSF Capacity and Development Parameters	83
Table 11-2: Confinement Wall Geometry.....	89
Table 11-3: TSF Drainage System Components	90
Table 11-4: Penstock Hydraulic Capacity per Cell	94
Table 11-5: Material Parameters Adopted in the TSF Stability Analyses	96
Table 11-6: Interface Shear Strength of the Liner System (Howell and Kirsten, 2016).....	97
Table 11-7: TSF Closure Cover Concept.....	99
Table 12-1: WRD Geometry.....	100
Table 12-2: WRD Loading Conditions and Minimum Factors of Safety	103
Table 13-1: RoM Pad Geometry	104
Table 13-2: RoM Pad Layerworks	104

Table 14-1: Dam Safety Information	108
Table 14-2: HDPE Barrier System Build-up	110
Table 14-3: Plant PCD Concrete Lining Build-up	112
Table 14-4: Veneer Stability Parameters	113
Table 14-5: Veneer Stability Calculations for the HDPE Lined Dams	114
Table 14-6: Material Parameters Adopted in the Dam and Flood Diversion Berm Stability Analyses	116
Table 15-1: Design of Silt Traps	119
Table 15-2: Silt Trap Inflows and Sizing	120
Table 16-1: Design of Dirty Water Channels	123
Table 16-2: Design of Dirty Water Culvert Crossings	123
Table 17-1: Design of Clean Water Channels	125
Table 17-2: Design of Clean Water Channel Outlets	126
Table 17-3: Design of Culvert Crossings	127
Table 18-1: Design of the Flood Diversion Berm	128
Table 19-1: Concrete Specification	132
Table 22-1: Tailings Breach Outflow Volumes	138
Table 23-1: Pre-construction Programme	140

LIST OF FIGURES

Figure 3-1: Locality Map	22
Figure 3-2: Site Layout Including Ecological Importance and Topsoil Stockpiles	23
Figure 3-3: Quaternary Catchment A61E of the Limpopo Water Management Area	24
Figure 3-4: Geology in Relation to the Volspruit Mine	25
Figure 4-1: Test Positions, TSF and RWD Area (Peregrine Geoconsultants, 2021)	30
Figure 4-2: Test Positions, New WRD Area (Peregrine Geoconsultants, 2021)	31
Figure 4-3: Test Positions, RoM Area (Peregrine Geoconsultants, 2021)	31
Figure 4-4: Test Positions, Plant Area (Peregrine Geoconsultants, 2021)	32
Figure 6-1: Water Balance Flow Diagram	45
Figure 7-1: Site Catchment Delineation	51
Figure 7-2: Clean Water Runoff Flow Diagram	55
Figure 7-3: Dirty Water Runoff Flow Diagram	56
Figure 8-1: View of the Zone of Influence	57
Figure 10-1: GN R636 Class C Barrier Requirements (Extract from GN R636, as amended by GN 5523)	67
Figure 10-2: GN R636 Class D Barrier Requirements (Extract from GN R636, as amended by GN 5523)	68
Figure 10-3: Interim Filter Bands	79
Figure 11-1: TSF General Layout	82
Figure 11-2: Stage Capacity Curve	84
Figure 11-3: Inverted Barrier System Layerworks	87

Figure 11-4: Typical Liner Penetration Detail.....	87
Figure 11-5: Typical Starter Wall Section	89
Figure 11-6: Sub-Surface Drainage Layout	91
Figure 11-7: Basin Leachate Drainage Network.....	91
Figure 11-8: Final Penstock Intake Structures.....	93
Figure 13-1: RoM Pad Layerworks	105
Figure 14-1: Typical Dam Lining Detail.....	111
Figure 18-1: Flood Diversion Berm Typical Sections.....	129



ABBREVIATIONS

ABBREVIATION	TERM
AASHTO	American Association of State Highway and Transportation Officials
ANCOLD	Australian National Committee on Large Dams
ASTM	American Society for Testing and Materials
BS EN	British Standard European Norm
CBR	California Bearing Ratio
CCL	Compacted Clay Liner
CDA	Canadian Dam Association
CH	Channel (drawing and channel designation)
COLTO	Committee of Land Transport Officials
CQA	Construction Quality Assurance
CW	Clean Water (channel designation)
DW	Dirty Water (channel designation)
DWS	Department of Water and Sanitation
EcoE	Eco Elementum (Pty) Ltd
ECSA	Engineering Council of South Africa
EPDM	Ethylene Propylene Diene Monomer
ERPP	Emergency Response and Preparedness Plan
FSL	Full Supply Level
GCL	Geosynthetic Clay Liner
GEM-GPT	Groundwater Environmental Management GPT (Pty) Ltd
GISTM	Global Industry Standard on Tailings Management
GN	Government Notice
GRI	Geosynthetic Research Institute
GSI	Geological Strength Index
HDPE	High Density Polyethylene
HEC-RAS	Hydrologic Engineering Centre River Analysis System
ISO	International Organization for Standardization
KHH	Knight Hall Hendry
LCT	Leachable Concentration Threshold
LL	Liquid Limit
mamsl	Metres above mean sea level
MDD	Maximum Dry Density
MRD	Mine Residue Deposit
OMC	Optimum Moisture Content
PCD	Pollution Control Dam

ABBREVIATION	TERM
PGM	Platinum Group Metal
PI	Plasticity Index
PN	Nominal Pressure (pipe class)
PVC	Polyvinyl Chloride
RoM	Run-of-Mine
RWD	Return Water Dam
SAICE	South African Institution of Civil Engineering
SANAS	South African National Accreditation System
SANCOLD	South African National Committee on Large Dams
SANRAL	South African National Roads Agency SOC Limited
SANS	South African National Standard
SAWIC	South African Waste Information Centre
SAWS	South African Weather Service
SCS	Soil Conservation Service (United States)
SN	Nominal Ring Stiffness (pipe class)
SPT	Standard Penetration Test
SWM	Stormwater Management
SWMP	Stormwater Management Plan
TCT	Total Concentration Threshold
TDBA	Tailings Dam Breach Analysis
TLB	Tractor-Loader-Backhoe
TSF	Tailings Storage Facility
UCS	Unconfined Compressive Strength
USDA	United States Department of Agriculture
UV	Ultraviolet
WMA	Water Management Area
WRC	Water Research Commission
WRD	Waste Rock Dump

PART A — PROJECT AND SITE INFORMATION



1 KEY PROJECT INFORMATION

Table 1-1: Information Pertaining to Site

LOCALITY	MOKOPANE; LIMPOPO PROVINCE
Mining Method	Open pit
Product	Chrome and Platinum Group Metals (PGMs)
River Catchment	Nyl River, located in the Limpopo Water Management Area (WMA) and more specifically within the A61E quaternary catchment. The site lies on the eastern bank of the Nyl River.
Site Midpoint Coordinates	Latitude: 24°20'57.54" S Longitude: 28°57'29.63" E
Farm Description	Volspruit 326 KR and Zoetveld 294 KR, Mokopane, Limpopo Province
Waste Classification	The waste types adopted for the design are Type 4 for the WRD and Type 3 for the TSF and the RoM pad. The WRD, the irrigation dam and the PCDs are treated as Type 3, and each is lined with a Class C equivalent barrier (Chapters 5 and 10).
Description of Waste Stream	Tailings Storage Facility (TSF) – leachate drainage collected from the basin and toe line of the facility, with supernatant water decanted from the pool by a penstock system. Waste Rock Dump (WRD) – surface runoff and seepage collected by perimeter paddocks and toe drainage. Run-of-Mine (RoM) Pad – surface runoff and seepage collected by the pad drainage system. All dirty water reports to the respective Pollution Control Dams (PCDs).

Table 1-2: Project Information

PROJECT	VOLSPRUIT MINING PROJECT – MINE RESIDUE AND STORMWATER MANAGEMENT
Client Trading Name	Sylvania Resources (Pty) Ltd (SA)
Client Contact Person	Albert Jordaan
Client Email	albert@sylvania.co.za
Client Telephone	082 307 7364
EcoE Contact Person	Chris Fourie for Eco Elementum Engineering (Pty) Ltd
EcoE Postal Address	Eco Elementum: Block@Nature, Block B, 472 Botterklapper Street, Die Wilgers, Pretoria East
EcoE Postal Code	0184
EcoE Telephone	012 807 0383
EcoE Email	chrisf@ecoe.co.za / info@ecoe.co.za
Pr. Eng	Natasjha de Beer (Professional Engineer) PrEng Number 202001336
Qualifications & Relevant Experience	Natasjha de Beer has 12 years' experience in water resource, civil infrastructure and mine waste projects.

The report is laid out so that each item of the DWS Compliance and Readiness Checklist for the technical report on civil engineering (February 2023) can be found directly. Table 1-3 gives the section or table in which each checklist item is addressed. The completed checklist is provided with the submission and does not form part of this report.

Table 1-3: DWS Checklist Cross-Reference

CHECKLIST ITEM	REFERENCE	REPORT SECTION OR TABLE
Applicant, representative and certifying engineer with ECSA registration	Checklist 1(a) to (c)	Table 1-2 and the document control page
Title, date and reference number of the technical report	Checklist 1(d)	Cover page and document control page
Quaternary catchment	Checklist 1(f)	Table 1-1 and section 3.3
Site topography, geotechnics, geology and geohydrology	Checklist 2	Chapter 3 and Chapter 4
Waste risk assessment (GN R634 and GN R635)	21(g)	Chapter 5 and Table 5-3
Description of the waste streams	21(g)	Table 1-1
Storage of water: capacity, wall height, spillway and outlet works	21(b)	Table 14-1 and section 14.7
Crossings and diversion works	21(c) and (i)	Chapter 16, Chapter 17 and Chapter 18
Design report and drawings certified by a professional civil engineer	GN R636 3(2)(a)	Document control page and section 2.3
Service life: polluting and operating periods, leachate temperature, tensile strain, drainage and liner service life	GN R636 3(2)(b)	Table 9-5 and Table 10-5
Total solute seepage: liner area, wrinkles, leakage rates and total leakage	GN R636 3(2)(c)	Section 10.9, Table 10-6 and Table 10-7
Alternative elements of proven equivalent performance	GN R636 3(2)(d)	Section 10.8, section 11.6, section 14.4
Drainage layers and atmospheric pressure in the drains	GN R636 3(2)(e)	Section 11.9 and Chapter 14
Barrier systems on slopes steeper than 1V:4H	GN R636 3(2)(f)	Section 14.4 and section 14.6
Construction quality assurance, interface shear testing and electrical leak location survey	GN R636 3(2)(g)	Chapter 20, section 10.7 and Annexure I
Standards and specifications	GN R636 3(2)(h)	Table 10-3, section 10.5 and Chapter 19
Compatibility of the liner materials with the waste stream	GN R636 3(2)(i)	Section 10.6
Factors of safety for stability	Checklist	Table 9-3, Table 9-4, Annexure G, section 14.8 and section 18.4
Gas management	Checklist	Not applicable, as the waste contains no putrescible material

2 INTRODUCTION

2.1 Background

Eco Elementum (Pty) Ltd (EcoE) is the design engineer for the mine residue facilities and the surface water management infrastructure at the proposed Volspruit Mine near Mokopane in the Limpopo Province. EcoE prepared the original design, issued in December 2022, and is acting for Sylvania Resources (Pty) Ltd in updating that design.

The facilities covered by this report are:

- Tailings Storage Facility (TSF);
- Waste Rock Dump (WRD);
- Run-of-Mine (RoM) pad;
- Return Water Dam (RWD) serving the TSF;
- WRD Pollution Control Dam (WRD PCD);
- RoM Pollution Control Dam (RoM PCD);
- Plant Pollution Control Dam (Plant PCD);
- Irrigation Dam;
- Clean and dirty water conveyance systems; and
- Flood Diversion Berm.

Volspruit is a proposed open pit platinum group metals (PGM) mine. The design supports the water use licence application and the associated waste management licence application.

EcoE issued the original design in December 2022 as two separate documents, the Volspruit MRD Design Report and the Volspruit Stormwater Management Plan, both Version 0.0 and both dated 2 December 2022. Feedback from the Department of Mineral Resources and Petroleum during the waste management licensing process required these reports to be updated.

The mine residue and water management designs, previously issued as two separate documents, are consolidated into this single integrated design report. The design criteria, the barrier design framework and the hydrological criteria are stated once and applied consistently across all facilities.

This report supersedes both Version 0.0 documents.

2.2 Scope of Work

The scope of work covered by this report is as follows:

- collate and review the existing project information, including the previous design issues, the geotechnical investigation and the specialist studies commissioned for the project;
- describe the site in terms of topography, hydrology, geology, geohydrology and seismicity;

- classify the waste streams and determine the barrier class required for each facility;
- determine the safety classification of the facilities;
- establish the design criteria and the design basis applied to all facilities;
- the detailed design of the TSF, the WRD and the RoM pad, including the barrier systems, drainage, containment walls and decant arrangements;
- the detailed design of the water management infrastructure, including the dams, silt traps, conveyance systems and flood diversion berm listed in section 2.1;
- justification of design barrier performance by filter compatibility, geomembrane strain and solute transport assessment;
- specify the construction materials, the quality assurance requirements and the operating and monitoring requirements; and
- compile this detailed design report in support of the water use licence and waste management licence applications.

2.3 Design Stage and Basis of Certification

This report presents the detailed design of the mine residue and water management infrastructure. It is issued as a final design at Engineering Council of South Africa (ECSA) stage 3 and is signed off by the Professional Engineer named in the document control page of this report. The drawings listed in the drawing register and attached as Annexure J are signed.

Regulation 3(2)(a) of Government Notice (GN) R636 requires that design reports and drawings be certified by a registered professional civil engineer prior to submission to the competent authority.

2.4 Reliance Documents and Superseded Issues

The documents listed in Table 2-1 have been relied upon in the preparation of this design. Where a document is attached to this report, the annexure is identified in the table of annexures.

Table 2-1: Reliance Documents

DOCUMENT	AUTHOR	REFERENCE	DATE
Geotechnical Investigation Report	Peregrine Geoconsultants	PG19-003/673 Rev 02	30 June 2021
Sylvania Volspruit Waste Classification, 2026 Update	Eco Elementum (Pty) Ltd	19-918, Version 1.0	9 October 2026
Flood Line Report: 50- and 100-year flood lines for the Nyl River at the Sylvania Resources Volspruit Mine	Garfield Krige Pr.Sci.Nat. of African Environmental Development. Flood line modelling techniques verified and certified by Peter Bond Pr.Eng.	AED0151, Revision 00.a	22 July 2010
Computerised Water Balance Simulation Model	Water Hunters	Technical Report 1023 (SX-TR0124), version 1.2 final	May 2024

DOCUMENT	AUTHOR	REFERENCE	DATE
Groundwater Specialist Study Update	Groundwater Environmental Management GPT (Pty) Ltd (GEM-GPT)	SYVOL-26-14406,	June 2026
Environmental Impact Assessment Report (Part A) and Environmental Management Programme (Part B)	EcoPartners	Refer to the environmental authorisation file	29 September 2024
Construction Quality Assurance Plan	K R Legge	Final	5 October 2026
Tailings Dam Breach Analysis Report: Volspruit TSF	Eco Elementum (Pty) Ltd	19-918-WMF (Volspruit TSF Tailings Dam Breach Analysis), Version 0.0	30 September 2026
Slope Stability Assessment: PCDs, RWD, Irrigation Dam and Flood Diversion Berm	Eco Elementum (Pty) Ltd	19-918 DDR-RPT-001, Version 0.0	9 October 2026
Simulation of Water Quality from North Pit, Volspruit Mine, Updated Report	Mills Water	Final	28 September 2026
Sylvania Platinum Volspruit Mine Water Use Licence Application: Review of Engineering Documentation	K R Legge	Independent review	25 June 2026

The documents listed in Table 2-2 are superseded by this report and are no longer valid for design or licensing purposes.

Table 2-2: Superseded Issues

DOCUMENT	REFERENCE	VERSION	DATE
Volspruit MRD Design Report	19-918-SW (Volspruit MRD Design)	0.0	2 December 2022
Volspruit Stormwater Management Plan	19-918-SW (Volspruit SWMP)	0.0	2 December 2022

2.5 Legislation and Guidance Documents

The design has been carried out in accordance with the following legislation, regulations, standards and guidelines.

Legislation

- National Water Act, 1998 (Act 36 of 1998);
- National Environmental Management Act, 1998 (Act 107 of 1998);
- National Environmental Management: Waste Act, 2008 (Act 59 of 2008);
- Mine Health and Safety Act, 1996 (Act 29 of 1996); and

- Engineering Profession Act, 2000 (Act 46 of 2000), and the Code of Conduct for Registered Persons issued under it.

Regulations

- Government Notice R704 of 1999, Regulations on the use of water for mining and related activities aimed at the protection of water resources;
- Government Notice R139 of 2012, Regulations regarding the Safety of Dams;
- Government Notice R634 of 2013, Waste Classification and Management Regulations;
- Government Notice R635 of 2013, National Norms and Standards for the Assessment of Waste for Landfill Disposal, as amended on 7 November 2024;
- Government Notice R636 of 2013, National Norms and Standards for Disposal of Waste to Landfill, as amended by Government Notice 5523 of 7 November 2024 (Government Gazette 51521); and
- Government Notice 5576 of 2024, Notice for the Registration of Existing Dams with a Safety Risk.

Standards

- SANS 10286:1998, Code of Practice for Mine Residue;
- SANS 1526:2015, Geomembranes: High-Density Polyethylene (HDPE) Geomembranes;
- SANS 10409:2020, The Design, Selection and Installation of Geomembrane Liners;
- BS EN 1992-1-1:2023, Eurocode 2: Design of Concrete Structures. General Rules and Rules for Buildings, Bridges and Civil Engineering Structures; and
- BS EN 1992-3:2006, Eurocode 2: Design of Concrete Structures. Liquid Retaining and Containing Structures.

Guidelines

- Department of Water Affairs and Forestry, 2007. Best Practice Guideline A4: Pollution Control Dams;
- Department of Water Affairs and Forestry, 2008. Best Practice Guideline A2: Water Management for Mine Residue Deposits;
- Department of Water and Sanitation, 2016. Guideline for the Development and Implementation of Water Conservation and Water Demand Management Plans for the Mining Sector;
- Department of Water and Sanitation. Guideline for Sampling and Analysis of Wastewaters, Soils and Wastes;
- Department of Forestry, Fisheries and the Environment. Guideline for Pollution Control Barrier System Design, referred to in regulation 3(2)(j) of GN R636 as amended; and
- Water Research Commission (WRC), 2011. Guidelines on Freeboard for Dams, Volumes I and II, WRC Reports 1759/1/11 and 1759/2/11.

The following international guidelines were referenced to supplement the existing guidelines of SANS 10286 and SANCOLD:

- Global Industry Standard on Tailings Management (GISTM), August 2020;
- International Commission on Large Dams bulletins in terms of TSFs, embankment dams and filter design;
- Australian National Committee on Large Dams. Guidelines on Tailings Dams: Planning, Design, Construction, Operation and Closure, May 2012, as amended by the Addendum of July 2019; and
- Australian National Committee on Large Dams. Guidelines on the Consequence Categories for Dams, October 2012.

SANS 10286:1998 is the published standard and is the standard applied in this design. A revision of SANS 10286 was issued in draft in 2022. Where the minimum factors of safety and the safety classification in Chapters 8 and 9 follow the draft revision, this is stated in those chapters, and the draft status of that document is acknowledged.

3 DESCRIPTION OF THE CURRENT ENVIRONMENT

3.1 Locality and Physical Characteristics

The proposed Volspruit Mine site is situated approximately 15 km south of Mokopane in the Limpopo Province, on the farms Volspruit 326 KR and Zoetveld 294 KR.

The site is adjacent to the Nyl River, which flows in a north-easterly direction towards Mokopane.

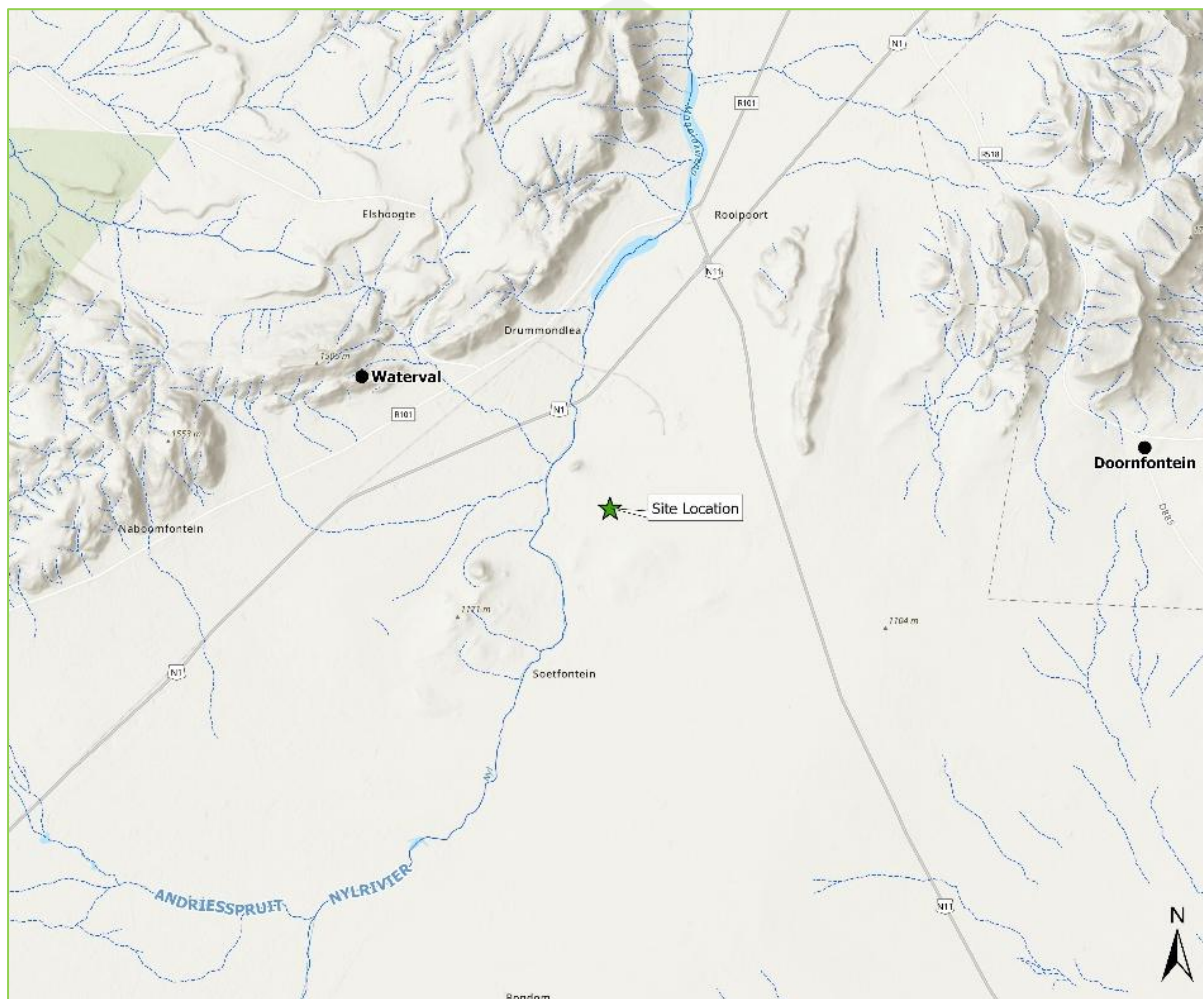


Figure 3-1: Locality Map

3.2 General Layout and Site Development Plan

The mine plan proposes two open pits, referred to as the North Pit and the South Pit, a processing plant, and the residue and water management facilities that form the subject of this report. The general arrangement is shown on the site layout drawing in Annexure J.

The clean and dirty water areas and the way each drain is set out in Chapter 7. A flood diversion berm is positioned along the Nyl River so that the 1:100-year flood line does not encroach on the North Pit.

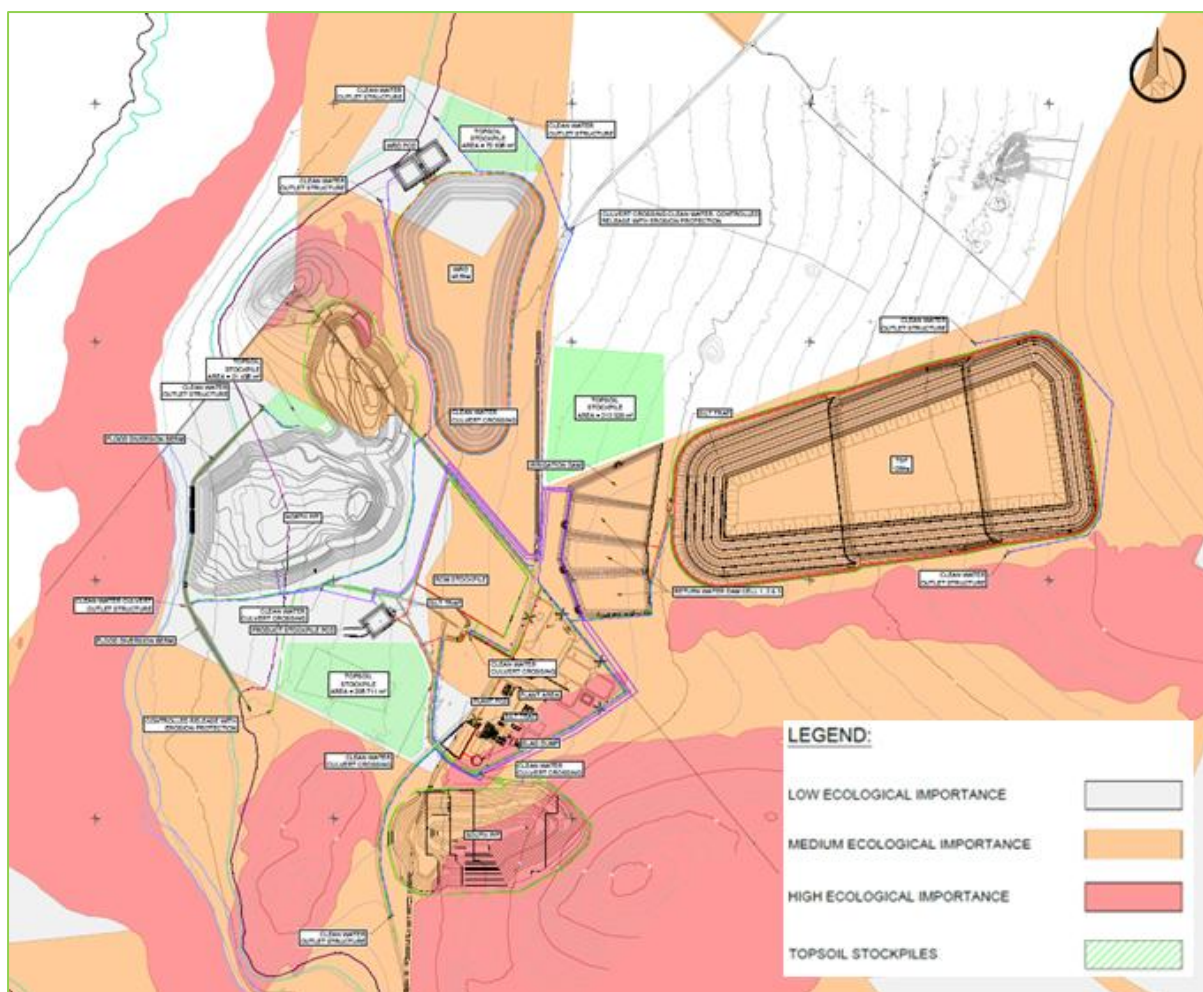


Figure 3-2: Site Layout Including Ecological Importance and Topsoil Stockpiles

3.3 Topography and Drainage: Quaternary Catchment A61E

The site lies on the eastern bank of the Nyl River, in the north-eastern section of quaternary catchment A61E. Locally the site drains westwards towards the Nyl River at an average slope in the order of 1.7%. The regional catchment drainage is towards the north-east.

The catchment delineation and the derivation of flood peaks are set out in Chapter 7.

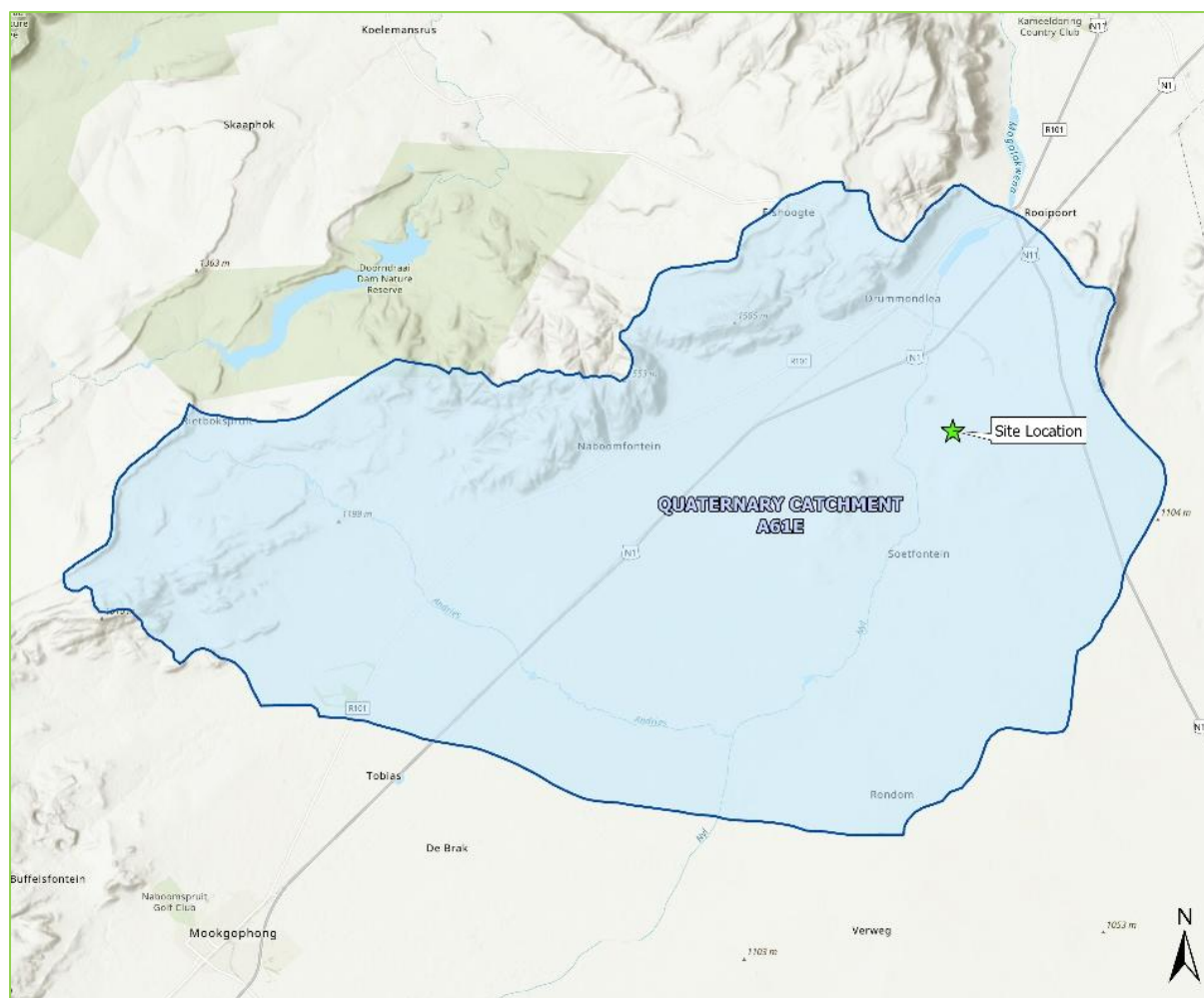


Figure 3-3: Quaternary Catchment A61E of the Limpopo Water Management Area

3.4 Underground and Adjacent Mining

No underground mining is planned by Volspruit Mining Company beneath the residue facilities, and the mining activities on the Volspruit property lie to the south-west of the facility footprints.

The adjacent Grasvally property is owned by another company. Its current mine plans show that underground mining on that property will not affect the TSF, and the open pit workings close to the property boundary are being backfilled. Mining on the adjacent property is therefore not expected to affect the stability of the residue facilities.

As a precaution, the TSF footprint has been positioned on the assumption that mining on the adjacent property could extend to a depth of 60 m, and the offset between the facility and the property boundary provides the buffer required for foundation stability on that assumption. If deeper or closer mining is permitted on the adjacent ground, the footprint geometry is to be reviewed.

3.5 Geology

3.5.1 Regional Geology

The Volspruit Project is situated on the northern limb of the Bushveld Complex, whose mafic rocks host the layers rich in platinum group elements and chromium that are mined at the site. The site falls on the 1:250 000 geological map sheet 2428 Nylstroom (Council for Geoscience, 1978).

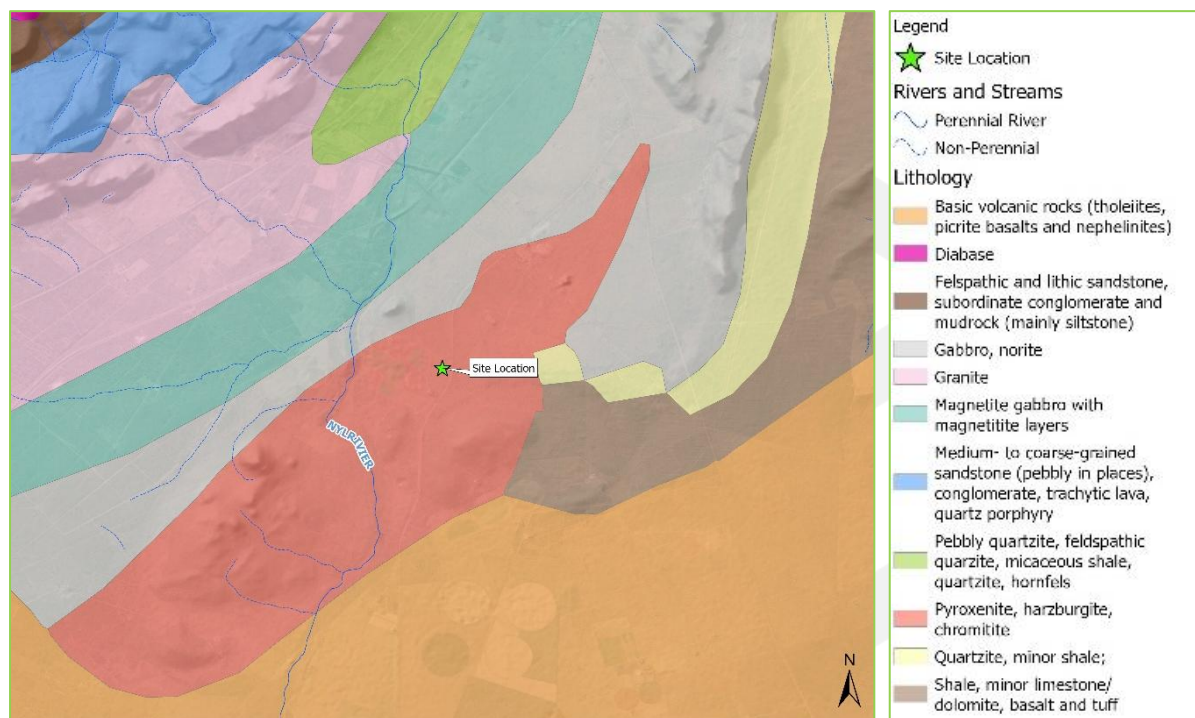


Figure 3-4: Geology in Relation to the Volspruit Mine

3.5.2 Site Geology

The site is underlain by pyroxenite and harzburgite of the Lower Zone of the Rustenburg Layered Suite. These rocks have generally weathered to a clayey sand residual material, overlain by a thin cover of sandy transported soils. Nodular ferricrete was encountered in only a few test pits. Highly weathered, closely fractured very soft rock typically occurs at depths shallower than 2 m across the site, with rock quality and hardness increasing with depth. Test pit excavations frequently met soft to medium hard rock immediately below the residual soils, resulting in refusal.

The geotechnical investigation is reported in Chapter 4 and attached as Annexure C.

3.6 Groundwater

Groundwater is hosted in fractures associated with the block faulting across the area, with the upper weathered and fractured zone forming the principal water-bearing horizon. Water levels within the proposed pit shells range from 1.6 m to 13.5 m below ground level, shallowest closest to the Nyl River. No groundwater seepage was encountered in any of the test pits excavated within or outside the perimeter of the TSF, nor in the test pits excavated for the WRD.

The aquifer is classified as a major aquifer system, and a high level of groundwater protection is required. The barrier system adopted for each facility is set out in Chapter 10.

The Nyl River is underlain by an impermeable clay layer that separates river flow from the aquifer beneath it. The river does not recharge groundwater under normal flow conditions. Recharge from the river occurs where it overtops its banks and flood water spreads across the floodplain beyond the extent of the clay. The groundwater study estimates the regional recharge at 2.6% to 4.8% of the mean annual precipitation, or 14 mm/a to 25.5 mm/a, and the geochemical study notes that additional recharge from irrigation and from flood events is likely.

The geohydrological study and the geochemical assessment of pit water quality are attached as Annexure F.

3.7 Seismicity

The peak ground acceleration for the site is 0.08 g, taken from the Seismic Hazard Map of South Africa (Kijko et al., 2003). This is the maximum ground acceleration during an earthquake having a 10% probability of being exceeded in a 50-year period, which corresponds to a return period of approximately 1:475 years.

No site-specific probabilistic seismic hazard assessment has been carried out for the project. The seismic design parameters adopted are given in Chapter 9.

3.8 Regional Climate

The site falls within the summer rainfall region of South Africa. The climate is warm temperate, with cold dry winters and warm summers. Summer rainfall is sporadic and associated with frequent thunderstorms producing high-intensity rainfall events.

The mean annual precipitation adopted for the site is 559 mm, taken from the Palmer Estate weather station (SAWS 0634050 W) over the record period 1903 to 2000. The Doorndraai Dam weather station (A6E001) is the closest active station on the DWS database. It lies 19 km north-west of the site, in the upper reaches of the adjacent quaternary catchment A61J, and is the source of the S-pan evaporation record, which gives a mean annual evaporation of 1 756 mm.

Evaporation exceeds rainfall in every month of the year, producing a marked moisture deficit. The maximum evaporation occurs in December, at a mean of 6.30 mm per day (Table 3-1).

3.9 Rainfall, Evaporation and Storm Rainfall Depths

The mean monthly precipitation and S-pan evaporation figures adopted for the design are given in Table 3-1.

Table 3-1: Rainfall and S-Pan Evaporation Data

MONTH	PRECIPITATION (mm)	S-PAN EVAPORATION (mm)
January	109.3	192.7
February	88.8	161.8
March	65.6	155.4
April	33.2	121.3
May	11.5	99.5
June	4.5	79.2
July	3.8	87.9
August	4.4	122.0
September	13.6	163.0
October	45.6	192.1
November	83.0	185.9
December	95.5	195.3
Total	559	1 756

Design rainfall depths were determined using the Design Rainfall Estimation in South Africa software (Smithers and Schulze, 2003). The depths are gridded point values for the block centred on 24°21' S, 28°57' E at an altitude of 1 063 m, which is the site position. Best estimate values are adopted. The depths for each duration and return period are given in Table 3-2, in millimetres.

The design rainfall depths and the mean annual precipitation come from different sources and are used for different purposes. The depths drive the flood peak calculations. The mean annual precipitation is reported as the site climate and, in the flood peak calculations, selects the rural runoff coefficient band. The mean annual precipitation of 559 mm from the Palmer Estate record has been adopted for the design calculations in place of the mean annual precipitation of 530 mm returned for the site grid point, because it derives from a quality controlled daily rainfall record for a gauge 9.2 km from the site and is consistent with the two closest gauges, Rondebos at 5.1 km and Kalkfontein at 8.0 km, which return 551 mm and 544 mm respectively. Both values fall within the same rural runoff coefficient band, so the choice between them does not alter the computed flood peaks.

Table 3-2: Adopted Design Rainfall

DURATION	1:2	1:5	1:10	1:20	1:50	1:100	1:200
5 min	9.4	12.9	15.3	17.8	21.3	24.1	27.1
10 min	14.0	19.1	22.8	26.5	31.7	35.9	40.3
15 min	17.7	24.1	28.7	33.4	40.0	45.2	50.8
30 min	22.4	30.5	36.3	42.3	50.6	57.3	64.3
45 min	25.7	35.0	41.7	48.5	58.1	65.7	73.9
1 hour	28.4	38.6	46.0	53.5	64.1	72.5	81.5
1.5 hour	32.6	44.3	52.8	61.5	73.5	83.2	93.5
2 hour	35.9	48.9	58.2	67.8	81.1	91.8	103.1
4 hour	42.1	57.4	68.3	79.5	95.2	107.7	121.0
6 hour	46.2	63.0	75.0	87.3	104.5	118.3	132.9
8 hour	49.4	67.3	80.2	93.3	111.6	126.4	142.0
10 hour	52.0	70.9	84.4	98.2	117.5	133.0	149.5
12 hour	54.3	73.9	88.0	102.4	122.6	138.8	155.9
16 hour	58.0	79.0	94.1	109.5	131.0	148.3	166.6
20 hour	61.0	83.2	99.0	115.3	137.9	156.1	175.4
24 hour	63.7	86.7	103.3	120.2	143.8	162.8	182.9
1 day	52.9	72.1	85.9	100.0	119.6	135.4	152.1
2 day	65.2	88.8	105.7	123.0	147.2	166.6	187.2
3 day	73.6	100.2	119.4	138.9	166.2	188.1	211.4
4 day	80.0	108.9	129.7	150.9	180.6	204.4	229.7
5 day	85.3	116.2	138.4	161.0	192.6	218.1	245.0
6 day	89.9	122.5	145.8	169.7	203.1	229.9	258.2
7 day	94.0	128.0	152.5	177.4	212.3	240.3	270.0

4 GEOTECHNICAL INVESTIGATION

Peregrine Geoconsultants was appointed by EcoE in June 2020 to carry out a feasibility level geotechnical investigation for the proposed surface infrastructure of the Volspruit Mine. The fieldwork was completed in July 2020. The position of the WRD was changed in January 2021, which required further fieldwork on the new WRD footprint in February 2021. The findings are given in report PG19-003/673 Rev 02, dated 30 June 2021, which is attached as Annexure C. The report also draws on the test pits of the 2012 feasibility investigation by Knight Hall Hendry (KHH) where these fall within or near the facility footprints.

The objectives of the investigation were to establish the nature and engineering properties of the soil and rock strata underlying the site, to comment on possible shallow groundwater seepage, to establish the excavatability of the ground profile, and to provide recommendations for the design of foundations, bulk earthworks and drainage and for the use of the soils as road and liner construction materials.

4.1 Investigation Programme

The fieldwork comprised 38 test pits and three rotary cored boreholes, carried out in accordance with the SAICE Site Investigation Code of Practice (2010). All test pits and boreholes were profiled by an engineering geologist in accordance with Brink and Bruin (2002). The 2020 test pits were excavated with a TLB supplied by the mine and the 2021 test pits with a hired 20 t tracked excavator. SPT tests were carried out at 1.5 m intervals in the soil horizons of the boreholes, and standpipe piezometers were installed in boreholes TSF-BH01 and WRD-BH01. The coverage of each area is given in Table 4-1, and the test positions are shown in Figure 4-1 to Figure 4-4.

Twenty-six soil samples were tested at a SANAS accredited laboratory for foundation indicators, maximum dry density and optimum moisture content, CBR, pH and electrical conductivity, falling head permeability on samples remoulded to 95% of Standard Proctor density, collapse potential by double oedometer, and shear strength by slow shearbox. Three rock cores were tested for unconfined compressive strength. The closely fractured upper rock did not yield core of the minimum test length, so the rock tests are skewed towards the better-quality rock at depth.

Table 4-1: Investigation Coverage by Area

INVESTIGATION AREA	FACILITIES IN THIS DESIGN	TEST PITS (PEREGRINE 2020 AND 2021)	BOREHOLES	OTHER INFORMATION USED
TSF and RWD	TSF, RWD cells and irrigation dam	12 (TSF-TP01 to TSF-TP06, MPCD-TP01 to MPCD-TP06)	TSF-BH01, TSF-BH02	9 KHH test pits (TA1 to TA7, TA11, TA12)
New WRD	WRD and WRD PCD	5 (WRD-TP09 to WRD-TP13)	None	9 KHH test pits (TW2 to TW10)
RoM	RoM pad and RoM PCD	3 (ROM-TP01 to ROM-TP03)	None	2 exploration boreholes (GV003S, GV004S)

INVESTIGATION AREA	FACILITIES IN THIS DESIGN	TEST PITS (PEREGRINE 2020 AND 2021)	BOREHOLES	OTHER INFORMATION USED
Plant	Plant PCD	7 (PPCD-TP01, PPCD-TP02, PL-TP01 to PL-TP05)	None	9 KHH test pits (TP1 to TP9)
Old WRD (superseded position)	None	8 (WRD-TP01 to WRD-TP08)	WRD-BH01	4 exploration boreholes
Road realignment	None	3	None	None

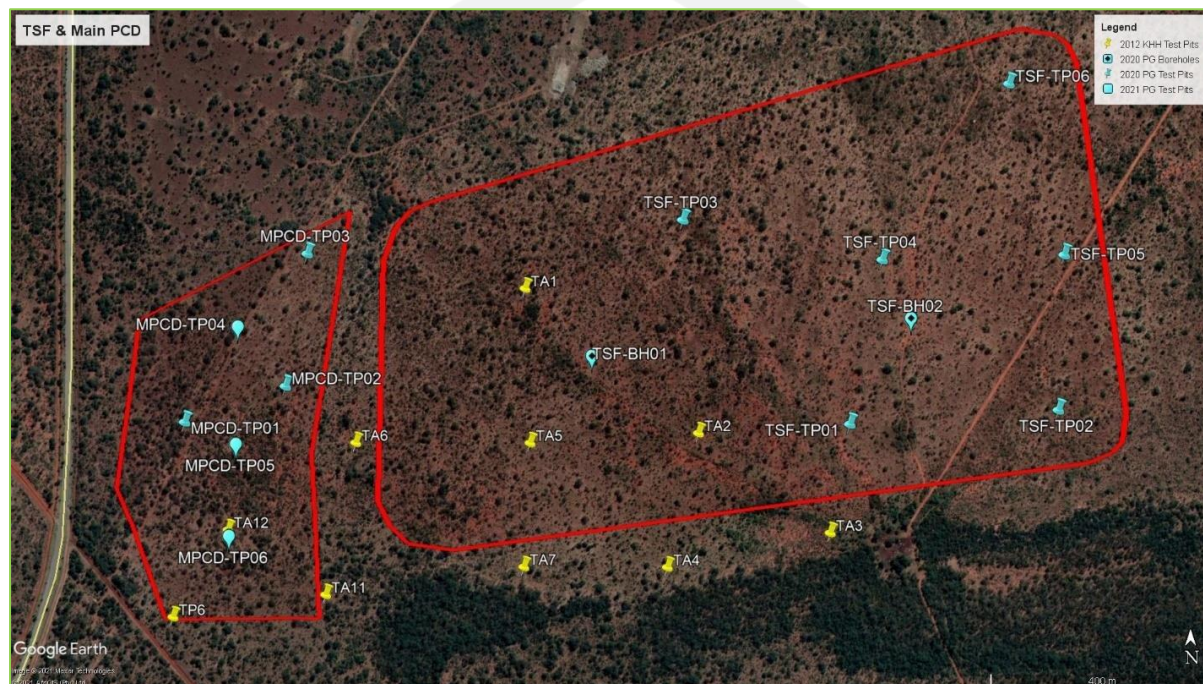


Figure 4-1: Test Positions, TSF and RWD Area (Peregrine Geoconsultants, 2021)

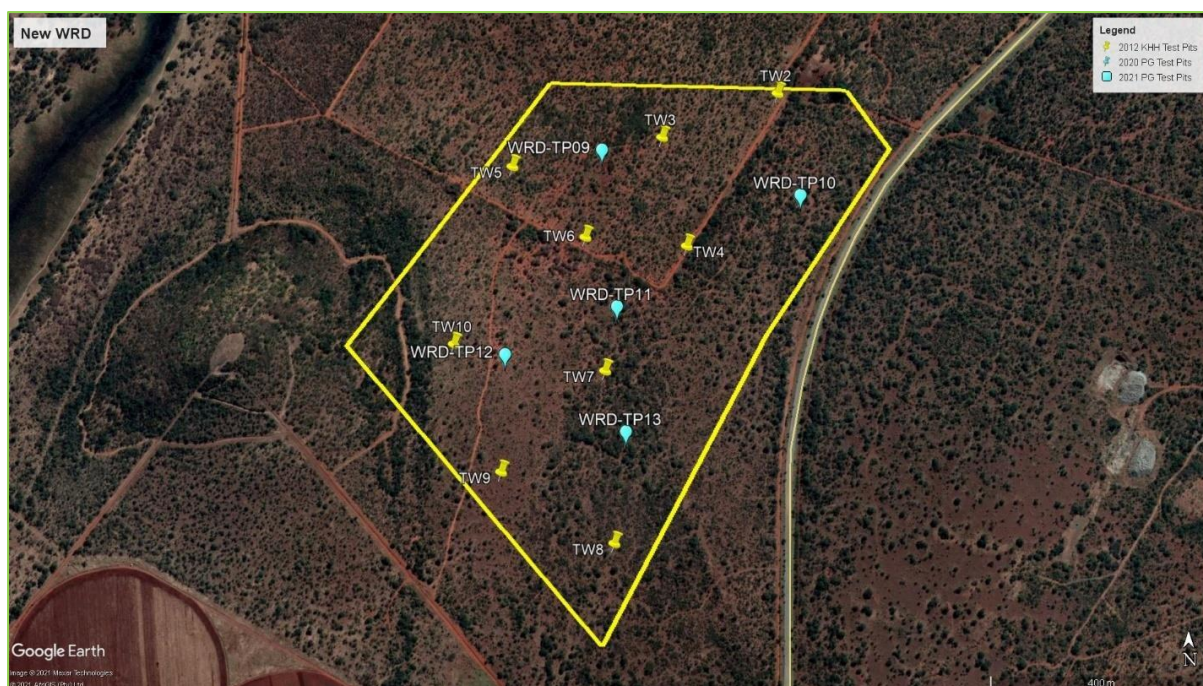


Figure 4-2: Test Positions, New WRD Area (Peregrine Geoconsultants, 2021)

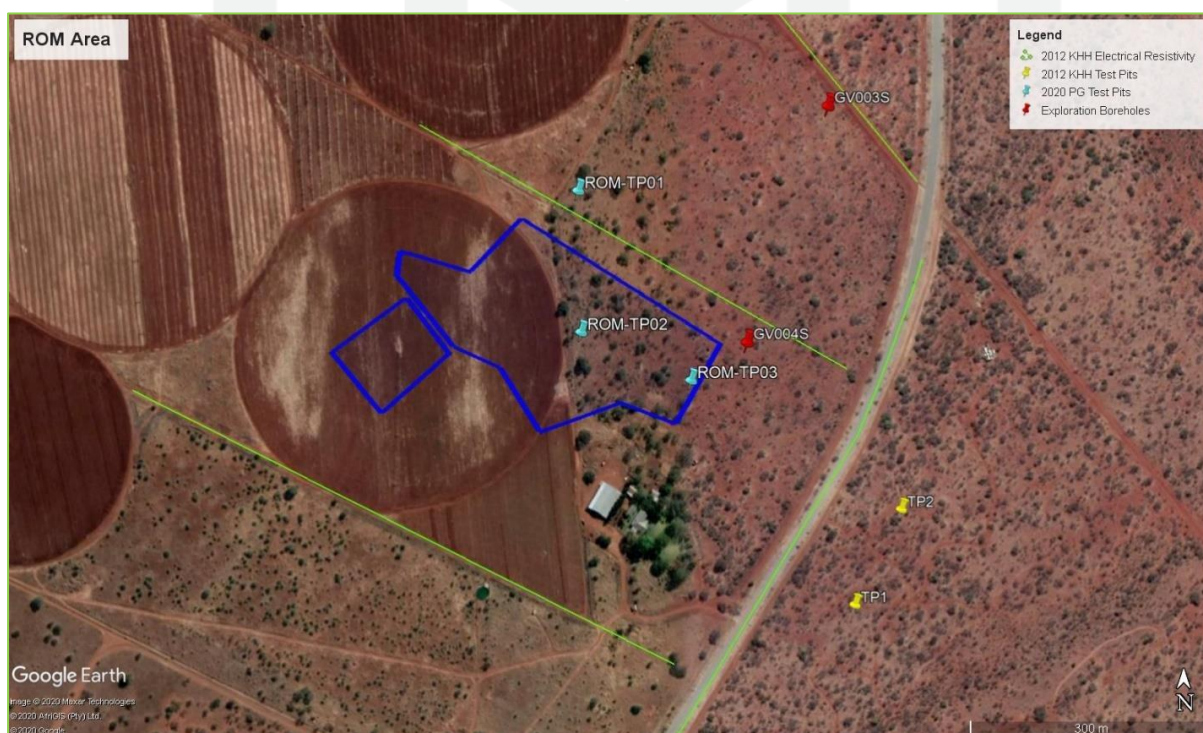


Figure 4-3: Test Positions, RoM Area (Peregrine Geoconsultants, 2021)

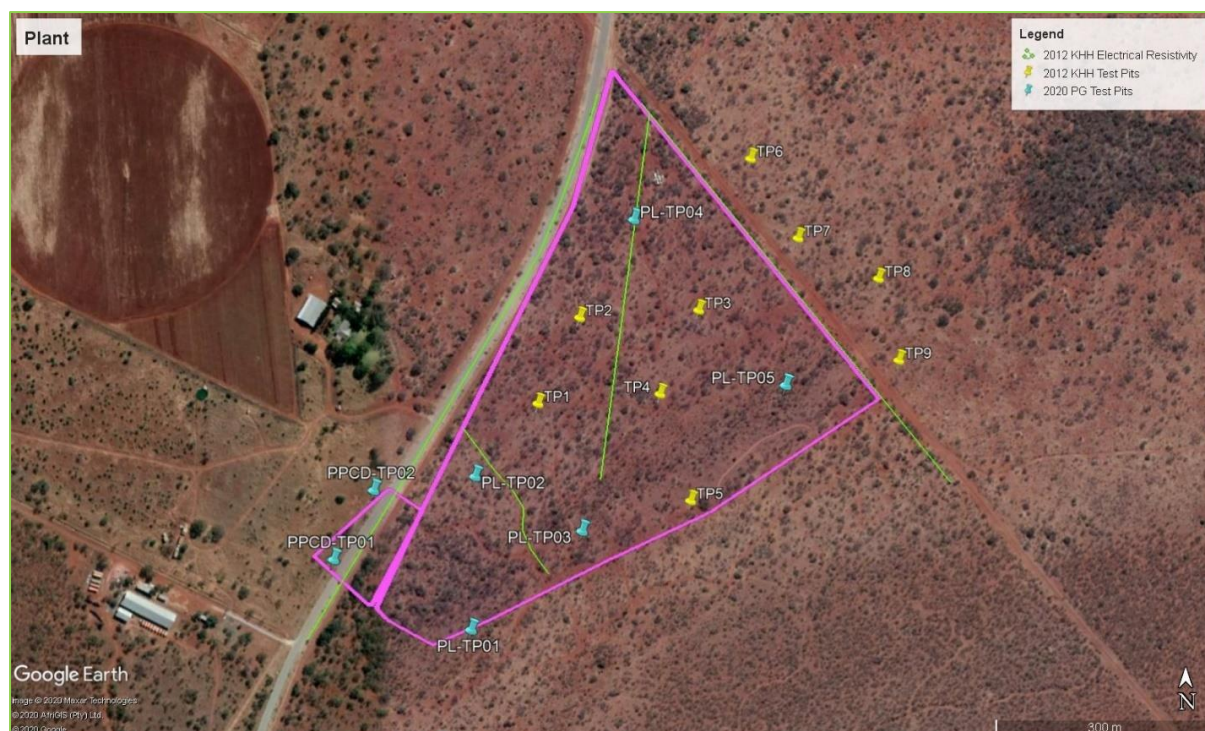


Figure 4-4: Test Positions, Plant Area (Peregrine Geoconsultants, 2021)

4.2 Soil Profile and Founding Conditions

The generalised soil and rock layers identified across the site are described in Table 4-2, with the allowable bearing capacities recommended in the geotechnical report. Bedrock was encountered in all, but two of the 53 test pits considered, with refusal of the TLB and the excavator typically less than 2.0 m below natural ground level. The rock generally increases in quality and hardness with depth, and the transition from very soft, to soft-to-medium hard rock is often rapid. None of the soils showed signs of historic heave or dispersivity.

Table 4-2: Generalised Ground Profile

LAYER	DESCRIPTION	THICKNESS	ALLOWABLE BEARING CAPACITY
Sandy transported soil (TR-S)	Dry, reddish brown, pinholed, loose to dense clayey and silty sand, with gravel in places. Highly variable, with fines contents of 10% to 80% and PI of 5 to 22. Prone to collapse settlement.	0.1 m to 2.3 m	40 kPa (loose to medium dense) and up to 100 kPa (medium dense to dense), once the collapse potential has been removed or minimised
Nodular ferricrete (F-N)	Slightly moist, brownish red speckled black, medium dense to dense ferricrete nodules in a silty sand matrix. Encountered sporadically, mostly in an area west of the WRD.	0.2 m to 3.05 m	100 kPa
Sandy residual soil (RES-S)	Residual pyroxenite and harzburgite. Clayey gravelly sand, loose to very dense, with a pinholed structure in places. Highly variable, from G6 to poorer than G9.	0.1 m to 1.6 m	50 kPa (loose to medium dense) and up to 150 kPa (dense), with the founding excavations inspected for collapse potential

LAYER	DESCRIPTION	THICKNESS	ALLOWABLE BEARING CAPACITY
Very soft to soft rock (R-VS)	Completely to highly weathered, fine to medium grained, closely jointed, very soft to soft rock pyroxenite or harzburgite. Present below the residual soil in about half of the test pits.	Generally thin	300 kPa
Soft to medium hard rock (R-SM)	Pyroxenite or harzburgite, highly weathered and highly fractured near surface, improving in quality and hardness with depth. Refusal of the TLB and the excavator.	Not determined	1 000 kPa

4.2.1 TSF and RWD Area

The TSF area is generally covered by a 0.4 m to 1.7 m thick layer of sandy transported soil. No ferricrete was encountered in the TSF area. Below the transported soil, a 0.1 m to 1.15 m thick layer of sandy residual soil was found in most test pits, followed by harzburgite bedrock. In most test pits the TLB refused within 1.0 m of reaching the very soft rock. The test pits on and around the RWD refused at 0.8 m to 2.8 m below natural ground level, most of them at 1.6 m to 2.8 m. The test profiles are summarised in Table 4-3.

The borehole logs are very similar to the test pit profiles but show some variation in rock quality across the TSF footprint. The poor material and core recoveries over the first 9.0 m of TSF-BH02 are attributed to the drilling and are not a true representation of the profile. TSF-BH01 encountered good quality rock from shallow depths, confirmed by unconfined compressive strengths of more than 250 MPa. The geotechnical report recommends a geophysical survey across the TSF footprint, confirmed by additional drilling, to identify any deeply weathered, faulted or fractured zones. The survey and drilling have not yet been done and form part of the pre-construction geotechnical investigation, and the design is reviewed against their results before construction starts. The design does not depend on them for containment. The TSF, the RWD cells and the dams are lined, with sub-surface drainage beneath the liner, so a fractured zone in the foundation does not form a seepage path from the facility. Any faulted, fractured or deeply weathered zone found is excavated and replaced, or treated, as directed by the Engineer. Zones missed by the survey are picked up when the stripped footprint is inspected and mapped by an engineering geologist before ground improvement starts.

Table 4-3: Summary of Test Profiles, TSF and RWD Area

TEST PIT OR BOREHOLE	TR-S (m)	RES-S (m)	R-VS (m)	R-SM (m)	REFUSAL (m)
KHH (2012)					
TA1	0 – 0.90	0.90 – 1.20	1.20 – 1.40	> 1.40	1.40
TA2	0 – 1.10	1.10 – 2.00		> 2.00	2.00
TA3	0 – 1.30	1.30 – 2.10		> 2.10	2.10
TA4	0 – 1.70	1.70 – 2.20		> 2.20	2.20

TEST PIT OR BOREHOLE	TR-S (m)	RES-S (m)	R-VS (m)	R-SM (m)	REFUSAL (m)
TA5	0 – 1.00	1.00 – 2.00		> 2.00	2.00
TA6	0 – 0.70	0.70 – 1.40		> 1.40	1.40
TA7	0 – 1.00	1.00 – 1.70		> 1.70	1.70
TA11	0 – 1.30	1.30 – 1.60		> 1.60	1.60
TA12	0 – 0.70	0.70 – 1.70		> 1.70	1.70
Peregrine (2020 and 2021)					
MPCD-TP01	0 – 0.75			0.75 – 0.80	0.80
MPCD-TP02	0 – 0.50	0.50 – 1.65	1.65 – 2.20	> 2.20	2.20
MPCD-TP03	0 – 0.55	0.55 – 0.70	0.70 – 2.20	> 2.20	2.20
MPCD-TP04	0 – 0.60	0.60 – 1.55	1.55 – 1.90	1.90 – 2.80	2.80
MPCD-TP05	0 – 0.70	0.70 – 1.30	1.30 – 2.20		2.20
MPCD-TP06	0 – 1.35	1.35 – 2.30		2.30 – 2.50	2.50
TSF-TP01	0 – 1.00	1.00 – 1.10	1.10 – 2.30	> 2.30	2.30
TSF-TP02	0 – 0.90		0.90 – 1.20	1.20 – 1.25	1.25
TSF-TP03	0 – 0.45		0.45 – 0.85	0.85 – 0.90	0.90
TSF-TP04	0 – 0.80	0.80 – 1.00	1.00 – 1.90	> 1.90	1.90
TSF-TP05	0 – 0.90		0.90 – 0.95	0.95 – 1.00	1.00
TSF-TP06	0 – 0.40	0.40 – 0.55	0.55 – 0.90	> 0.90	0.90
TSF-BH01	0 – 0.50	0.50 – 1.00	1.00 – 1.80	1.80 – 7.31	Not applicable
TSF-BH02	0 – 0.85	0.85 – 2.60	2.60 – 12.00	12.00 – 15.00	Not applicable

4.2.2 New WRD Area

The New WRD area is generally covered by a 0.8 m to 2.3 m thick layer of sandy transported soil. Below this, a 0.1 m to 0.9 m thick layer of nodular ferricrete was found in eight of the fourteen test pits, followed by a 0.3 m to 1.3 m thick layer of sandy residual soil over harzburgite and pyroxenite bedrock. In all the test pits, including those excavated by the 20-t excavator, refusal was encountered within 1.0 m of reaching the very soft rock. Bedrock is present below the entire WRD area at depths of not more than 3.1 m. The degree and depth of weathering and the fracture frequency of the rock vary greatly, possibly indicating historic shear zones within the rock. These zones are covered by the same pre-construction geophysical survey and footprint inspection as the TSF (section 4.2.1). The test profiles are summarised in Table 4-4.

Refusal around the WRD PCD varied from 1.2 m to 3.1 m below natural ground level. For a 4 m deep PCD basin, intermediate and hard excavation should therefore be expected over 0.9 m to 2.8 m of the bulk earthworks.

Table 4-4: Summary of Test Profiles, New WRD Area

TEST PIT	TR-S (m)	F-N (m)	RES-S (m)	R-VS (m)	R-SM (m)	REFUSAL (m)
KHH (2012)						
TW2	0 – 1.50	1.50 – 2.40	2.40 – 2.70		> 2.70	2.70
TW3	0 – 1.50	1.50 – 1.70	1.70 – 2.50		> 2.50	2.50
TW4	0 – 1.30	1.30 – 1.80	1.80 – 2.10		> 2.10	2.10
TW5	0 – 1.70		1.70 – 2.70		> 2.70	2.70
TW6	0 – 2.30	2.30 – 2.50	2.50 – 2.90		> 2.90	2.90
TW7	0 – 1.50	1.50 – 1.70	1.70 – 2.10		> 2.10	2.10
TW8	0 – 1.00		1.00 – 2.30		> 2.30	2.30
TW9	0 – 1.30		1.30 – 1.80		> 1.80	1.80
TW10	0 – 1.30		1.30 – 1.80		> 1.80	1.80
Peregrine (2021)						
WRD-TP09	0 – 2.10	2.10 – 2.20		2.20 – 3.10		3.10
WRD-TP10	0 – 1.60	1.60 – 2.00		2.00 – 2.85		2.85
WRD-TP11	0 – 1.10	1.10 – 1.45		1.45 – 1.60		1.60
WRD-TP12	0 – 1.40			1.40 – 1.70		1.70
WRD-TP13	0 – 0.80				0.80 – 1.20	1.20

4.2.3 RoM Area

The RoM area is generally covered by a 0.65 m to 1.6 m thick layer of sandy transported soil. Only in test pit ROM-TP02 was this underlain by sandy residual soil, 1.0 m thick. Below this, poor quality rock was encountered that rapidly hardens, with refusal of the TLB at 0.9 m to 2.7 m below natural ground level. The two exploration boreholes nearby give no detail of the soil strata and are not considered a reliable representation of the ground profile. The test profiles are summarised in Table 4-5.

The depth to bedrock is variable. There was no access to the RoM PCD or the western portion of the RoM area during the investigation, and the geotechnical report recommends test pits in these areas to verify the expected excavation conditions. These have not yet been excavated and form part of the pre-construction investigation. The RoM test pits show the same shallow soil over rock as the rest of the site, and the design of the RoM pad and PCD does not rely on a more favourable profile: the collapsible soils are treated, and intermediate and hard excavation is allowed for. The founding is confirmed by inspection when the basin is excavated.

Table 4-5: Summary of Test Profiles, RoM Area

TEST PIT	TR-S (m)	RES-S (m)	R-VS (m)	R-SM (m)	REFUSAL (m)
ROM-TP01	0 – 0.65		0.65 – 0.85	0.85 – 0.90	0.90
ROM-TP02	0 – 1.10	1.10 – 2.10	2.10 – 2.70	> 2.70	2.70
ROM-TP03	0 – 1.60			1.60 – 1.70	1.70

4.2.4 Plant PCD Area

Two test pits were excavated at the Plant PCD. PPCD-TP01 found 0.35 m of sandy transported soil over 0.55 m of sandy residual soil, and PPCD-TP02 found 0.4 m of transported soil over 0.4 m of residual soil, with refusal on soft to medium hard rock at 1.0 m and 0.85 m respectively. Most of the plant area is underlain by competent rock within the top 1.8 m.

4.2.5 Groundwater

No groundwater seepage was encountered in any of the test pits. The water rest levels measured in the boreholes, 72 hours or more after drilling, were 8.75 m in TSF-BH01, 13.93 m in TSF-BH02 and 9.68 m in WRD-BH01 below the borehole collar. The geotechnical report foresees no problems with groundwater for the development of the TSF, the WRD and their PCDs and ancillary structures.

4.3 Laboratory Test Results

The laboratory results are summarised by layer in Table 4-6.

Table 4-6: Summary of Laboratory Results by Layer

LAYER	FINES (%)	CLAY (%)	LL	PI	PERMEABILITY (m/s)	MDD (kg/m ³)	OMC (%)	CBR (%)
TR-S	40 (10 to 80)	14 (2 to 32)	29 (20 to 43)	12 (5 to 22)	4.35×10^{-10} to 2.74×10^{-9}	1 895 (1 642 to 2 032)	13.1 (9.6 to 19.0)	12 (10 to 15)
F-N	24 (19 to 29)	9 (5 to 12)	27 (25 to 30)	11 (9 to 13)	1.26×10^{-9}	1 880	13.6	Not tested
RES-S	21 (6 to 35)	6 (1 to 18)	33 (19 to 40)	14 (7 to 22)	7.14×10^{-9}	1 916 (1 712 to 2 044)	13.4 (10.2 to 18.0)	23 (9 to 38)
R-VS	11 (8 to 13)	2 (1 to 5)	19 (0 to 33)	9 (0 to 19)	1.69×10^{-8}	2 095	10.6	7
R-SM	No samples tested							

Average values, with the range in brackets. LL is the liquid limit and PI the plasticity index. Falling head permeability on samples remoulded to 95% of Standard Proctor density. MDD and OMC by Standard Proctor. CBR at 95% of Mod AASHTO density.

Pinholed structures were noted in the transported soils at numerous locations, and undisturbed samples were tested for collapse potential by double oedometer (Table 4-7). The results give collapse potentials of 5 to 14%, which classify as "trouble" to "severe trouble" in terms of Jennings and Knight (1975), confirming that the transported soils are collapsible and require treatment beneath the facilities (section 4.4).

Table 4-7: Double Oedometer Results

TEST PIT	DEPTH (m)	LAYER	DRY DENSITY (kg/m ³)	COLLAPSE POTENTIAL AT 100 kPa (%)	COLLAPSE POTENTIAL AT 200 kPa (%)	COLLAPSE POTENTIAL AT 400 kPa (%)
WRD-TP07 (old WRD)	0 – 1.2	TR-S	1 463	7.0	9.9	12.1
TSF-TP02	0 – 0.9	TR-S	1 401	5.2	9.8	14.2
TSF-TP04	0 – 0.8	TR-S	1 409	7.6	11.7	15.6
TSF-TP05	0 – 0.9	TR-S	1 295	5.2	9.8	14.3

Slow shearbox tests on two samples of transported soil, remoulded to 95% of Standard Proctor density, gave friction angles of 26.9° (TSF-TP02) and 27.6° (WRD-TP07), with zero cohesion. The two 2012 shearbox tests were remoulded to 95% of Mod AASHTO density and are disregarded in the geotechnical report. The friction angle and zero cohesion adopted for the transported soil in the stability analyses (Table 11-5) are based on these shearbox results. The residual soil, the ferricrete and the rock were not tested for shear strength, and their parameters were taken from geotechnical literature (Look, 2014). The rock strengths adopted are well below the measured core strengths, because the fractured upper rock is weaker than the cores tested.

The unconfined compressive strengths of the rock cores were 281.2 MPa and 259.1 MPa from TSF-BH01, classed as extremely hard rock, and 23.3 MPa from WRD-BH01, classed as medium hard rock.

The soils tested have pH values of 5.8 to 7.3, which is slightly acidic to neutral, and electrical conductivities of 0.006 S/m to 0.033 S/m, rated generally not corrosive to corrosive. In terms of SANS 1200-G this corresponds to moderate exposure conditions, and the geotechnical report recommends a minimum concrete cover of 40 mm for 20 MPa and 25 MPa concrete, 30 mm for 30 MPa, 25 mm for 40 MPa and 20 mm for 50 MPa concrete. Buried steel pipes and electrical wiring are to be adequately insulated.

4.4 Problem Soils and Ground Improvement

There is a high probability that the collapsible nature of the transported soils will show itself as a densification of the soil, with almost instantaneous settlement, when the moisture content increases while the soil is under an applied load such as a waste storage facility. This form of collapse settlement may crack layerworks and liner systems. The settlement is expected to be in the order of a 10% volume reduction on saturation.

The collapsible soils are near surface, generally less than 1.5 m thick, and can be treated by removal and replacement or by in-situ wetting and compaction. For the large areas of the TSF and the WRD, the geotechnical report expects impact rolling combined with wetting of the exposed soils to be the most time and cost-efficient ground improvement and recommends the same approach for both facilities. The approach is given in section 12.3.

4.5 Excavation Conditions

In terms of COLTO Subclause 3303 (1998), the transported soil, the ferricrete, the residual soil and the very soft rock are classified as soft excavation. The TLB and the excavator encountered abrupt refusal in the soft to medium hard rock, which is classified as intermediate and probably hard excavation in places. Allowance for these conditions is required in the bill of quantities, particularly for the dam and PCD basins, which are expected to be up to 4 m deep. Whether a large, tracked excavator can progress some way into the rock depends on the plant used by the contractor and should be tested in a trial section on site.

4.6 Material Suitability

Some portions of the transported soil would be suitable for engineered layerworks, but these are very difficult to separate from the unsuitable portions, and the transported soil is not considered a feasible source of material for engineered layerworks. It is suitable as bulk fill in platforms but does not compact well. The ferricrete is too irregular in occurrence and thickness to be a viable source. The residual soil is suitable as subgrade and lower selected layers but is too variable for upper selected or subbase layers or for engineered fill. The very soft rock tested is unsuitable for layerworks, and the soft to medium hard rock is expected to be crushed by compaction into a fine-grained material that behaves like a silty soil.

Both the transported and the residual soils meet the permeability requirement of 1×10^{-9} m/s at 95% of Standard Proctor density for a natural liner material, and in many instances the criteria of Benson et al. (1994). The grading and plasticity of the transported soils are highly variable. The residual soil is considered marginally suitable for the construction of the TSF starter walls. The geotechnical report recommends a more widespread material sourcing test pit exercise across the mine property before construction.

4.7 Implications for the Design

The findings of the investigation are carried into the design as follows:

- the collapsible transported soils beneath the TSF, the WRD and the RoM pad are treated by ground improvement before the foundation preparation layer is placed (sections 11.5 and 12.3, and Chapter 13);
- beneath the silt traps, all transported soil is removed, and the founding layer is confirmed by a competent geotechnical engineer not to be collapsible (Chapter 15);

- light structures are founded within medium dense or dense residual soil or on bedrock, and are to tolerate total settlements of up to 15 mm and differential movements of up to 1:500;
- the penstock towers, the only heavy structures at the TSF, are recommended to be founded directly on the soft to medium hard rock, at depths of 0.8 m to 2.3 m;
- allowance is made for intermediate and hard excavation in the dam and PCD basins, the box cuts and the deeper foundations;
- concrete cover and the protection of buried services are set for moderate exposure conditions;
- no groundwater is expected in the excavations, and the 2 m unsaturated zone below the base of the facilities required by GN R636 is maintained (sections 3.6 and 12.3);
- faulted, fractured or deeply weathered zones beneath the TSF and the WRD are identified by a geophysical survey and confirmatory drilling before construction, and by inspection of the stripped footprint by an engineering geologist before ground improvement starts (section 4.2.1);
- test pits are excavated at the RoM PCD and the western part of the RoM area before construction, to confirm the founding and excavation conditions (section 4.2.3); and
- the friction angle of the transported soil used in the stability analyses is based on the shearbox tests, and the parameters of the other in-situ materials on geotechnical literature (Look, 2014) (sections 4.3, 11.14 and 12.7).

5 MATERIALS WASTE CLASSIFICATION

5.1 Sampling and Analysis Methodology

The waste types are taken from the waste classification update prepared by EcoE (2026), attached as Annexure A. It updates the 2022 classification (19-918-GEOH), using the 2021 samples (Waterlab certificates 103739 and 105875) assessed against GN R635 as amended by GN 5522 of 2024, and adds a classification of the water to be held in the irrigation dam.

The earlier 2012 waste assessment by Escience Associates is not used. It is a draft, prepared before GN R635 was promulgated on 23 August 2013 and assessed against draft limits that differ materially from the final LCT and TCT values. It is more than 13 years old, well beyond the five-year re-classification interval in regulation 4 of GN R634, and pre-dates the amendments to GN R634 and GN R635 of 7 November 2024, so it is not a classification under the current regulations.

Five samples were analysed by Waterlab for major elements by X-ray fluorescence, mineralogy by X-ray diffraction, total and leachable concentrations with the SANS 12457 distilled water leach, and acid-base accounting by the modified Sobek method. The samples are listed in Table 5-1. Liquid waste cannot be leach tested, so the modelled water quality of the North Pit sump (Mills Water, 2026), which is pumped to the irrigation dam, is compared directly against the leachable concentration thresholds.

Table 5-1: Waste Classification Samples

SAMPLE	MATERIAL	REPRESENTS	SOURCE
Rougher tails composite	Tailings from metallurgical testwork	TSF	Mintek
ROM1	Tremolite and serpentine altered orthopyroxenite with base metal sulphides	RoM pad	Drill core
ROM2	Biotite rich orthopyroxenite with disseminated base metal sulphides	RoM pad	Drill core
Pyroxenite	Olivine orthopyroxenite	WRD	Drill core
Norite	Norite	WRD	Drill core

5.2 Tailings

The rougher tails composite exceeds TCT0 for copper, manganese and nickel, with no leachable concentration above LCT0. The tailings are non-acid forming.

The tailings are deposited as a slurry and remain saturated with recirculated process water drawn from groundwater and pit dewatering, which already carries nitrate above LCT0 from historical agricultural fertiliser use. As this water is recirculated through the TSF, the character of the tailings as deposited is not stable over time and condition 7(6)(b) of GN R635 is not met (section 5.6). The tailings therefore classify as Type 3, and the TSF is designed as a Type 3 facility (section 5.7).

The tailings solids themselves, drained and dried, have no leachable concentration above LCT0 and classify as Type 4. On that basis, reclaimed tailings may be used as a construction material for the

receiving face beneath the geomembrane in cells 2 and 3, subject to the acceptance criteria in section 11.5.

5.3 Waste Rock

Both waste rock samples are non-acid forming. The pyroxenite exceeds TCT0 for cobalt and nickel and the norite for barium and lead, with no leachable concentration above LCT0 in either. The waste rock is a stable geological solid and is placed on its own on the WRD, so conditions 7(6)(b) and 7(6)(c) are met and it classifies as Type 4. The waste classification assesses the nitrate exposure of the waste rock as negligible, as it is exposed to a single flush only.

Condition 7(6)(a), on organics and pesticides, has not been tested for the waste rock. The waste classification considers the risk low, as the rock is unprocessed and has no contact with process reagents.

5.4 Run-of-Mine Ore

ROM1 exceeds TCT0 for cobalt, copper and nickel, and its leachable mercury of 0.013 mg/l exceeds LCT0 (0.006 mg/l) but not LCT1, so it classifies as Type 3. ROM2 has no leachable concentration above LCT0, but the ore is stockpiled on the RoM pad as one stockpile without segregation, so ROM2 is not disposed of without other waste, condition 7(6)(c) is not met and it also classifies as Type 3. Both samples have an intermediate acid forming potential. The RoM pad is designed for Type 3 ore.

5.5 Wastewater: Irrigation Dam

The North Pit sump water classifies as Type 3 in all six modelled scenarios. Nitrate exceeds LCT0 in every scenario, and in the low inflow scenario sulphate, chloride, fluoride and hexavalent chromium also exceed LCT0. No scenario exceeds LCT1 (Table 5-2). The nitrate is already present in the background groundwater before mining. The geochemical model does not include the generation of hexavalent chromium from the pit walls, the TSF or the WRD, so the modelled hexavalent chromium concentrations are a lower bound. Changes in the water once it stands in the irrigation dam, such as further evaporative concentration or the uptake of nitrate, are not modelled.

Table 5-2: Modelled North Pit Sump Water Quality Against the Leachable Concentration Thresholds

DETERMINANT	MODELLED RANGE (mg/l)	LCT0 (mg/l)	LCT1 (mg/l)
Nitrate as N	39 to 316	11	550
Sulphate as SO ₄	71 to 569	250	12 500
Chloride as Cl	51 to 416	300	15 000
Fluoride as F	0.42 to 3.40	1.5	75
Chromium (VI)	0.008 to 0.066	0.05	2.5
Nickel	0.001 to 0.007	0.07	3.5
Manganese	< 0.001	0.5	25

The irrigation dam also receives water from RWD Cell 3 through the RoM PCD (Chapter 6). The water in the RWD and the PCDs is treated as Type 3, because it comes from the same groundwater and pit dewatering sources and, in the RWD, is recirculated through the process. The water in the irrigation dam is therefore Type 3 whichever source governs.

5.6 Waste Type Determination

Section 7(6) of GN R635, as amended by GN 5522 of 2024, classifies waste in which all leachable concentrations of metal ions and inorganic anions are at or below LCT0 as Type 4, irrespective of the total concentrations, provided that all organics and pesticides are below the prescribed total concentration limits (condition (a)), the physical and chemical character of the waste is stable and will not change over time (condition (b)), and the waste is disposed of without any other waste (condition (c)). The Type 4 outcome is therefore conditional, and each material is assessed against all three conditions. The results for each material are summarised in Table 5-3.

Table 5-3: Waste Type per Material

MATERIAL	ABOVE TCT0	ABOVE LCT0	ACID FORMING POTENTIAL	WASTE TYPE
Rougher tails composite	Cu, Mn, Ni	None	Non-acid forming	Type 3 as deposited (solids Type 4)
ROM1	Co, Cu, Ni	Hg	Intermediate	Type 3
ROM2	Co, Cu, Ni	None	Intermediate	Type 3
Pyroxenite	Co, Ni	None	Non-acid forming	Type 4
Norite	Ba, Pb	None	Non-acid forming	Type 4
North Pit sump water	Not applicable	Nitrate in all scenarios; sulphate, chloride, fluoride and Cr (VI) in the low inflow scenario	Non-acid, pH above 7	Type 3

5.7 Barrier Class Requirement by Facility

The barrier class required for each facility follows from the waste type it contains (GN R636) and is given in Table 5-4. Where a facility handles more than one stream, the higher risk stream governs. The barrier systems adopted are described in Chapter 10.

Table 5-4: Barrier Class Required per Facility

FACILITY	GOVERNING STREAM	WASTE TYPE	BARRIER CLASS
TSF	Tailings as deposited and decant water	Type 3 (solids Type 4)	Class C
WRD	Waste rock	Type 4	Class D
RoM pad	RoM ore (ROM1 and ROM2, one stockpile)	Type 3	Class C
Irrigation dam	North Pit sump water and RWD Cell 3 water	Type 3	Class C
RWD Cells 1 to 3	TSF return water	Treated as Type 3	Class C
WRD, RoM and Plant PCDs, silt traps and dirty water channels	Dirty runoff	Treated as Type 3	Class C

6 WATER BALANCE

6.1 Water Balance Model

The water balance for the mine was prepared by Water Hunters as a dynamic computerised water balance simulation model, Technical Report 1023 (SX-TR0124 in the document history), version 1.2 final, dated May 2024. The report is attached as Annexure B. It updates the earlier model with the production forecast available in 2024 and with the groundwater inflows to the pits.

The approach follows the DWS guideline for water conservation and water demand management in the mining sector (2016). The flow diagram of the model is shown in Figure 6-1.

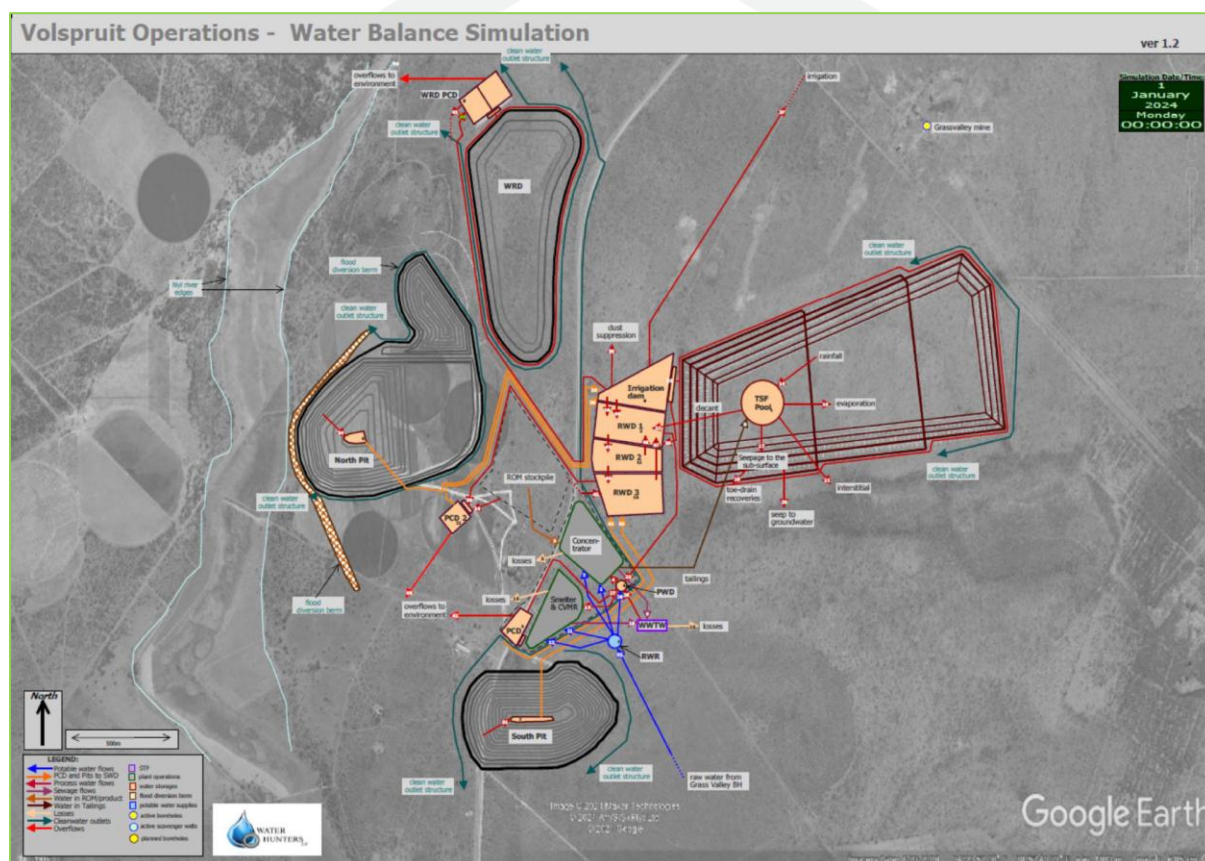


Figure 6-1: Water Balance Flow Diagram

6.2 Model Inputs

The inputs to the model that bear on the design of the residue and water management facilities are given in Table 6-1. The full input set, the flow logic and the production schedule are given in Annexure B. The last column gives the corresponding basis adopted for the design in this report. The water balance pre-dates this design. The production plan has changed since it was prepared, from 147 000 t/month of tailings in the water balance to the 243 000 t/month adopted for the design and may change again. The water balance is therefore used to check that the design capacities are not undersized, and not as the basis of the design.

Table 6-1: Water Balance Model Inputs

PARAMETER	VALUE ADOPTED IN THE WATER BALANCE	DESIGN BASIS IN THIS REPORT	EFFECT ON THE DESIGN
Simulation period	20 years, January 2024 to December 2043, on an hourly time step	The 20-year simulation covers the 14.6-year TSF life (Table 9-6)	Neutral
Production	Concentrator feed ramping up to 150 000 t/month, with 147 000 t/month of tailings to the TSF at steady state. North Pit for 9 years and 4 months, then the South Pit at the same rate. Smelter and refinery from January 2028	243 000 t/month of tailings to the TSF, giving a TSF life of 14.6 years (Table 9-6)	Conservative for spills (Table 6-4)
Plant water use	2.55 m ³ per tonne, with plant losses of 10% of the incoming water	As modelled. Not a design input for the facilities in this report	Neutral
Rainfall	Mean annual rainfall of 618.1 mm, from the Volspruit Stormwater Management Plan Rev CC1, generated stochastically about the monthly averages	Mean annual precipitation of 559 mm (section 3.8). The modelled 618.1 mm is higher, and conservative for the water volumes	Conservative
TSF pool	Minimum operating volume 10 000 m ³ , maximum operating volume 92 250 m ³ , maximum pool area 30 ha, total catchment 100.5 ha	Maximum operating pond of 50 000 m ³ (Table 9-6)	Neutral. The extra volume decanted to the RWD is within the additional RWD design capacity (Table 6-2)
TSF seepage and drainage	10 mm per day beneath the pool and part of the wet beach. All seepage is recovered by the toe drains apart from a negligible loss to groundwater	Toe drain outflow reports to the RWD through the solution trench (section 11.10)	Neutral
Interstitial losses	15% of the water delivered to the TSF with the tailings	As modelled	Neutral
Groundwater inflow to the pits	Fissure inflow rates from the 2013 specialist studies	The 2026 groundwater model (Annexure F) predicts combined pit inflows of about 4 000 m ³ /day at the start of mining, rising to 7 000 m ³ /day at the end, without grouting	Conservative. The modelled inflows are at or above the 2026 predictions
Pit dewatering	North Pit to the irrigation dam and South Pit to RWD Cell 3, at up to 400 m ³ /h and 500 m ³ /h respectively, which the water balance recommends be confirmed as cost effective and manageable	Irrigation dam and RWD Cell 3 design capacities (Table 14-1)	Neutral. An operating rate. The receiving dams are sized for storage (Table 6-2)
Dam seepage	Lined dams, with nominal	Lined dams, with the	Neutral

PARAMETER	VALUE ADOPTED IN THE WATER BALANCE	DESIGN BASIS IN THIS REPORT	EFFECT ON THE DESIGN
	seepage only	barrier systems in Table 10-2	
External supply	Grass Valley borehole, 10 L/s, for drinking and service water	Not applicable	Neutral

6.3 Water Storage Components

The storages modelled are compared with the design capacities in this report in Table 6-2. The process water dam and the raw water reservoir form part of the plant and are not part of this design. The design capacities of the Plant and RoM PCDs are those of the Ver 0.0 design. The WRD PCD has been enlarged to a design capacity of 63 345 m³, above the 63 340 m³ in the water balance, which is taken as its required capacity. The design capacities are confirmed in Chapter 14.

Table 6-2: Water Balance Storages and Design Capacities

STORAGE	WATER BALANCE CAPACITY (m ³)	WATER BALANCE MAXIMUM OPERATING VOLUME (m ³)	DESIGN CAPACITY (m ³)
TSF pool	500 000	92 250	50 000 (maximum operating pond)
RWD Cell 1	119 091	95 273	152 949
RWD Cell 2	116 613	93 290	188 092
RWD Cell 3	118 840	95 072	202 211
Irrigation dam	100 842	90 758	102 907
Plant PCD	35 095	30 000	38 097
WRD PCD	63 340	35 000	63 345
RoM PCD	20 900	16 000	20 900

6.4 Results

The average monthly sources and losses of water over the 20-year simulation are given in Table 6-3. Groundwater inflow to the pits is the largest source of water, followed by the water recovered from the TSF toe drains. The borehole supplies drinking and service water only, and no other make-up water is needed for the plant.

No water was discharged to the environment over the simulation. The operating rules route excess water from the pits and from RWD Cell 3, through the RoM PCD, to the irrigation dam, from where it is supplied to farmers for irrigation. Irrigation is the largest loss from the system. The report recommends that the simulated irrigation rate of 103 000 m³/month be confirmed as viable, and notes that there are no discharges to the environment at this irrigation rate. The absence of discharges therefore depends on irrigating this volume. Where the water is not suitable for irrigation, it is treated to the required standard before use or retained for reuse in the plant and on site, including dust suppression.

Table 6-3: Average Monthly Water Sources and Losses

ITEM	AVERAGE (m ³ /month)	SHARE OF TOTAL
Sources:		
Groundwater inflow to the pits	205 820	61%
Toe drain recoveries from the TSF	71 598	21%
Rainfall runoff	46 392	14%
Grass Valley borehole	8 119	2%
Moisture in the RoM ore	3 729	1%
Losses:		
Irrigation water to farmers	103 068	31%
Evaporation	64 514	19%
Plant losses	48 990	15%
Interstitial losses in the TSF	46 229	14%
Seepage to the TSF toe drains (recovered)	71 598	21%
Dust suppression	510	<1%
Overflows to the environment	0	0%

The water balance was run at 147 000 t/month of tailings, against the 243 000 t/month adopted for the design. The effect of the higher production rate was checked by scaling the tailings-related flows in the water balance in proportion to the tonnage, with the rainfall, evaporation and pit inflows unchanged (Table 6-4). The return water from the TSF to the RWD increases, but the plant demand drawn from the RWD increases in the same proportion, and the interstitial and plant losses increase with it. The system therefore has less surplus water at the higher production rate, which reduces the volume to be irrigated and the risk of a spill. The storm storage above the maximum operating level does not depend on production and is set by the design criteria in Chapter 9.

Table 6-4: Scaled Check of the Production Rate

ITEM (AVERAGE m ³ /month)	WATER BALANCE, 147 000 t/month	SCALED, 243 000 t/month
Slurry water to the TSF	308 000	509 000
Return water to the RWD, decant and toe drains	250 000	421 000
Plant water drawn from the process water dam	335 000	554 000
Interstitial and plant losses	80 000	133 000
RWD Cells 1 to 3, combined capacity (m ³)	354 544 (modelled)	543 252 (design)

Tailings-related flows scaled in proportion to the tonnage from the average annual flows in Annexure B, with the rainfall, evaporation and pit inflows unchanged.

6.5 Implications for the Design

The findings of the water balance that affect the design in this report are:

- the RWD design capacities exceed the modelled capacities, and the scaled check in Table 6-4 shows that the higher design production rate reduces the surplus water, so the design capacities of Cells 1 to 3 are not undersized;
- the dams did not spill over the simulation period while operated below their maximum operating levels, and on this basis the dam capacities are considered adequate for the 1:50-year storm. The largest simulated 24-hour rainfall is lower than the 1:50-year, 24-hour design depth in Table 3-2, so this finding is conditional on the re-run of the water balance described below;
- the average toe drain recovery provides a check on the drain outflow assumed in the solution trench design in section 11.10; and
- the irrigation dam receives water from RWD Cell 3 through the RoM PCD as well as pit water and is therefore lined with the same HDPE barrier as the RWD (Chapters 10 and 14).

Before construction, the water balance is to be re-run on the updated production plan, with the design capacities in Table 6-2 and the 2026 groundwater inflows (Annexure F). The re-run is to confirm the maximum operating level of each dam, so that the 1:50-year, 24-hour storm is held between the maximum operating level and FSL in line with the criterion in Table 9-1. Where this cannot be achieved, the operating levels, transfer rates or dam capacities are to be adjusted and the design updated before construction.

7 HYDROLOGY AND SURFACE WATER ASSESSMENT

7.1 Catchment Characteristics

The catchments within and around the Volspruit Mine, together with the clean water diversion controls, are shown in Figure 7-1.

The catchments were delineated on a composite surface of the natural ground survey and the design surfaces of the facilities, with the diversion and containment berms treated as catchment boundaries. Where an area drains to more than one channel, it is split at the divide so that each sub-catchment reports to a single receiving channel. The catchment characteristics and flood peaks are given in Table 7-1 for the clean water catchments and in



Table 7-2 for the dirty water catchments. The calculation sheets are attached in Annexure D.

The North Pit (D3) and South Pit (D6) catchments drain to the pit sumps and are contained within the pits, so no channel design flow arises from them. The top surface of the TSF (D1-TOP) drains to the penstock intakes. Its discharge is set by the penstock design in section 11.12 and enters DW CH 01B.

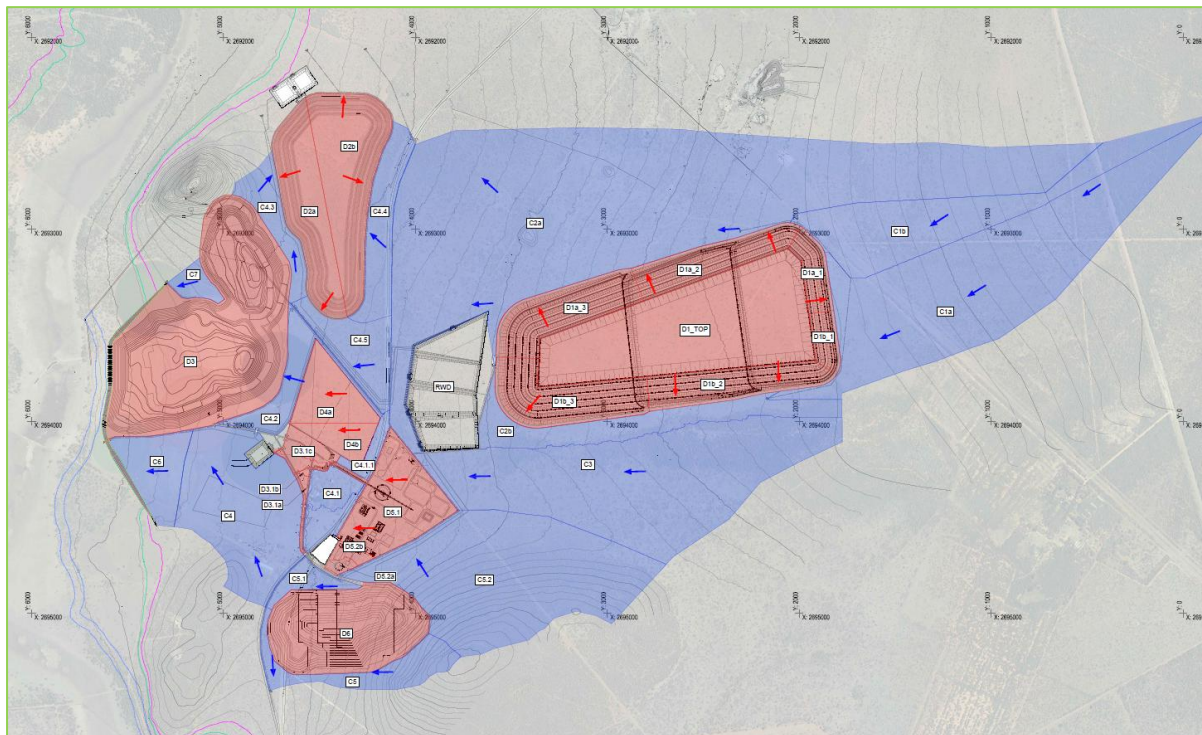


Figure 7-1: Site Catchment Delineation

7.2 Flood Peaks and Volumes

The flood peaks were calculated by the Rational Method in accordance with the SANRAL Drainage Manual (6th edition). Point rainfall for the time of concentration of each catchment is taken from the design rainfall depths in section 3.9, with an area reduction factor of 1.0. The rural runoff coefficients are those for a mean annual precipitation below 600 mm, consistent with the 559 mm adopted for the site. The dirty water catchments are compacted gravel surfaces carrying vehicle traffic. A heavy industrial urban runoff coefficient of 0.75, the middle of the SANRAL range for heavy industry, is applied to the plant area and to part of the RoM pad and its surrounds.

The time of concentration is the overland flow time over the first 100 m of the flow path, by the Kerby formula, plus the watercourse flow time over the remainder, by the US SCS formula, with a minimum of 15 minutes. The US SCS formula is applied to the concrete lined dirty water channels as well as to the natural watercourses.

The flood peaks for the delineated sub-catchments are included in Annexure D. The hydrological design criteria applied to each facility are set out in Chapter 9.

The peaks in Table 7-1 and

Table 7-2 are for each catchment on its own. The design flow in each channel and structure is the sum of the peaks of all the catchments upstream of it, for each return period. This is conservative, because the catchment peaks occur at different times of concentration. The design flows are given with each structure in the design chapters.

Table 7-1: Clean Water Catchment Characteristics and Flood Peaks

CATCHMENT	AREA (km ²)	T _c (min)	Q 1:2 (m ³ /s)	Q 1:10 (m ³ /s)	Q 1:50 (m ³ /s)	Q 1:100 (m ³ /s)
C1a	0.910	60	0.96	1.88	3.62	4.93
C1b	0.386	53	0.44	0.86	1.66	2.26
C2a	2.103	86	1.77	3.43	6.61	9.01
C2b	0.225	69	0.22	0.42	0.81	1.11
C3	0.876	83	0.75	1.45	2.80	3.81
C4	0.414	44	0.54	1.06	2.04	2.78
C4.1a	0.041	42	0.06	0.11	0.22	0.30
C4.1b	0.047	34	0.07	0.14	0.27	0.37
C4.1.1	0.028	40	0.04	0.08	0.16	0.22
C4.2	0.104	58	0.11	0.22	0.42	0.57
C4.3	0.093	66	0.10	0.19	0.36	0.49
C4.4	0.145	66	0.15	0.29	0.55	0.76
C4.5	0.133	51	0.16	0.31	0.60	0.82
C5	0.123	29	0.25	0.48	0.93	1.27
C5.1	0.044	39	0.06	0.12	0.24	0.32
C5.2	0.717	66	0.79	1.54	2.97	4.05
C6	0.128	41	0.17	0.34	0.65	0.89
C7	0.033	30	0.06	0.11	0.21	0.28

Table 7-2: Dirty Water Catchment Characteristics and Flood Peaks

CATCHMENT	AREA (km ²)	T _c (min)	Q 1:2 (m ³ /s)	Q 1:10 (m ³ /s)	Q 1:50 (m ³ /s)	Q 1:100 (m ³ /s)
D1a_1	0.057	18	0.40	0.74	1.15	1.37
D1a_2	0.134	22	0.84	1.55	2.42	2.88
D1a_3	0.171	20	1.13	2.08	3.25	3.87
D1b_1	0.039	15	0.31	0.57	0.89	1.06
D1b_2	0.148	24	0.89	1.63	2.54	3.02
D1b_3	0.189	23	1.16	2.12	3.31	3.94
D2a	0.211	51	0.69	1.26	1.97	2.34
D2b	0.282	47	0.95	1.75	2.72	3.24
D3.1a	0.024	29	0.16	0.27	0.40	0.46
D3.1b	0.008	17	0.07	0.13	0.19	0.22
D3.1c	0.032	17	0.29	0.50	0.74	0.86
D4a	0.109	40	0.57	0.98	1.44	1.67
D4b	0.054	17	0.50	0.85	1.24	1.44
D5.1	0.208	30	1.95	3.16	4.41	4.99
D5.2a	0.010	15	0.15	0.24	0.34	0.38
D5.2b	0.016	18	0.20	0.33	0.46	0.52
D1-TOP	0.610	Refer to section 11.11				
D3	0.775	Contained in the pit				
D6	0.325	Contained in the pit				

7.3 Flood Lines

Flood lines for the Nyl River at the site were determined by African Environmental Development under reference AED0151 Rev 00.a. The study was prepared by G Krige and verified by P Bond, and is attached as Annexure E. The study is dated 22 July 2010. EcoE has reviewed it for this submission against the current mine layout and considers the flood lines to remain applicable.

The critical storm duration and peak discharge for each return period are given in Table 7-3.

Table 7-3: Nyl River Design Flood Peaks

RETURN PERIOD	CRITICAL STORM DURATION (h)	PEAK DISCHARGE (m ³ /s)
1:50	7	886.7
1:100	7	1 133.2

The N1 Bridge governs flood levels through the study reach. The flood elevations it produces override the influence of the farm dam adjacent to the site and of any flood barrier constructed on the mine side of the river.

Both the 1:50 and 1:100-year flood lines extend into the North Pit area. The study modelled a flood barrier 1.765 km long on the mine bank. The barrier raised the 1:100-year flood level at cross-sections

9 and 10 by 0.05 m and 0.23 m, but these levels remain below the 1 063.15 mamsl produced by the N1 Bridge, and the study concludes that neither the barrier nor the removal of the farm dam has any impact on the elevation of the flood lines. The study notes that a barrier on one bank raises the flood line on the opposite bank, which belongs to a different landowner. Although the study did not model other flood scenarios, it considers it very likely that the 1:10 and 1:25-year flood lines would also intersect the barrier, which is therefore protected against scouring by flowing water on the side facing the river.

7.4 Clean and Dirty Water Catchment Delineation

There are six dirty areas: the North Pit, the South Pit, the TSF, the WRD, the RoM pad and the plant area. The two pits are self-containing in terms of dirty water. The dirty water areas generally drain in a western direction, and each area has a dedicated dirty water management system draining to its RWD/PCD.

In terms of GN 704 the clean water runoff arising within the catchment of the Volspruit Mine is required to be diverted and returned to the natural watercourse. Clean water diversion channels and berms intercept this runoff and discharge it to the northeast. The channels are designed in Chapter 17 and the dirty water collection system in Chapter 16.

The routing of each catchment through the channels, culverts and containment structures is shown in Figure 7-2 for the clean water system and in Figure 7-3 for the dirty water system.

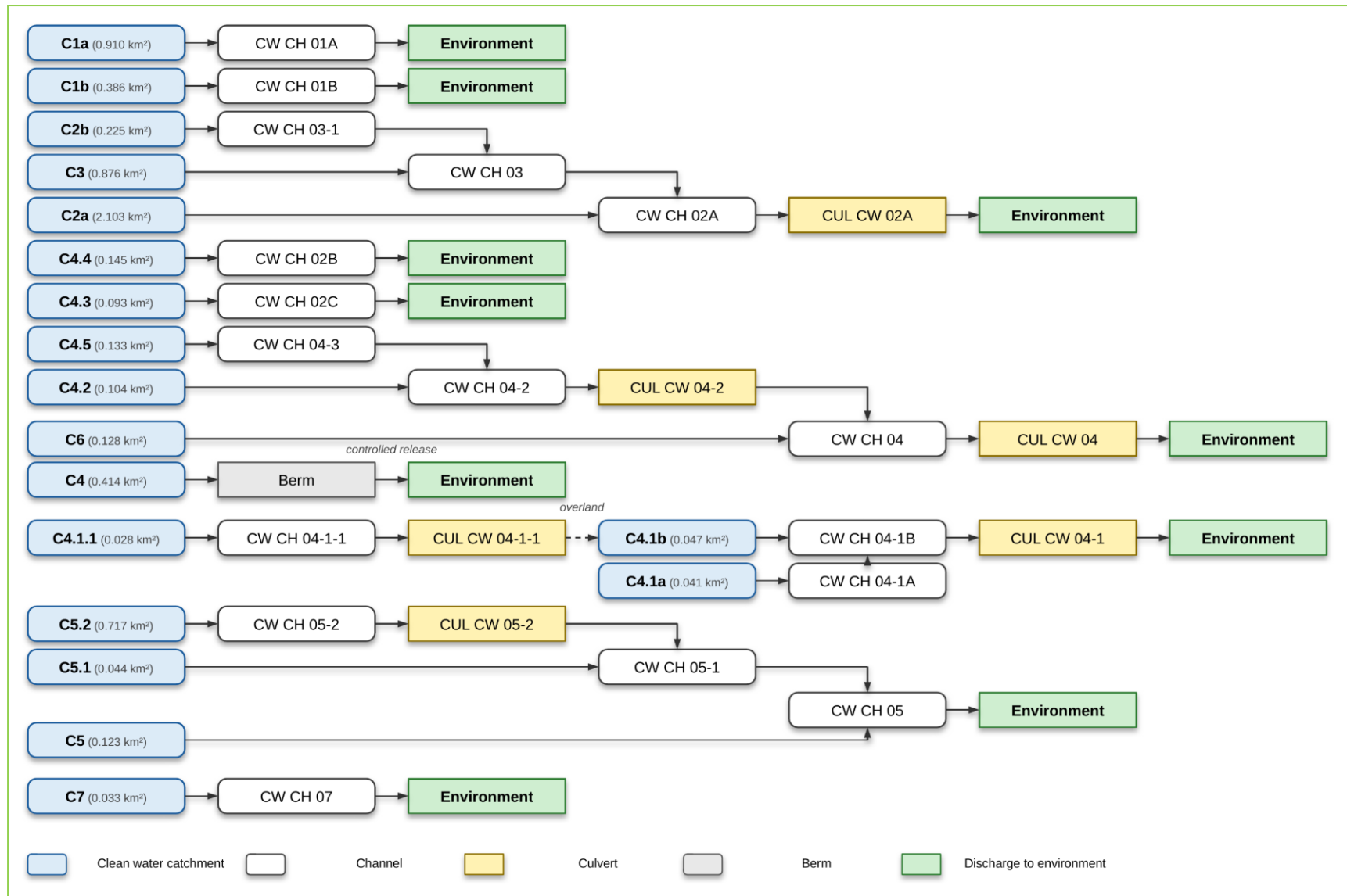


Figure 7-2: Clean Water Runoff Flow Diagram

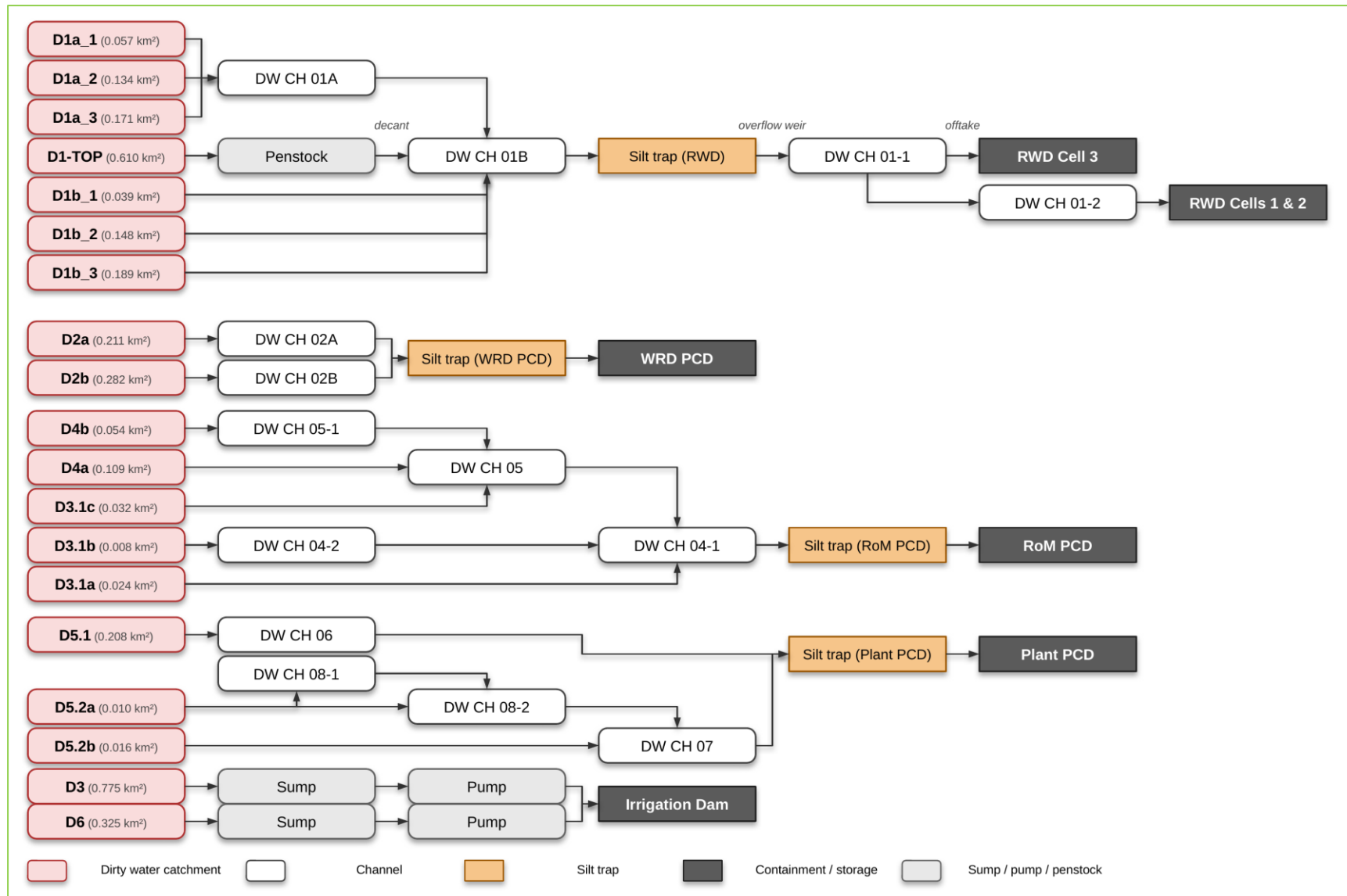


Figure 7-3: Dirty Water Runoff Flow Diagram

8 SAFETY CLASSIFICATION

8.1 Zone of Influence

The zone of influence is set in accordance with SANS 10286 and adjusted to the site topography and the expected behaviour of the PGM tailings.

A flow failure towards the northwest would reach the N1 highway, 1.5 km downstream, which is a heavily trafficked road, and part of the plant area to the west. The flow from the TSF is obstructed by the WRD, which limits the reach of the flow. The predominant wind direction has not been included in the consideration of the zone of influence.

The combined inundation zone of the tailings dam breach analysis (Chapter 22, drawing 19-918-DDR-DBA-801) confirms this zone of influence. It covers the RWD, the North and South Pits, the PCDs and the plant area, and extends along the Nyl River to the N1 at the river bridge. The extent along the river is indicative, as river flow was not modelled.

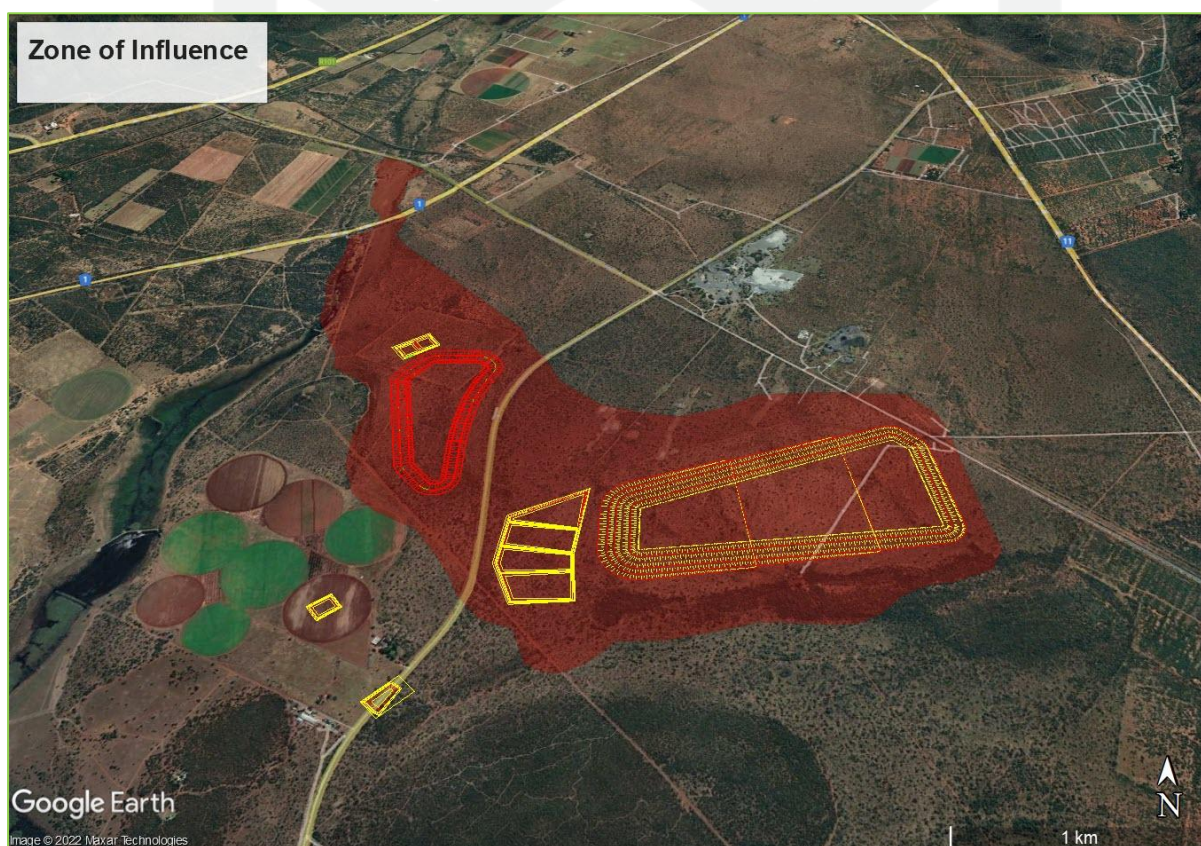


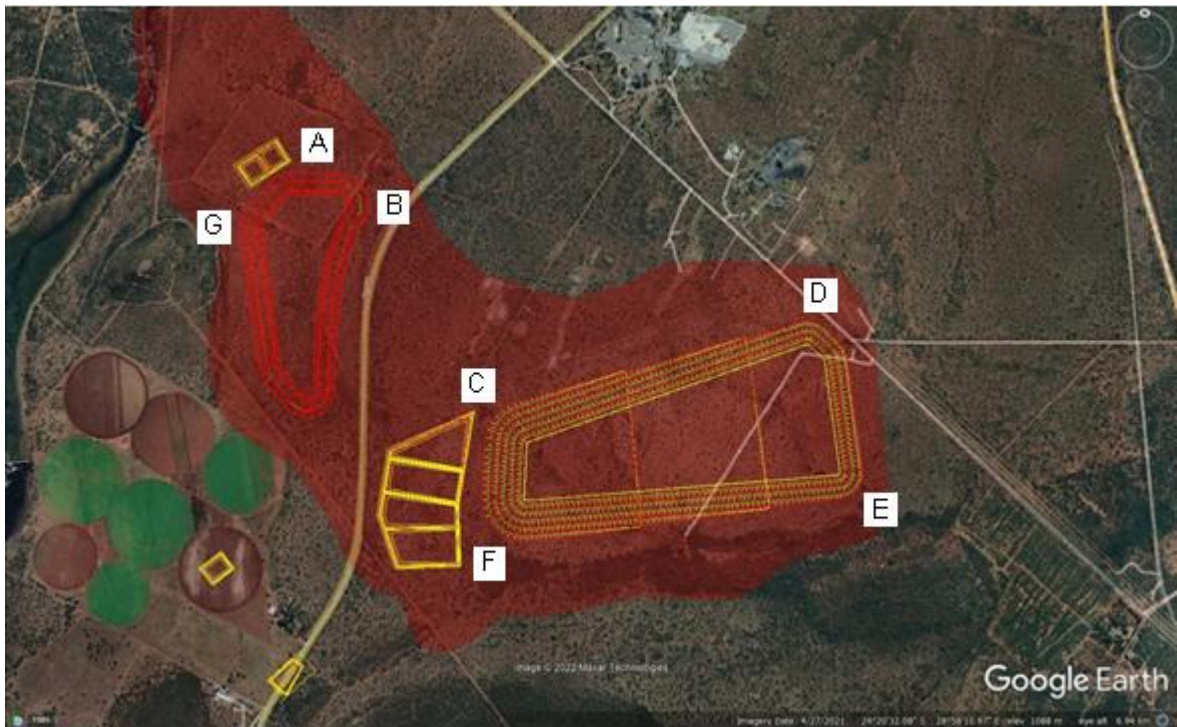
Figure 8-1: View of the Zone of Influence

8.2 Classification Information

The safety classification information required by SANS 10286 is given in Table 8-1.

Table 8-1: Safety Classification Information

1.1	Description of Residue Being Deposited
	PGM tailings.

1.2	Is Residue Deposited Hydraulically	<u>Yes</u>	No
1.3	Is the Deposit Still Active	<u>Yes</u>	No
1.4	If not, How Long Ago was the Deposit Decommissioned		
	N/A		
1.5	What will the ultimate height of the deposit be at each corner upon decommissioning		
	Figure showing corners, points of consideration and plan direction.		
			
	<p>Height of the various corners and points of consideration.</p> <p>A – 17 m</p> <p>B – 15 m</p> <p>C – 34 m</p> <p>D – 21 m</p> <p>E – 21 m</p> <p>F – 32 m</p> <p>G – 18 m</p>		
1.6	What is the current maximum height of the deposit	(New) 0	m
1.7	When did deposition start?	New dam	
1.8	What is the steepest overall outer slope of the deposit?	19	%
1.9	What is the steepest ground slope gradient? (Measured downstream of the deposit on the perimeter of the deposit, averaged over a distance of 200m from the toe)	1.05	%
1.10	Is the Deposit located on underground mining?	Yes	<u>No</u>
1.11	If yes, what is the shallowest depth to underground excavations?	N/A	m

8.3 Classification per SANS 10286

The safety classification criteria of SANS 10286 are given in Table 8-2 and the allocation for the TSF in Table 8-3. Applying the zone of influence in accordance with the guidelines, the TSF classifies as a high hazard facility.

Table 8-2: SANS 10286 Safety Classification Criteria

1	2	3	4	5
Nr of residents in Zone of Influence	Nr of workers in Zone of Influence ^a	Value of property in Zone of Influence ^b	Depth to underground mine workings ^c	Classification
0	<10	0 - R 5 mil	> 200 m	Low hazard
1-10	11-100	R 5 – R 50 mil	50 – 200 m	Medium hazard
>10	>100	> R 50 mil	< 50 m	High hazard

^a Not Including workers employed solely for the purpose of operating the deposit.

^b The value of third-party property should be the replacement value in 1996 terms.

^c The potential for collapse of the residue deposit into the underground workings effectively extends the zone of influence to below the ground level. The depth is that directly below the footprint of the mine residue deposit.

Table 8-3: Safety Classification Using the Zone of Influence

CRITERION	ALLOCATION	NOTES
Number of residents in the zone of influence	Medium	N1 road users at the Nyl River bridge
Number of workers in the zone of influence	High	Pit crews, RWD and PCD operators, and plant personnel, all within the inundation zone (19-918-DDR-DBA-801)
Value of third-party property in the zone of influence	High	Plant, PCDs, N1 bridge and Nyl River
Depth to underground mine workings	Low	No underground mining beneath the TSF (section 3.4)

Using the Zone of Influence Appendix A, as per the Guidelines, the classification according to Table 8-2 above from SANS 10286 Revision No.2 is:

High

8.4 Classification per the Global Industry Standard on Tailings Management Consequence Classification Matrix

The consequence classification of the TSF against the GISTM consequence classification matrix is given in Table 8-4. The classification is high level.

Table 8-4: GISTM Consequence Classification Matrix for the TSF

DAM FAILURE CONSEQUENCE CLASSIFICATION	POTENTIAL POPULATION AT RISK	POTENTIAL LOSS OF LIFE	ENVIRONMENT	HEALTH, SOCIAL AND CULTURAL	INFRASTRUCTURE AND ECONOMICS
Low					
Significant					
High	X	X			
Very high			X	X	X
Extreme					

Consequence classification for the upstream side at final height.

PART B — DESIGN BASIS



9 DESIGN CRITERIA

9.1 Design Criteria by Facility

The hydrological design criterion adopted for each facility is given in Table 9-1. The clean and dirty water systems are designed so that they do not spill into one another more than once in fifty years.

Table 9-1: Design Criteria by Facility

FACILITY	REQUIREMENT	SOURCE	ADOPTED
Temporary works	Conveyance of the 1:2-year flood peak during construction	Design standard adopted for temporary diversion structures	1:2-year flood peak
TSF	1:100-year, 72-hour drawdown of the stormwater storage held above the maximum operating pond, through the penstock system	ANCOLD Guidelines on Tailings Dams (2012), Table 4, extreme storm storage allowance	1:100-year, 72-hour drawdown
Solution trench	Conveyance of the 1:50-year flood peak from the TSF side slopes together with the penstock outflow and the drain outflow	GN 704, regulation 6, for the side slope runoff. The penstock outflow is set by the TSF drawdown criterion. The drain outflow is taken as the full-bore capacity of the drain outfall pipes	1:50-year flood peak plus penstock and drain outflow
Waste rock dump	Separation of clean and dirty water, with the runoff from the facility footprint contained and conveyed to the WRD PCD	GN 704, regulation 6	1:50-year flood peak
Run-of-mine pad	Separation of clean and dirty water, with the runoff from the pad contained and conveyed to the RoM PCD	GN 704, regulation 6	1:50-year flood peak
RWD, irrigation dam, Plant PCD, WRD PCD and RoM PCD	1:50-year, 24-hour storm capacity above the maximum operating level, with a minimum freeboard of 0.8 m	GN 704, regulation 6	1:50-year, 24-hour
Silt traps	Retention of the solids entrained in the dirty water collection channels before the water reports to a PCD	GN 704, regulation 6	1:10-year flood peak
Dirty water collection channels	Conveyance of the 1:50-year flood peak without overflowing	GN 704, regulation 6	1:50-year flood peak
Clean water diversion channels	Conveyance of the 1:50-year flood peak without overflowing, with erosion protection	GN 704, regulation 6. SANS 10286, 10.4.11.3, requires the 1:100-year peak discharge for drains diverting runoff around a residue deposit	1:50-year flood peak
Culverts and crossings	Conveyance of the 1:50-year flood peak without surcharge	GN 704, regulation 6	1:50-year flood peak

FACILITY	REQUIREMENT	SOURCE	ADOPTED
Flood diversion berm	Exclusion of flood water from the North Pit. The berm does not impound water and does not trigger the dam safety regulations	GN 704, regulation 4	1:100-year flood line

9.2 Freeboard

Freeboard is taken as the vertical difference between the level of the lowest point on the perimeter of the outer wall and the level of the supernatant water on the upper surface of the facility. The freeboard adopted is built up from the maximum operating pool depth, the stormwater storage required above the maximum operating level, and the dry freeboard required above full supply level (FSL). The FSL is the level reached by the design storm on top of the maximum operating level.

The components of the freeboard criterion are given in Table 9-2. The stormwater storage is taken as the 1:100-year, 72-hour event in accordance with the ANCOLD Guidelines on Tailings Dams, which is the more onerous of that event, the 1:100-year, 24-hour event of SANS 10286 and the once in fifty-year spill criterion of GN 704. GN 704 requires a dry freeboard of 0.8 m above FSL. The dry freeboard adopted for the TSF is given in the chapter for the facility. The criterion is maintained throughout the operating life of the facility, including during commissioning.

Table 9-2: TSF Freeboard Criteria

COMPONENT	REQUIREMENT	SOURCE	ADOPTED
Maximum operating pool depth	Not prescribed	Design limit adopted for the facility	0.3 m
Stormwater storage above the maximum operating level	1:100-year, 72-hour event	ANCOLD Guidelines on Tailings Dams (2012), Table 4	1:100-year, 72-hour event
Minimum dry freeboard above FSL	0.8 m	GN 704, regulation 6, for a dam or tailings dam forming part of a dirty water system	0.8 m

9.3 Embankment Stability and Earthquake Criteria

The earthquake loading is taken as an operating basis earthquake of 0.08 g at a 1:475-year event. The loading conditions and minimum factors of safety adopted for the TSF are given in Table 9-3. Damage and deformation are permitted within the freeboard allowance, with no release of tailings or water.

Table 9-3: TSF Loading Conditions and Minimum Factors of Safety

SCENARIO	LOADING CONDITION	DESCRIPTION	MINIMUM FACTOR OF SAFETY
1	After construction, before deposition, peak drained and undrained strength conditions	Short-term stability at the end of construction before deposition has started	1.3
2	Long-term, peak drained	Long-term stability once steady-state	1.5

SCENARIO	LOADING CONDITION	DESCRIPTION	MINIMUM FACTOR OF SAFETY
	strength conditions, steady-state seepage	seepage has been achieved during operation and closure, with no rapid change in the phreatic surface or dam geometry	
3	Short-term, peak undrained strength	Short-term stability where a trigger event other than an earthquake causes the tailings and in-situ materials to behave in an undrained manner	1.3
4	Earthquake, pseudo-static, peak undrained strength	Ground acceleration applied as a pseudo-static force, with peak undrained strength assumed for saturated contractive fine-grained materials	1.1
5	Post-liquefaction, residual undrained strength	Stability after a flow liquefaction event, with saturated contractive materials reduced to residual strength	1.1

The loading conditions and minimum factors of safety adopted for the HDPE lined dams, the Plant PCD and the flood diversion berm are taken from the Dam Safety Guidelines of the Canadian Dam Association (CDA, 2013) and are given in Table 9-4.

Table 9-4: Dam and Flood Diversion Berm Loading Conditions and Minimum Factors of Safety (CDA, 2013)

SCENARIO	LOADING CONDITION	DESCRIPTION	MINIMUM FACTOR OF SAFETY
1	After construction, before filling (peak drained and undrained strength)	Short-term stability at the end of construction, before the dam is filled. Both drained and undrained strength parameters are considered, particularly for the foundation materials.	1.30
2	Long-term (peak drained strength, steady-state seepage)	Long-term stability once steady-state seepage is reached during operation and closure, with drained effective strengths and no rapid change in the phreatic surface or dam geometry.	1.50
3	Earthquake, pseudo-static (peak undrained strength)	The ground acceleration is applied to the facility as a pseudo-static force.	1.10
4	Post-liquefaction (residual undrained strength)	Stability after a flow liquefaction event, with the strengths of all fully saturated contractive materials reduced from peak to residual values.	1.10
5	Flood loading (flood diversion berm only)	The 1:100-year flood level, taken 1 m below the berm crest, with steady-state seepage through the berm. Adopted for this short-term condition, which the CDA guidelines do not define.	1.30
6	Rapid drawdown (flood)	The river falls rapidly from the flood level, with	1.20

SCENARIO	LOADING CONDITION	DESCRIPTION	MINIMUM FACTOR OF SAFETY
	diversion berm only)	the pore pressures of the flood loading condition as the starting point, on the river side slope.	

9.4 Available Mineral Resources, Life of Mine and Polluting Period

The life of mine and service life considerations adopted for the design are given in Table 9-5.

Table 9-5: Life of Mine and Service Life Considerations

ASPECT	VALUE
TSF operating period	14.6 years
TSF polluting period	44.6 years (14.6-year operating period plus 30 years post closure)
Anticipated leachate temperature range	Not applicable, the tailings do not generate heat. The surface temperature of the liner is not expected to exceed 35 °C, as the geomembrane is covered by slimed fine tailings under the inverted barrier.

9.5 Design Parameters and Tonnages

The design parameters and tonnages adopted for the facilities are given in Table 9-6. The TSF is not designed to accommodate external water inflows or to function as a water holding dam, other than for the 72-hour drawdown after a storm event. Any additional inflow requires a review of the water management impacts before discharge.

Table 9-6: Design Parameters and Tonnages

PARAMETER	VALUE
Tailings production rate	243 000 tonnes per month of dry tailings
TSF deposition method	Hydro-cyclone with a spray bar connected to the overflow
Maximum rate of rise above the starter wall	4.0 m per year
Minimum operating pond	10 000 m ³ , or an operating pool depth of 300 mm
Maximum operating pond	50 000 m ³
WRD deposition method	End-tipped by haul trucks in lifts, each lift completed and benched before the next is started

10 BARRIER DESIGN FRAMEWORK

10.1 Applicable Legislation

The barrier systems are designed in terms of the National Norms and Standards for the Disposal of Waste to Landfill, GN R636 of 23 August 2013 (Government Gazette 36784), as amended by GN 5523 of 7 November 2024 (Government Gazette 51521). The amendment replaced the prescribed barrier layouts in regulation 3(1), substituted regulation 3(2)(c) and added regulation 3(2)(j). Regulation 4(1) assigns the barrier class from the waste type determined under GN R635, and regulation 3(1) sets out the prescribed barrier for each class.

Regulation 3(2) lists the containment barrier requirements that must be addressed in an application for a waste management licence. Table 10-1 gives the section of this report in which each requirement is addressed.

Table 10-1: GN R636 Regulation 3(2) Compliance

REF	REQUIREMENT OF GN R636 REGULATION 3(2)	REPORT SECTION
3(2)(a)	Design reports and drawings certified by a registered professional civil engineer prior to submission to the competent authority	Document control page, section 2.3 and Annexure J
3(2)(b)	Service life of the containment barrier quantified, taking account of temperature effects	Section 10.6, Table 9-5 and Table 10-5
3(2)(c)	Total solute seepage, inorganic and organic, calculated in determining the acceptable leakage rates and the action leakage rates, including seepage through discontinuities and diffusion through intact areas of the geomembrane	Section 10.9, Table 10-6 and Table 10-7
3(2)(d)	Alternative elements of proven equivalent performance, including clay components replaced by geomembranes or geosynthetic clay liners (GCLs)	Section 10.8, section 11.6 and section 14.5
3(2)(e)	Drainage layers contain drainage pipes of adequate size, spacing and strength to maintain atmospheric pressure in the drainage layer for the service life	Section 11.9, section 12.4 and Chapter 14
3(2)(f)	Alternative barrier layouts on slopes steeper than 1V:4H, provided equivalent performance is demonstrated	Section 14.4 and section 14.6, for the 1V:3H inside slopes of the HDPE lined dams
3(2)(g)	Construction quality assurance during construction	Chapter 20 and Annexure I
3(2)(h)	Geosynthetic materials comply with the relevant SANS specifications, or a management practice or standard that ensures equivalent performance	Sections 10.4 and 10.5
3(2)(i)	Compatibility of the liner materials with the waste stream, in particular clay soils exposed to waste containing salts	Chapter 5 and section 10.6
3(2)(j)	Guideline for Pollution Control Barrier System Design, available on SAWIC	Chapter 10
3(1), Note 10	In-situ soil foundation to incorporate a sub-surface drainage system, and to maintain a 2 m unsaturated zone between the base of the liner system and the wet season high	Section 10.2, Table 10-2 and the facility chapters

REF	REQUIREMENT OF GN R636 REGULATION 3(2)	REPORT SECTION
	groundwater level	

10.2 GN R636 Prescribed Barrier Systems

Type 3 waste is to be disposed of on a Class C barrier and Type 4 waste on a Class D barrier. The prescribed Class C barrier and Class D foundation, as amended by GN 5523, are shown in Figure 10-1 and Figure 10-2. The Class D foundation has no liner.

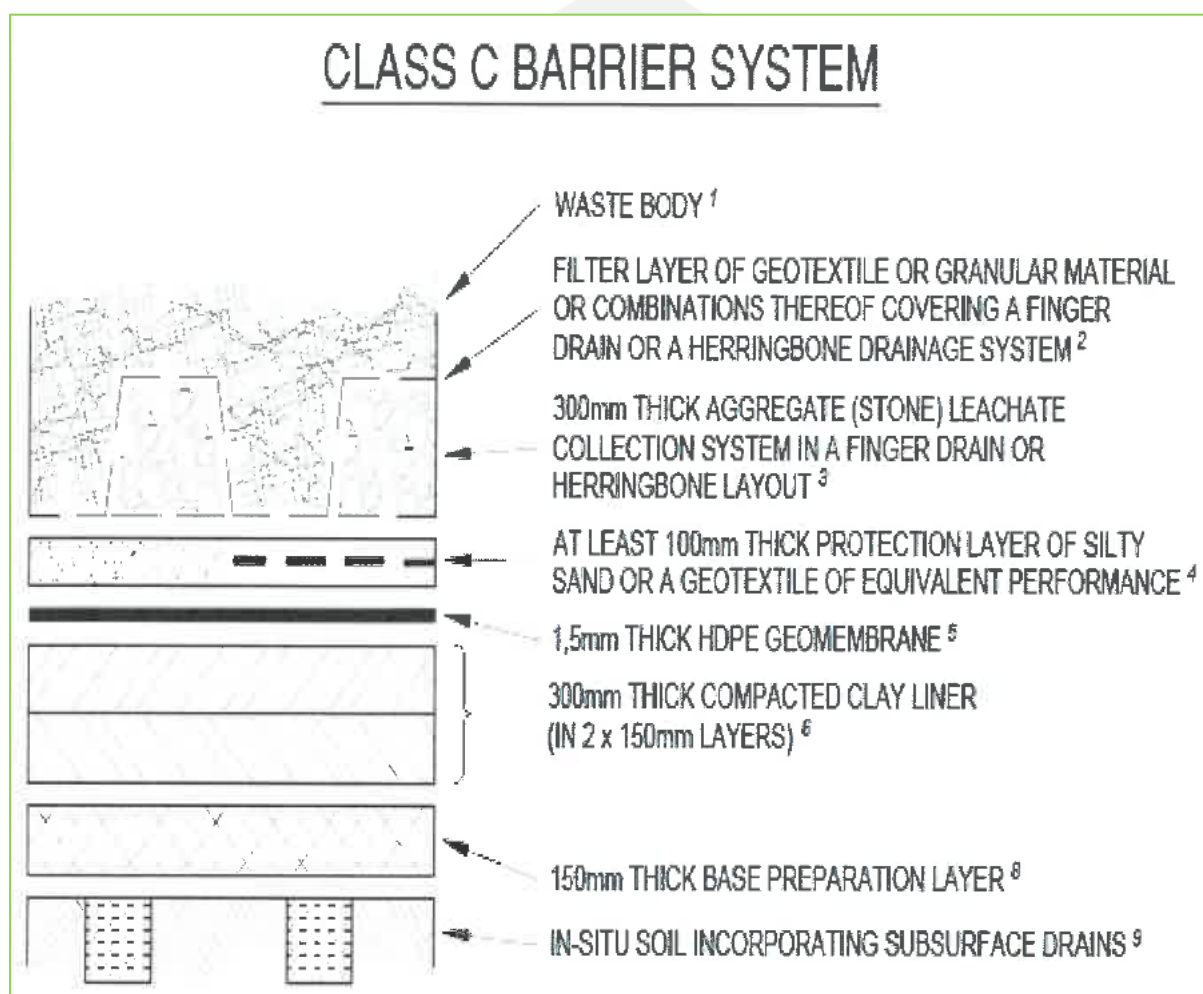


Figure 10-1: GN R636 Class C Barrier Requirements (Extract from GN R636, as amended by GN 5523)

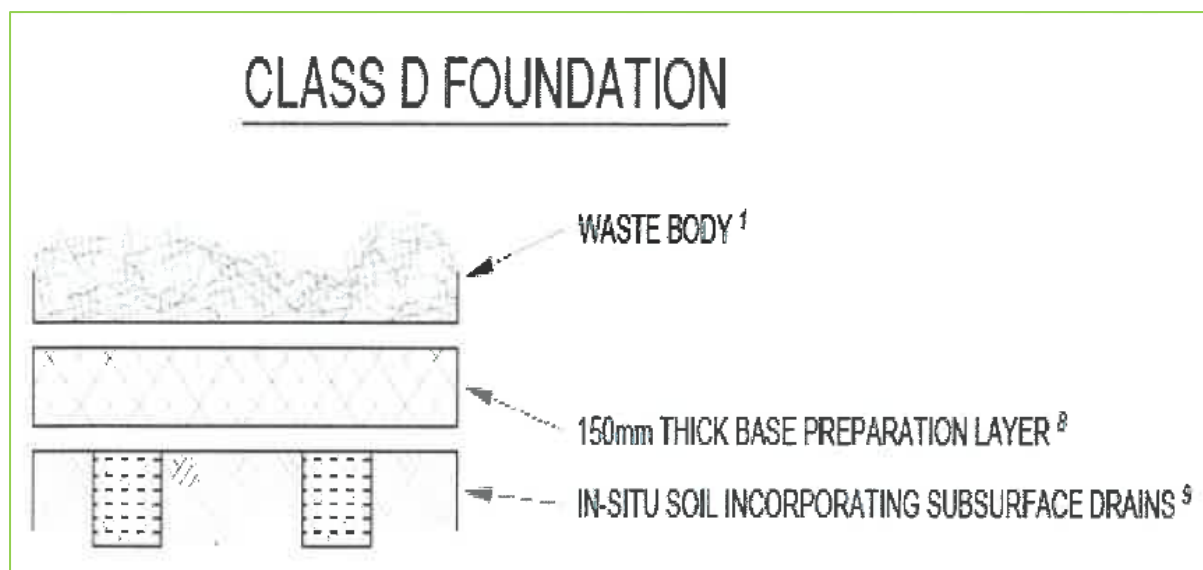


Figure 10-2: GN R636 Class D Barrier Requirements (Extract from GN R636, as amended by GN 5523)

For both barrier classes, the amendment requires the in-situ soil foundation to incorporate a sub-surface drainage system, to prevent uplift pressure from a fluctuating groundwater level, and to maintain an unsaturated zone of 2 m between the base of the liner system and the wet season high groundwater level (Note 10 to regulation 3(1)). The foundation material properties are included in the stability assessment of each facility in this report. Where a facility departs from the prescribed layout, its barrier is motivated as a Class C equivalent barrier under regulation 3(2)(d).

10.3 Barrier Class per Facility

The waste types are determined in Chapter 5. The water in the RWD, the PCDs and the dirty water collection systems is treated as Type 3. The barrier class and barrier system adopted for each facility are given in Table 10-2.

Table 10-2: Barrier Class and Barrier System per Facility

FACILITY	WASTE TYPE	BARRIER CLASS	BARRIER SYSTEM
TSF	Type 3; solids Type 4 (Chapter 5)	Class C equivalent	Inverted barrier: 1.5 mm double textured HDPE geomembrane on a receiving face of imported fine selected earth fill, or of compacted reclaimed tailings in cells 2 and 3, with the fine tailings slimed above it (sections 11.5 and 11.6)
WRD	Type 4	Class D	150 mm base preparation layer over the in-situ soil foundation, with a sub-surface drainage system in the foundation as required by GN 5523 (Chapter 12)
RoM pad	Type 3 (ROM1 and ROM2)	Class C equivalent	Reinforced concrete slab on stabilised layerworks with sub-surface drainage (Chapter 13)

FACILITY	WASTE TYPE	BARRIER CLASS	BARRIER SYSTEM
RWD Cells 1 to 3, irrigation dam, WRD PCD and RoM PCD	Irrigation dam Type 3; RWD and PCDs treated as Type 3	Class C equivalent	2 mm HDPE geomembrane on a SANS 10409 (2020) receiving face, under a soilcrete cover, with a sub-surface drainage system in the foundation (section 14.4)
Plant PCD	Treated as Type 3	Class C equivalent	Reinforced concrete, water retaining, on G7 layerworks, with a sub-surface drainage system in the foundation (Chapter 14)
Silt traps	Treated as Type 3	Class C equivalent	Reinforced concrete, water retaining, with a sub-surface drainage system in the foundation (Chapter 15)
Dirty water channels and solution trenches	Treated as Type 3	Class C equivalent	Concrete lined, for water conveyance (Chapter 16)

10.4 Material Specifications

The specification of the barrier materials is given in Table 10-3. The geosynthetic materials comply with the relevant SANS specifications, as required by regulation 3(2)(h). Compliance is verified under the construction quality assurance plan (Annexure I).

Table 10-3: Barrier Material Specifications

MATERIAL	SPECIFICATION
HDPE geomembrane	<p>Manufactured to SANS 1526:2015 and GRI-GM13. Selected and installed in accordance with SANS 10409:2020.</p> <p>Flat die cast, with a high-pressure oxidative induction time of not less than 600 minutes.</p> <p>Welds tested in accordance with GRI-GM19.</p> <p>Double textured 1.5 mm beneath the TSF. Smooth 2 mm beneath the HDPE lined dams.</p>
Soilcrete and cement stabilised material	<p>Minimum unconfined compressive strength of 5 MPa at 7 days (SANS 3001-GR53), for the geocell infill and ballast of the HDPE lined dams, the anchor trench backfill and the culvert pipe encasement (19-918-SWM-900). Hydraulic conductivity of the geocell infill and ballast of the HDPE lined dams not more than 1×10^{-7} m/s, tested on the approved mix in accordance with ASTM D5084 before placement.</p>
Drainage pipes	<p>Solid and drilled pipes: HDPE PE100 to SANS 4427-2, PN12.5, joined by butt fusion welding.</p> <p>Sub-surface drains below the barriers: Ø110 mm corrugated slotted HDPE, class SN16 (ring stiffness 16 kN/m² to ISO 9969) or similar approved, joined with the pipe manufacturer's couplers.</p> <p>Pipe types and diameters are given in the facility chapters.</p>
Drainage aggregate and filter sand	<p>Grading to the filter bands set by the filter compatibility criteria in section 10.10, on the gradings of the tailings and the founding soil.</p>

10.5 Geotextile Specification

All geotextiles are non-woven, continuous filament, needle punched polyester, except the TSF toe drain cover and basin leachate drain wrap, which are in contact with the tailings and are polypropylene. They are laid with a 300 mm heat bonded overlap, meeting or exceeding GRI-GT13(b) and SANS 12956, SANS 11058, SANS 13433, SANS 1525, SANS 12236, SANS 9863, SANS 13348 and SANS 12224. The geotextile applications are given in Table 10-4.

Table 10-4: Geotextile Applications

APPLICATION	LOCATION	MASS PER UNIT AREA (g/m ²)
Filter wrap and separation	TSF sub-surface drains below the geomembrane (19-918-SWM-466) WRD sub-surface drains below the footprint (section 12.4) Sub-surface drains beneath the dams and the silt traps (19-918-SWM-906 and 19-918-SWM-907) Beneath the Reno mattresses at the dam spillways (19-918-SWM-212) Beneath the dump rock at the clean water channel outlets (19-918-SWM-903) RoM pad pavement layerworks (19-918-SWM-904) Culvert inlet and outlet protection (19-918-SWM-905)	210
Filter wrap	WRD drainage and containment berm (19-918-SWM-152)	270
Separation, filter cover and protection	Beneath the 800 mm transition layer on the TSF confinement walls (19-918-SWM-451) TSF toe drain cover and basin leachate drain wrap, beneath the coarse tailings (19-918-SWM-456 and SWM-468) Beneath the barrels of the penstock walkway (19-918-SWM-461)	550
Protection	Geomembrane beneath the TSF toe drains and basin leachate drains (19-918-SWM-456 and SWM-468), and beneath the hydro-cyclones on the heel wall crest	1 200

10.6 Liner Longevity and Service Life of Geosynthetic Components

Covered HDPE geomembranes have a service life of more than 100 years at moderate temperatures of 20 °C to 30 °C (Koerner, 2012). The TSF geomembrane is covered by the tailings and the dam geomembranes by the soilcrete geocells and ballast, so they are protected from ultraviolet light, and the tailings do not generate heat. HDPE is resistant to the dilute solutions expected in the TSF and the dams (Chapter 5), and no clay component is used in any barrier, which addresses the compatibility of the liner materials with the waste stream (regulation 3(2)(i)).

The service life considerations for the TSF and the HDPE lined dams are given in Table 10-5, in the order of the DWS checklist. The geosynthetic components are expected to outlast the polluting period of each facility.

Table 10-5: Service Life Considerations

ASPECT FOR CONSIDERATION	TSF	HDPE LINED DAMS
Polluting period	44.6 years (14.6-year operating period plus 30 years post closure)	44.6 years (14.6-year operating period plus 30 years post closure)
Operating period	14.6 years	14.6 years
Anticipated leachate temperature	Not expected to exceed 35 °C, as the geomembrane is covered by slimed fine tailings	Not expected to exceed 35 °C, as the geomembrane is covered by the soilcrete geocells and ballast
Total tensile strain in the geomembrane	Limited to less than 3%, confirmed by an in-situ field test on the materials used in the works (section 11.7)	Not significant: the geogrid carries the downslope load of the cover, and the soilcrete strip limits strain at the make-off to concrete structures (section 14.6)
Service life of the drainage system materials	HDPE pipes a minimum of 50 years; geotextiles more than 100 years where not exposed	HDPE pipes a minimum of 50 years; geotextiles more than 100 years where not exposed
Service life of the liner system	More than 100 years, covered	More than 100 years, covered

10.7 Electrical Leak Location Survey Specification

An electrical leak location survey is carried out on every geomembrane by a specialist who is independent of the geomembrane installer, and the facility is not commissioned until all defects found have been repaired and re-tested, so that there are no known leaks at the end of construction. The survey methods, selected in accordance with ASTM D6747, are:

- TSF basin, where the geomembrane remains exposed until the tailings are placed: arc testing in accordance with ASTM D7953, after installation and before deposition starts; and
- HDPE lined dams: a dipole survey in accordance with ASTM D8265 or ASTM D7007, after the 5 MPa soilcrete geocell infill and ballast have been placed over the geomembrane.

The soilcrete geocell infill and ballast over the dam geomembranes are placed during the cooler part of the day, to limit wrinkling of the geomembrane under the cover. No plant runs directly on the geomembrane.

10.8 Inverted Barrier System, Basis and Motivation

Regulation 3(2)(d)(iii) of GN R636 allows the clay component of a prescribed barrier to be replaced by a geomembrane or a GCL where equivalent performance is proven. The inverted barrier adopted beneath the TSF uses this provision and is motivated as a Class C equivalent barrier.

In a conventional composite barrier, leakage is governed by holes that coincide with wrinkles in the geomembrane over the CCL (Rowe, 2012). The inverted barrier places the geomembrane on a SANS 10409 (2020) receiving face of imported fine selected earth fill and slimes the fine tailings directly above it, so that leakage through a hole is controlled by Darcian flow through the low permeability tailings above the hole (Rowe et al., 2017; Chou et al., 2022).

A single 42-week laboratory test on a 10 mm hole in a geomembrane beneath fine-grained tailings measured a leakage of 0.16 l/d at a pore pressure of 200 kPa, rising to only 0.46 l/d at 800 kPa, with no evidence of seepage-induced migration of fines within the tailings. On that basis, the leakage from a tailings storage facility containing fine-grained tailings can be limited to as little as 1 l/ha/d to 2 l/ha/d with a geomembrane liner containing up to five 10 mm holes per hectare, against three to four orders of magnitude more for an unlined facility (Chou et al., 2022). This estimate assumes that the tailings are the same as those tested, that the tailings and the material beneath the geomembrane are filter compatible, that the holes do not coincide with wrinkles and that the pond depth is less than 30 m. Where the material beneath the geomembrane is not filter compatible, it can intrude into the hole and increase the leakage, especially at low consolidation stress, with the effect decreasing as the tailings consolidate (Fan and Rowe, 2022). The receiving face beneath the TSF geomembrane is therefore finished smooth with a maximum particle size of 3 mm (section 11.5), and its filter compatibility with the fine tailings is confirmed by grading tests on both materials before the geomembrane is installed (section 10.10).

The system also removes the low strength clay layer, and its interface with the geomembrane, from beneath the outer wall zone, which may improve the stability of the confinement walls, and it removes the reliance on scarce suitable clay (Legge and Associates, 2024). The supporting literature, including Mnisi and Bester (2018) and the presentation of the barrier to the Department (Legge, 2024), is attached as Annexure H. The equivalence of the leakage performance is demonstrated in section 10.9.

10.9 Total Solute Seepage

The total solute seepage through each barrier is calculated as required by regulation 3(2)(c) and sets the acceptable leakage rate and the action leakage rate. Seepage through holes in the geomembrane governs. Diffusion through the intact geomembrane is not significant where the volatile organic compounds in the contained water are below 0.03 M/l (GN 5523).

Leakage through the TSF inverted barrier is calculated for flow through the fine tailings into a circular hole in the geomembrane, by the equation of Wissa and Fuleihan (1993). The equation matched the leakage measured in the tests described in section 10.8 to within 4% (Chou et al., 2022). It ignores the resistance of the receiving face beneath the geomembrane, which is conservative. The tailings are taken at a hydraulic conductivity of 5×10^{-8} m/s, based on project experience with PGM tailings from similar projects, processed in the same way. The adopted value is above the median of in-situ piezocone tests on two such facilities, and is conservative for flow into a hole, which is controlled by the lower vertical hydraulic conductivity. Five holes of 5 mm diameter per hectare are allowed for in the 1.5 mm geomembrane (Rowe, 2012). The head is taken as the saturated thickness of the tailings plus the 0.3 m operating pool, with no credit for the basin drains above the geomembrane. The acceptable leakage rate is calculated at the average head and the action leakage rate at the maximum head. The inputs and the calculation are given in Table 10-6. The hydraulic conductivity of the tailings is confirmed by tests on representative tailings samples before construction (section 11.14.1), and the leakage rates are recalculated with the measured value.

For comparison, the leakage through the prescribed Class C barrier is calculated by the equation of Rowe (2012) for a 1.5 mm geomembrane on a 300 mm compacted clay liner at 1×10^{-9} m/s, with 100 m of connected wrinkle per hectare, 0.2 m wide, and a transmissivity of 1.6×10^{-8} m²/s for good contact, as set out in the literature in Annexure H. The head is taken as 0.3 m, as limited by a leachate collection layer, which favours the prescribed barrier. The inverted barrier leaks less than the prescribed barrier, even though it carries the full head of the saturated tailings (Table 10-6).

The HDPE lined dams rely on a single geomembrane, which is confirmed to be leak free by a dipole survey after the soilcrete ballast has been placed (section 10.7). Leakage through the dam barrier is calculated by the equation of Giroud (1997) for a circular defect in a geomembrane on a low permeability soil, $Q = 0.976 C_{qo} [1 + 0.1(h/t_s)^{0.95}] d^{0.2} h^{0.9} k_s^{0.74}$, where C_{qo} is the contact quality factor, h is the head on the geomembrane, d is the hole diameter, and t_s and k_s are the thickness and hydraulic conductivity of the soil beneath the geomembrane. The 200 mm imported fine selected layer of the receiving face (Table 14-2) is taken as the soil layer, at a hydraulic conductivity of 1×10^{-8} m/s, with no credit for the reworked in-situ soil below it. Two holes of 2 mm diameter per hectare are nevertheless allowed for (Giroud and Bonaparte, 1989), in case one or two holes are missed during construction and the survey. Fewer holes are allowed for than on the TSF because the dam geomembranes are surveyed after the soilcrete cover is placed (section 10.7), which also finds any damage caused by placing the cover. The geomembrane is taken as lying flat on the receiving face, and wrinkling is limited by placing the soilcrete cover in the cooler part of the day. Each cell is calculated separately, with the areas, volumes and water levels taken from the dam models and the head taken as the water level less the floor level of the cell. The acceptable leakage rate is calculated for good contact ($C_{qo} = 0.21$) at the maximum operating level, and the action leakage rate for good contact at FSL. The rates for poor contact ($C_{qo} = 1.15$) are also given, as an upper bound for a hole where the geomembrane is not in good contact with the receiving face. The inputs and the calculation are given in Table 10-7.

In service, a hole in the dam geomembrane lies beneath the soilcrete cover, which controls the flow into the hole in the same way as the tailings above the TSF geomembrane. The expected leakage is therefore calculated by the equation of Wissa and Fuleihan (1993), with the soilcrete at the hydraulic conductivity specified in Table 10-3 and the same two holes of 2 mm diameter per hectare. The Giroud (1997) rates apply only where a hole lies beneath a crack in the cover, and the poor contact rates are the upper bound for that case. The dam barrier is compared with the prescribed Class C barrier on the same basis as the TSF, at the head in each cell. The expected leakage and the acceptable and action leakage rates are all below those of the prescribed barrier, so the dam barrier gives equivalent performance in terms of regulation 3(2)(d) (Table 10-7).

The calculated rates hold only where the leak location survey in section 10.7 has been carried out.

The Plant PCD, the RoM pad and the silt traps are water retaining reinforced concrete to BS EN 1992-3, tightness class 1, where leakage is controlled by limiting the crack widths rather than calculated. The Plant PCD and the silt traps pass a watertightness test before commissioning, and seepage below all three is monitored through the sub-surface drainage against the triggers in section 14.5.

After closure no wet tailings are deposited on the TSF and the head on the dam barriers falls as the dams are drawn down, so the post closure leakage rates are below the operating rates.

Table 10-6: TSF Total Solute Seepage and Class C Comparison

PARAMETER	SYMBOL	AVERAGE HEAD		MAXIMUM HEAD	UNIT
Inverted Barrier System (Wissa and Fuleihan, 1993)					
Hydraulic conductivity of the tailings	k	5 × 10 ⁻⁸			m/s
Hole diameter	d	0.005			m
Geomembrane thickness	t	0.0015			m
Holes per hectare	n	5			No./ha
Liner area	A	125.9			ha
Design capacity	V	26 650 791			m ³
Footprint area	A _f	1 258 953			m ²
Average tailings thickness, V / A _f	T _{avg}	21.17			m
Operating pool depth	p	0.3			m
Maximum deposit height	H _{max}	40			m
Head on the geomembrane, tailings thickness plus pool	h	21.47	40.30		m
Thickness correction, 1 + (8/π) (t/d)	–	1.764	1.764		–
Leakage per hole, Q = 2khd / [1 + (8/π) (t/d)]	Q	6.09 × 10 ⁻⁹	1.14 × 10 ⁻⁸		m ³ /s
Leakage per hole	Q	0.526	0.987		l/d
Leakage rate, n × Q	–	2.63	4.93		l/ha/d
Total leakage for the facility, A × leakage rate	–	331	621		l/d
Wrinkles	–	Not applicable, flow through the tailings at holes			–
Prescribed Class C barrier, for comparison (Rowe, 2012)					
Geomembrane thickness	t	0.0015			m
Thickness of the compacted clay liner	D	0.3			m
Hydraulic conductivity of the compacted clay liner	k _L	1 × 10 ⁻⁹			m/s
Connected wrinkle length per hectare (Annexure H)	L	100			m/ha
Wrinkle half width	b	0.1			m
Transmissivity, good contact	θ	1.6 × 10 ⁻⁸			m ² /s
Head on the geomembrane, limited by a leachate collection layer	h	0.3			m
Leakage rate, 2L [k _L b + (k _L D θ) ^{0.5}] (h + D) / D	–	79.2			l/ha/d
Inverted barrier leakage as % of Class C	–	3	6		%

Table 10-7: HDPE Lined Dam Total Solute Seepage and Class C Comparison

CELL	ACCEPTABLE LEAKAGE RATE, MAXIMUM OPERATING LEVEL					ACTION LEAKAGE RATE, FSL				
	AREA (ha)	h (m)	EXPECTED	GOOD CONTACT	CLASS C	AREA (ha)	h (m)	GOOD CONTACT	POOR CONTACT	CLASS C
RWD Cell 1	4.55	3.85	0.08	110	548	4.90	5.07	167	915	709
RWD Cell 2	3.77	4.11	0.08	121	582	4.41	6.43	243	1 330	888
RWD Cell 3	4.27	3.88	0.08	111	552	4.91	6.21	230	1 258	859
Irrigation dam	4.56	3.20	0.06	84	462	4.64	3.46	94	514	496
WRD PCD, each cell	0.77	3.16	0.06	82	457	0.93	4.59	143	786	645
RoM PCD	0.70	3.28	0.06	87	472	0.77	3.94	114	623	559

Leakage rates in l/ha/d. Expected: hole beneath the soilcrete cover (Wissa and Fuleihan, 1993). Good and poor contact: hole beneath a crack in the cover (Giroud, 1997). Class C: prescribed barrier at the same head (Rowe, 2012).

10.10 Filter Compatibility

Wherever a fine material lies against a coarser material or a geotextile through which water flows, the coarser layer has to retain the fine particles so that they are not carried into a drain, or into a hole in the geomembrane, while staying permeable enough that pore pressure does not build up behind it. The interfaces in this design where this applies are listed in Table 10-8, with the base (finer) material, the filter and the criteria applied to each.

Granular filters are designed to the criteria of the USDA National Engineering Handbook, Part 633, Chapter 26 (USDA, 2017). The base material is grouped by its fines content once its grading is adjusted to 100% passing the 4.75 mm sieve, and the D15 of the filter is limited so that it retains the base. The D15 of the filter must also be at least four times the d15 of the base material, and not less than 0.1 mm, so that the filter is more permeable than the material it protects. Filters contain no more than 5% fines passing the 0.075 mm sieve, and the fines are non-plastic. Each layer of a multi-layer filter, such as the graded filter drains of the TSF, is checked against the layer it protects.

Geotextile filters are checked for retention, permeability and resistance to clogging to Holtz et al. (2008), and for survivability to AASHTO M288. Where the clogging screen is not met, and for the widely graded and gap graded transported soils, resistance to clogging is confirmed by a gradient ratio test (ASTM D5101) on the soil or tailings concerned. The protection geotextile beneath the drains on the geomembrane is checked for puncture after Koerner (2012) and confirmed by a cylinder test (EN 14574).

No representative tailings from the Volspruit plant are available yet. In the interim, the filters and geotextiles are designed on an envelope of coarse (cycloned underflow) and fine (overflow) tailings gradings from similar projects, based on experience, and on generic geotextile properties. The checks are given in Table 10-9 and the filter bands in Figure 10-3. The transported soils have fines contents of 10% to 80%, averaging 40%, and the residual soils 6% to 35%, averaging 21% (Table 4-6). None of the soils showed signs of dispersivity (Chapter 4).

The checks are repeated on the Volspruit tailings and the tendered geotextiles as soon as representative samples are available. Grading analyses of the founding soils, the receiving face material, the filter sand and the aggregate are carried out before the materials are ordered and repeated under the construction quality assurance plan. The apparent opening size (ASTM D4751) and permittivity (ASTM D4491) of each geotextile are confirmed by conformance testing (Annexure I).

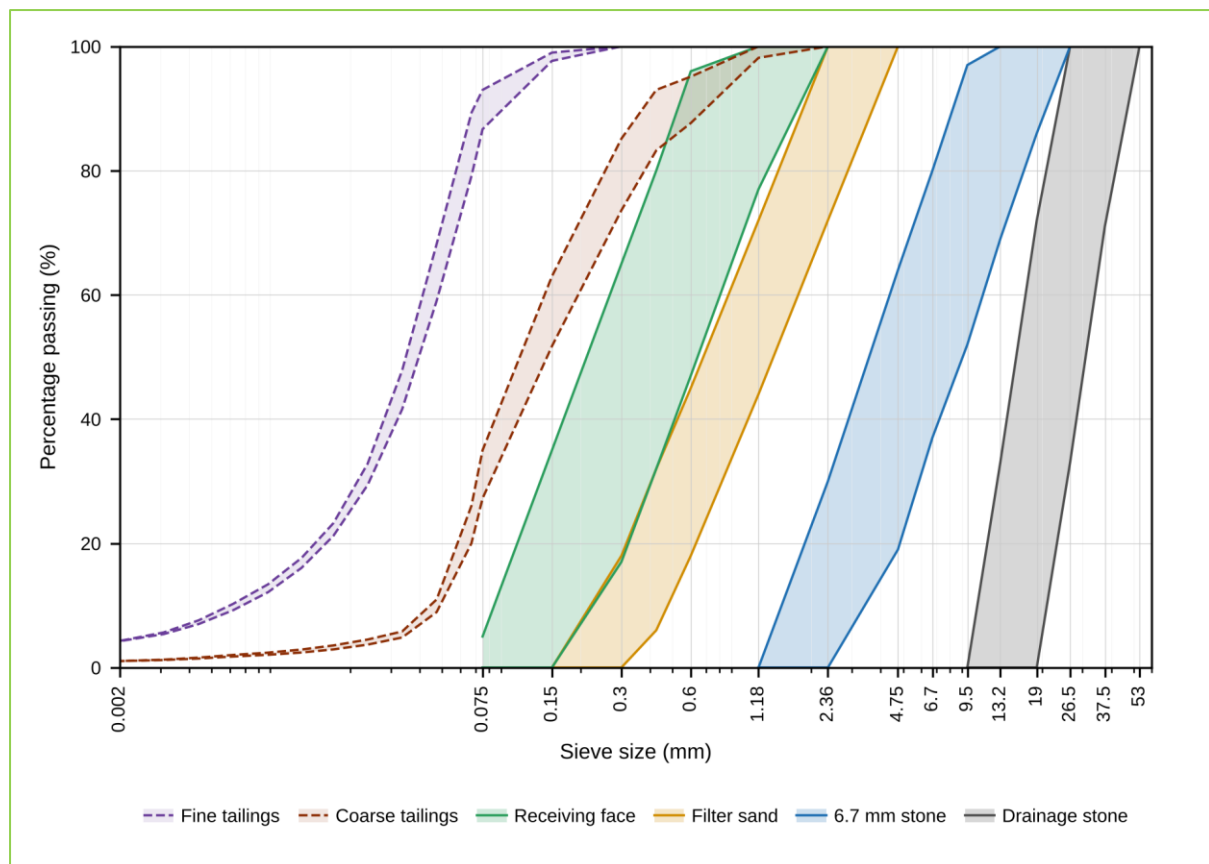
For the flood diversion berm, the sequence from the clay core through the two filter zones to the rock fill shell (Table 18-1) is checked layer by layer against the same criteria once the gradings of the North Pit materials are known. On the nominal sizes given, the step from the river sand to a gravel with a D50 of 40 mm is large, and the grading of filter zone 1 may need to be widened so that it retains the sand.

Table 10-8: Filter Compatibility Interfaces

INTERFACE	BASE MATERIAL	FILTER OR DRAIN	CRITERIA	REQUIREMENT
TSF inverted barrier, at holes in the geomembrane	Fine tailings slimed over the geomembrane	Receiving face of imported fine selected earth fill, or reclaimed tailings in cells 2 and 3 (section 11.5)	Granular criteria (USDA, 2017); maximum particle size 3 mm (SANS 10409)	Grading of the receiving face meets the criteria for the fine tailings. The fraction of reclaimed tailings used is checked against the same criteria.
TSF toe drains above the geomembrane	Coarse tailings	Graded filter drain of filter sand, 6.7 mm stone and drainage stone around drilled HDPE pipes, under a 550 g/m ² geotextile, on a 1 200 g/m ² protection geotextile (19-918-SWM-456)	Granular criteria, layer by layer; geotextile criteria; puncture	Filter bands in Figure 10-3. Pipe slots and holes smaller than the D85 of the stone around them.
TSF basin leachate drains above the geomembrane	Coarse tailings	19 mm stone around drilled HDPE pipes, wrapped in a 550 g/m ² geotextile, on a 1 200 g/m ² protection geotextile (19-918-SWM-468)	Geotextile criteria; puncture	Pipe slots and holes smaller than the D85 of the stone around them.
Sub-surface drains below the TSF, the WRD, the dams and the silt traps	In-situ transported and residual soils	Aggregate around corrugated slotted HDPE pipes, wrapped in a 210 g/m ² geotextile (19-918-SWM-466, SWM-906 and SWM-907, and section 12.4)	Geotextile criteria	Geotextile O95 and permeability checked against the site soil gradings, with a gradient ratio test on transported soil.
WRD perimeter cut-off drain	In-situ transported and residual soils	Geocomposite panel drain wrapped in a 270 g/m ² geotextile, on Class I aggregate (19-918-SWM-152)	Geotextile criteria	As for the sub-surface drains.
Starter wall upstream face	800 mm transition layer	Waste rock body, separated by a 550 g/m ² geotextile (19-918-SWM-451)	Geotextile criteria (separation and retention)	Transition layer retained by the geotextile, so that it does not migrate into the voids of the waste rock.
Flood diversion berm	Clay core	Filter zone 2 (river sand), filter zone 1 (gravel, D50 40 mm) and rock fill shell (D50 300 mm) (Table 18-1)	Granular criteria, layer by layer	Gradings of the North Pit core, filter and rock fill materials checked before construction.

Table 10-9: Interim Filter Compatibility Checks

INTERFACE	CHECK	RESULT
Granular filters (USDA, 2017)		
Filter sand on coarse tailings (toe drain)	Retention and permeability	Meets
6.7 mm stone on filter sand (toe drain)	Retention and permeability	Meets
Drainage stone on 6.7 mm stone (toe drain)	Retention and permeability	Meets
Receiving face below fine tailings	Retention	Meets
Geotextiles (Holtz et al., 2008; AASHTO M288; Koerner, 2012)		
550 g/m ² toe drain cover and basin drain wrap, on coarse tailings	Retention: $O_{95} \leq B \times D_{85}$	Meets (O_{95} 0.15 mm against a limit of 0.44 mm)
	Permeability: $k \geq 10 \times k$ of the tailings; permittivity $\geq 0.2 \text{ s}^{-1}$	Meets
	Clogging screen: $O_{95} \geq 3 \times D_{15}$	Not met. Gradient ratio test (ASTM D5101), $GR \leq 3$, on the Volspruit coarse tailings and the tendered geotextile (hold point)
	Survivability: AASHTO M288 Class 1	Grab strength meets. Tear and puncture strength confirmed on the tendered geotextile
1 200 g/m ² protection geotextile on the geomembrane, below the toe and basin drains	Puncture: $FS \geq 3$	Meets. Confirmed by a cylinder test (EN 14574) on the tendered geotextile and drainage stone (hold point)
550 g/m ² separator below the starter wall transition layer	Retention and survivability	Checked once the transition layer grading is set

**Figure 10-3: Interim Filter Bands**

PART C — FACILITY DESIGN



11 TAILINGS STORAGE FACILITY

11.1 Overview and Design Objectives

The TSF is designed to receive the PGM tailings generated by the Volspruit plant for the life of mine. The design has been developed in terms of the applicable legislation and the design criteria set out in Chapter 9.

The barrier design framework adopted for the facility is set out in Chapter 10.

The design objectives adopted for the facility are:

- secure containment of the PGM tailings within the TSF footprint;
- drawdown, storage and re-use of supernatant and free water emanating from the TSF operations, with allowance for the temporary containment of contaminated water arising from extreme storm events on the TSF and the management of surface runoff;
- control of the phreatic regime within the constructed structures and the minimisation and mitigation of possible seepage into the environment, through a barrier and drainage network system consisting of a sub-surface drainage system, a leachate drainage system, filter mediums and outfall lines;
- design of starter, toe and heel walls, with the starter wall functioning as the main structural wall and as the containment wall for the initial rate of rise of the overflow fine tailings and slurry water during the initial operational phase;
- design of a gravity penstock system and associated intake structures for the decanting of the collected supernatant and stormwater pool;
- installation of monitoring equipment to monitor the phreatic characteristics and the movement behaviour of the TSF over time;
- reasonable ease of operation during the development of the TSF; and
- effective rehabilitation for closure of the developed TSF.

11.2 Dam Safety Registration and Classification

The TSF is a proposed facility and will be registered with the Dam Safety Office of the Department of Water and Sanitation as a dam with a safety risk, in terms of section 120 of the National Water Act. This is in line with Government Notice 5576 of 22 November 2024 (Government Gazette 51637), which calls on owners of dams with a safety risk to register them with the Department.

Before construction starts, the facility will be classified by the Department and a licence to construct obtained in accordance with the Regulations regarding the Safety of Dams, GN R139 of 2012. An approved professional person will be appointed for the facility as required for the category assigned.

11.3 Layout, Capacity and Compartments

The TSF is viewed as a structure with a starter containment wall for the lower portion to the west, with the facility then developed as three compartments by means of the upstream construction method for the portion above the starter walls. The compartments allow for more efficient operability.

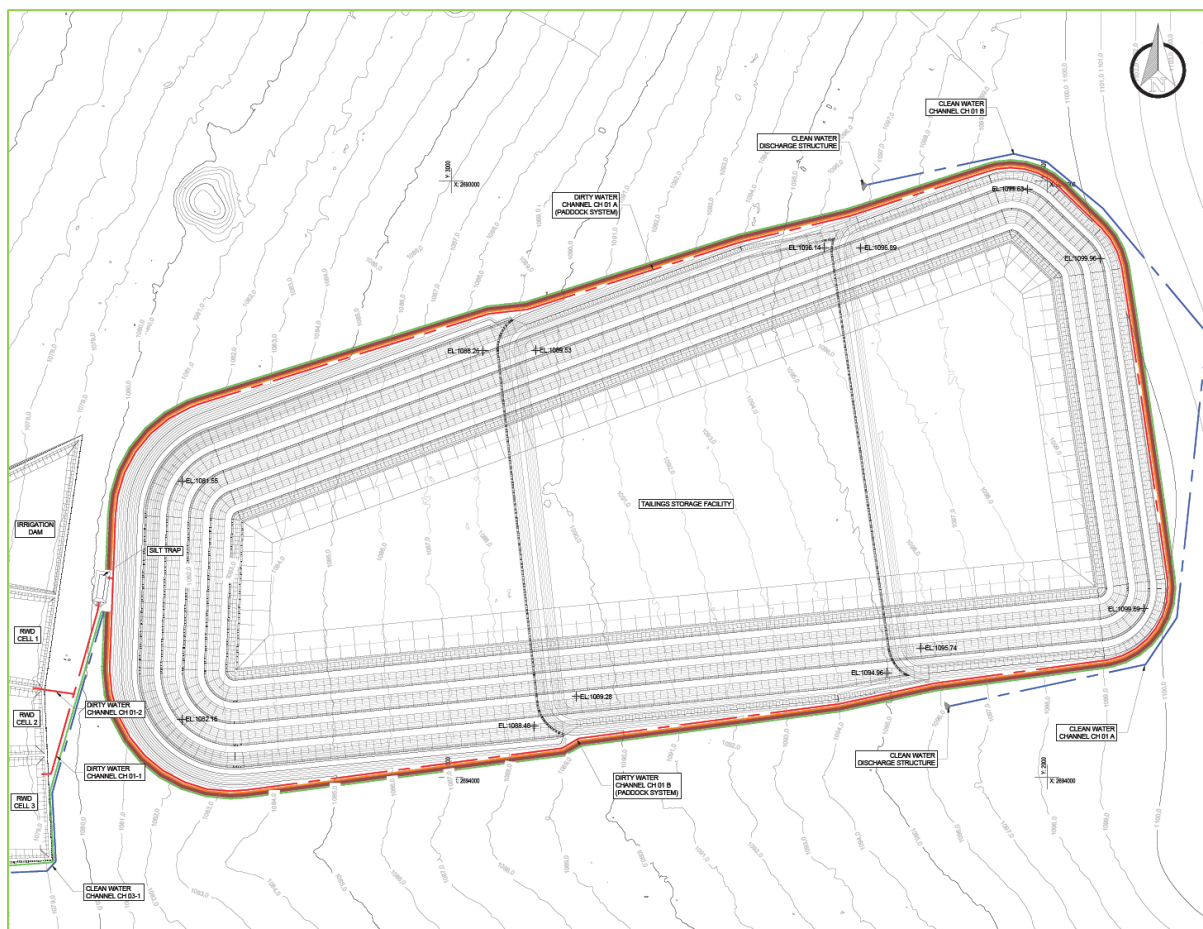


Figure 11-1: TSF General Layout

The stage capacity of the facility is based on the footprint area and the information supplied by the Client. The focus is to sequentially initiate the three cells, with each rising within the acceptable rate of rise while allowing each cell to consolidate. Cells 1 and 2 consolidate and rise together to a level at which cell 3 consolidates. The operations remain as individual cells, each having its own penstock. The result is a moderate annual rate of rise over the life of the facility, which remains within the maximum rate of rise given in Table 9-6.

The capacity and development parameters adopted for the facility are given in Table 11-1.

Table 11-1: TSF Capacity and Development Parameters

PARAMETER	UNIT	VALUE
Design capacity	m ³	26 650 791
Target in-situ density	t/m ³	1.60
Design tonnage	t	42 641 266
Life of facility at 243 000 t/month	years	14.6
Number of compartments	No.	3
Construction method above the starter wall	–	Upstream
Footprint area	m ²	1 258 953
Final outer wall crest level	m	1 122.00
Starter, toe and heel wall side slopes	–	1V:3H
Intermediate bench side slopes	–	1V:3.5H
Overall outer slope	–	1V:5.3H

The stage capacity curve for the facility is given in Figure 11-2. It shows the area, volume and rate of rise of each compartment against elevation, and the development of each compartment over the life of the facility.

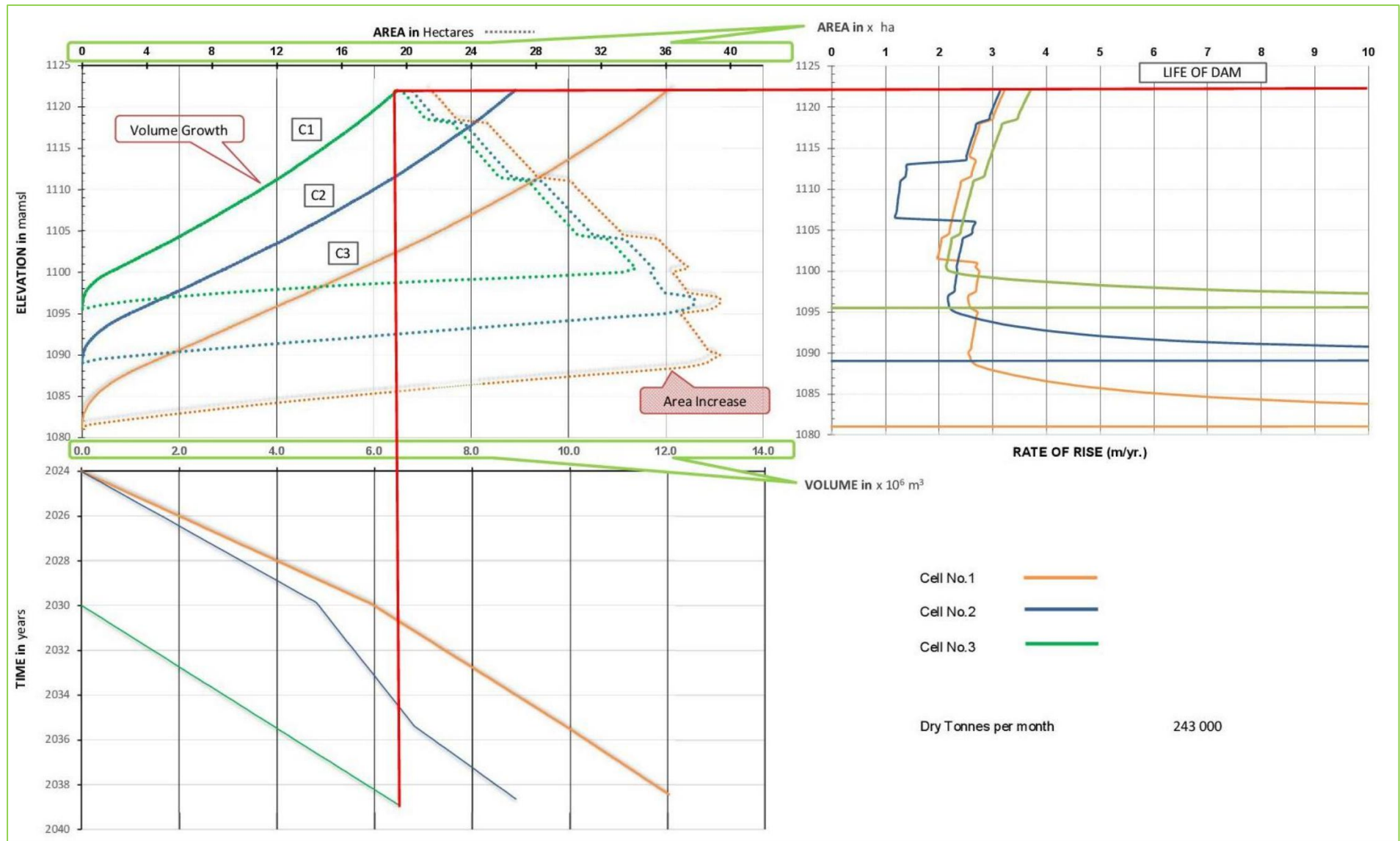


Figure 11-2: Stage Capacity Curve

11.4 Staged Development and Rate of Rise

The initial deposition focuses on covering the barrier and the drainage corridor as soon as reasonably possible. Coarse underflow tailings are placed over the drainage corridor between the confinement walls and the heel wall, and fine tailings are slimed over the geomembrane in the basin. This limits the deterioration of the exposed geomembrane, reduces the risk of theft and limits the clogging of the drainage structures by wind and water-borne solids. The fines runoff is initially concentrated around the temporary intakes to allow drawdown to the plant as early as possible.

When the facility has broken ground over the entire footprint area of a compartment, a tailings pool wall is built from the southern side. The pool wall controls the pool of the cell and provides access to the central decanting units, avoiding the continuous raising of the catwalks. The moderate rates of rise adopted for the facility allow the tailings to consolidate as the facility is raised, taking into account the safety classification of the facility and the fine material to be deposited.

11.5 Foundation Preparation

The founding area of the basin and of the confinement walls is to be cleared of all vegetation and the topsoil stripped and stockpiled for re-use during rehabilitation. Any soft, compressible or otherwise unsuitable material identified during construction is to be removed under the direction of the Engineer.

Collapsible sandy transported soils were identified within the TSF footprint during the geotechnical investigation (Peregrine Geoconsultants, PG19-003/673 Rev02, June 2021). Double oedometer tests on samples from test pits TSF-TP02, TSF-TP04 and TSF-TP05 gave collapse potentials of 5.2% to 7.6% at 100 kPa, rising to 14.2% to 15.6% at 400 kPa. These soils are to be treated by ground improvement, either impact rolling or removal and replacement as recommended in the geotechnical report, before the basin receiving face and the wall foundations are prepared. The extent, depth and method of treatment are to be confirmed by the detailed geotechnical investigation and approved by the Engineer.

The basin is then cut and recompacted to form the receiving face for the geomembrane. The base is ripped and recompacted as the foundation preparation layer to 98% of Standard Proctor maximum dry density at OMC 0% to +1.5%, and a 150 mm layer of imported fine selected earth fill is placed over it and compacted to 98% of Standard Proctor maximum dry density at OMC 0% to +1%. The finished surface is to be smooth and free of protrusions that could puncture the geomembrane, with a maximum particle size of 3 mm, as required by SANS 10409 (2020). The receiving face is inspected over its full area and accepted by the Engineer and the geomembrane installer before the geomembrane is placed.

In cells 2 and 3, the 150 mm layer of imported fine selected earth fill may be replaced by a compacted layer of tailings reclaimed from the operating cells. The tailings solids classify as Type 4 (Chapter 5), and the reclaimed tailings are used as the receiving face only if all of the following are met before the layer is placed:

- the layer is compacted to 98% of Standard Proctor maximum dry density at OMC 0% to +1%, and the finished surface meets the requirements above, with a maximum particle size of 3 mm;

- the grading of the layer is filter compatible with the fine tailings slimed above the geomembrane (section 10.8) and with the drainage aggregate of the sub-surface drains below it; and
- the interface shear strength between the layer and the geomembrane (ASTM D5321) is not less than the value assumed in section 11.14.

Where any of these criteria is not met, the imported fine selected earth fill is used. The fraction of the tailings to be used, the reclaim and drying method and the timing against the deposition programme are agreed with the Client before the cell is constructed. The leakage calculation in section 10.9 does not rely on the receiving face and is unchanged.

The base of the starter, toe and heel walls is prepared by stripping vegetation and topsoil prior to the excavation of a box cut, 1 000 mm deep below the starter wall and 300 mm deep below the toe and heel walls, taken down to a competent founding layer approved by the Engineer. The base of the box cut is ripped and recompacted in layers not exceeding 200 mm to 95% of Standard Proctor maximum dry density at OMC 0% to +2%. The wall fill is placed and compacted to 98% of Standard Proctor maximum dry density at OMC 0% to +2%.

11.6 Barrier System

The basin of the TSF is lined with a Class C equivalent inverted barrier system. The motivation for the system, the equivalence demonstration and the material specifications is set out in Chapter 10. The system places the geomembrane directly on a SANS 10409 (2020) compliant receiving face of imported fine selected earth fill, or in cells 2 and 3 of compacted reclaimed tailings (section 11.5), with the fine tailings slimed above it, and removes the clay layer from the barrier.

Phreatic and hydraulic head management across the barrier is provided by a leachate drainage network above the geomembrane and a sub-surface drainage network below it. The barrier system comprises, from the top down:

- a leachate drainage network of drilled HDPE drainpipes in a graded filter drain above the geomembrane, reporting by gravity to the solution trench Table 11-3;
- fine tailings slimed directly over the geomembrane;
- a 1.5 mm double textured HDPE geomembrane to SANS 1526 (2015), with manufacturing quality control as per GRI-GM13 and welds as per GRI-GM19, installed in accordance with SANS 10409 (2020);
- a receiving face to SANS 10409 (2020), prepared as described in section 11.5: in Phase 1 (cell 1), a 150 mm layer of imported fine selected earth fill; in Phases 2 and 3 (cells 2 and 3), a compacted layer of reclaimed tailings where it meets the acceptance criteria in section 11.5, with the imported fill used where it does not;
- a foundation preparation layer of the in-situ base, ripped and recompacted to 98% of Standard Proctor maximum dry density at OMC 0% to +1.5% after the ground improvement of the collapsible soils (section 11.5); and

- a sub-surface drainage network of corrugated slotted HDPE branch drains and solid HDPE mains below the geomembrane, reporting by gravity to the sub-surface drainage outlet sump Table 11-3.

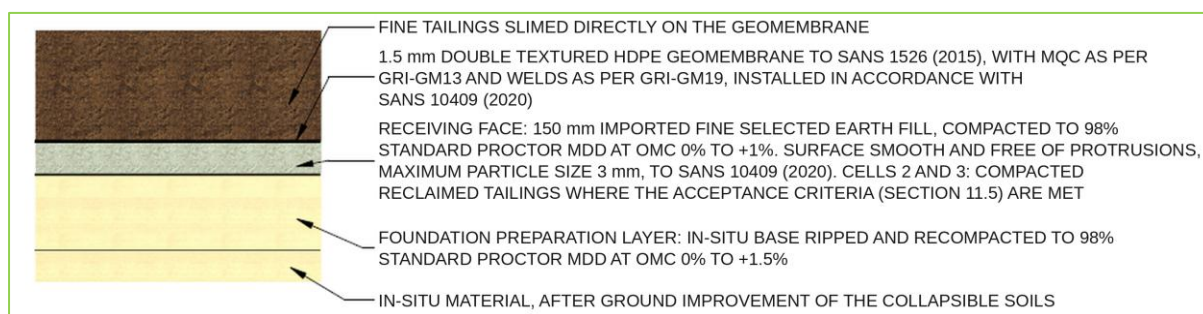


Figure 11-3: Inverted Barrier System Layerworks

The barrier requires specific detailing where it ties into concrete structures, load bearing structures and along the edge of the barrier surface, so that it is not torn and does not remain under strain that could weaken the material. The geomembrane is anchored along the outer edge in an anchor trench excavated along the crest of the confinement walls, which is then backfilled with selected material stabilised to a minimum strength of 5 MPa. Where the geomembrane ties into the penstock foundation a stainless-steel flat bar and rubber strip fixing is used to mechanically join the geomembrane to the concrete, and the perimeter area around the penstock foundation is provided with a 5 MPa soilcrete layer of approximately 150 mm thickness to limit the strain at the make-off to the rigid structure in the event of differential settlement. The fixing comprises a 50 × 5 mm 316 (A4) stainless steel flat bar, drilled at 150 mm centres and fastened with M10 × 130 mm 316 (A4) stainless steel chemical anchor bolts, over two 50 mm wide, 3 mm thick EPDM rubber strips placed above and below the geomembrane. The rubber is UV and chemical resistant, with a Shore A hardness of 60 to 70. Drainage pipes penetrate the barrier through a concrete anchor block cast in situ in C35/19 reinforced concrete, with a fusion welded HDPE puddle flange on the pipe and the geomembrane fixed to the block by the same flat bar fixing. The two Ø110 mm solid HDPE leachate outfall pipes then run through the embankment encased in C20/19 mass concrete, at a minimum slope of 0.5% (19-918-SWM-457). The liner penetration detail is shown in Figure 11-4.

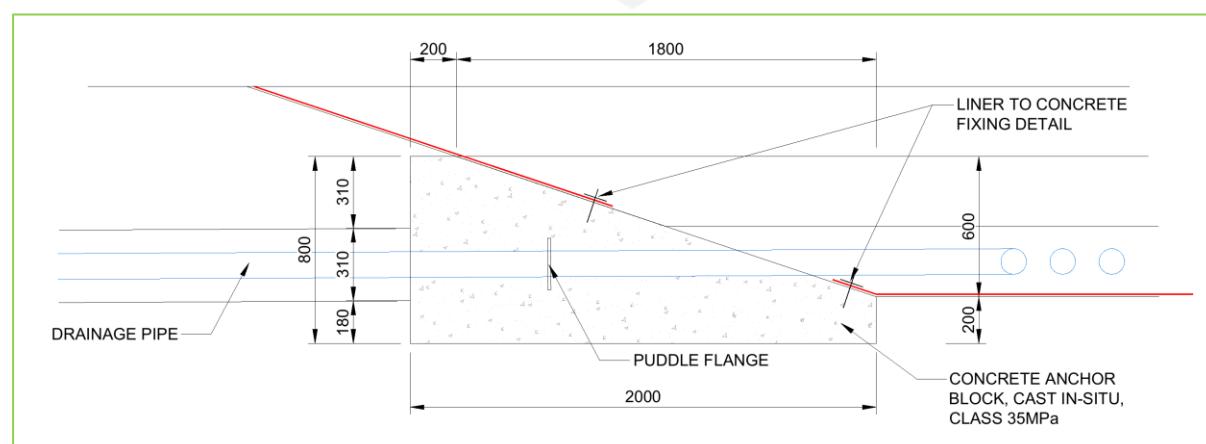


Figure 11-4: Typical Liner Penetration Detail

The geomembrane is taken over the heel wall and to the top of the confinement walls. A non-woven geotextile is placed to protect the geomembrane where the hydro-cyclone stands on the heel wall crest. The mass of this geotextile is confirmed against the hydro-cyclone loads once the equipment has been selected, before construction.

11.7 Barrier Performance Verification

The geomembrane is surveyed for leaks before deposition starts, as specified in section 10.7.

The total tensile strain in the geomembrane is to be limited to less than 3%. This is to be confirmed by an in-situ field test undertaken using the actual materials employed in the construction of the barrier system, evaluated using the Tognon et al algorithm for total tensile strain, being the sum of the plain and bending strain components.

The actual interface shear between the geomembrane and the tailings above it, and between the geomembrane and the foundation material below it, is to be confirmed by testing on the materials employed in the works. The stability analyses in this report are checked against the confirmed values before deposition starts. The verification requirements form part of the construction quality assurance programme set out in Chapter 20.

11.8 Starter, Toe and Heel Walls

The compacted starter wall and toe walls provide containment for the initial start-up of the compartments. The starter wall has a constant crest elevation and a crest width that allows construction vehicle access and pipelines. As the majority of the TSF footprint is located on ground sloping in a westerly direction, the starter wall reduces in height until it terminates at natural ground level, at which point it becomes the toe wall that is constructed around the remaining perimeter of the TSF cells. The toe wall has a constant height and therefore a crest elevation that varies in relation to the ground elevation. The toe walls demarcate the edge of the depositional area and provide a starter wall for the placement of the spigot pipes.

The heel wall runs parallel to the outer confinement walls and provides a drainage corridor in which the blanket and toe drains are located. The heel wall separates the coarse and fine tailings material, particularly during the initial deposition period, and functions as the base on which the hydro-cyclones stand. The heel wall is constructed in the same way as the toe wall, with the same crest width, side slopes and foundation preparation. The zone created between the confinement walls and the heel wall is where most of the phreatic management occurs, and it is extended vertically by coarse tailings forming an outer curtain that draws down and performs as drainage material leading to the filter drains along the drainage corridor.

The walls are zoned structures with a waste rock toe zone and a main constructed zone of locally sourced material, with the body of the walls consisting of fine and medium grade waste rock compacted to performance specifications. An 800 mm transition layer, detailed on drawing 19-918-SWM-451, is placed on the upstream slopes of the waste rock embankments over a non-woven geotextile separation

layer. Its top surface is finished smooth and free of protrusions, with a maximum particle size of 3 mm, in accordance with SANS 10409 (2020), and forms the receiving face for the geomembrane.

The geometry adopted for the confinement walls is given in Table 11-2. The material specifications are given in Chapter 19.

Table 11-2: Confinement Wall Geometry

PARAMETER	STARTER WALL	TOE WALL	HEEL WALL
Crest width	9 m	4.6 m	4.6 m
Maximum height	10 m	1 m	2 m
Side slopes	1V:3H	1V:3H	1V:3H
Box cut depth	1 000 mm	300 mm	300 mm

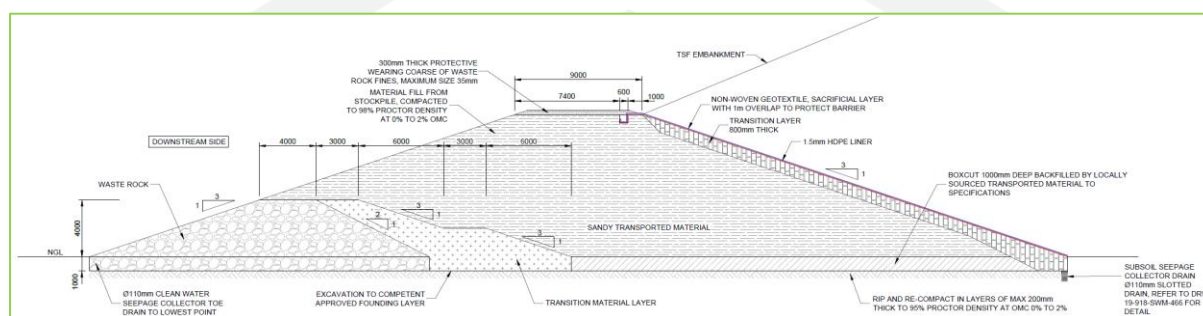


Figure 11-5: Typical Starter Wall Section

11.9 Sub-surface and Internal Drainage

The topography of the site makes the arrangement of the drainage network more complex, with each cell having its own drainage network and a portion of the outlet lines running beneath the downstream cell. Sub-surface drains are installed throughout the basin below the geomembrane and report by gravity through solid mains into the sub-surface drainage outlet sump. The leachate drains are installed throughout the basin above the geomembrane and report by gravity to the solution trench. The solid and drilled HDPE pipes are butt welded to form a strong, leak-proof joint, and the corrugated sub-surface branch drains are joined with the pipe manufacturer's couplers.

The basin drains are laid out as a herringbone network with dual, interconnected main lines to provide a level of redundancy. Cycloned coarse tailings are placed over the basin and toe drains before the fine tailings reach them, so that the drain geotextiles are not in direct contact with the fine tailings. The filter compatibility checks are given in section 10.10. Care is to be taken not to blind the filter material during construction or leave it exposed to the elements. Clogging of the drains by chemical precipitates or organic growth is not expected, as the tailings are non-acid forming and contain no significant organic matter (Chapter 5). A sustained fall in the drain outflow recorded under section 11.16 triggers an inspection of the drains.

The components of the drainage system are given in Table 11-3.

Table 11-3: TSF Drainage System Components

COMPONENT	LOCATION	DESCRIPTION
Sub-surface drainage	Below the geomembrane	Ø110 mm corrugated slotted HDPE sub-surface drainage pipe branches, class SN16 (ring stiffness 16 kN/m ² to ISO 9969) or similar approved, in Class I crushed stone aggregate wrapped in a non-woven, needle punched geotextile (Table 10-4). The branches are joined with the pipe manufacturer's couplers and connected to the mains with HDPE Y-bends and adaptors. Ø160 mm HDPE PE100 PN12.5 solid main pipes, butt welded, on a bedding cradle of selected granular sand with a fineness modulus of not less than 2 and a compactibility factor not greater than 0.4. The mains discharge into the sub-surface drainage outlet sump (19-918-SWM-465 and 19-918-SWM-466)
Toe and blanket drain	Drainage corridor between the confinement walls and the heel wall	7 m wide graded filter drain on a 1 200 g/m ² protection geotextile over the geomembrane, comprising drainage stone with two drilled 110 mm diameter drainpipes, 6.7 mm stone and filter sand to the filter bands in Figure 10-3, covered by a 550 g/m ² geotextile
Basin drainage network	Above the geomembrane	Herringbone network of drilled HDPE drainpipes, with dual 110 mm diameter main collectors, 110 mm diameter seepage drains and 75 mm diameter outer reaches, encased in washed 19 mm aggregate wrapped in a 550 g/m ² geotextile, on a 1 200 g/m ² protection geotextile over the geomembrane (19-918-SWM-468)
Outfall drainage	Through the confinement walls	Two Ø110 mm HDPE PE100 PN12.5 solid leachate outfall pipes, butt welded, encased in C20/19 mass concrete at a minimum slope of 0.5%, conveying the collected water to the solution trench (19-918-SWM-457)
Sub-surface drainage outlet sump	At the outlet of the sub-surface drainage mains	Reinforced concrete sump, water retaining (Table 19-1), receiving the five Ø160 mm mains through HDPE puddle flanges, not exceeding 300 mm outside diameter, fusion welded to the pipes and cast centrally in the wall, with no construction joint passing through a flange. Minimum cover to the reinforcement is 50 mm on faces in contact with the ground and 40 mm on all other faces. Founded on a 300 mm layer of imported G7 material compacted to 95% of Mod AASHTO density at OMC 0% to +2%, over the in-situ material ripped and recompacted to 93% of Mod AASHTO density at OMC 0% to +2% (19-918-SWM-467)

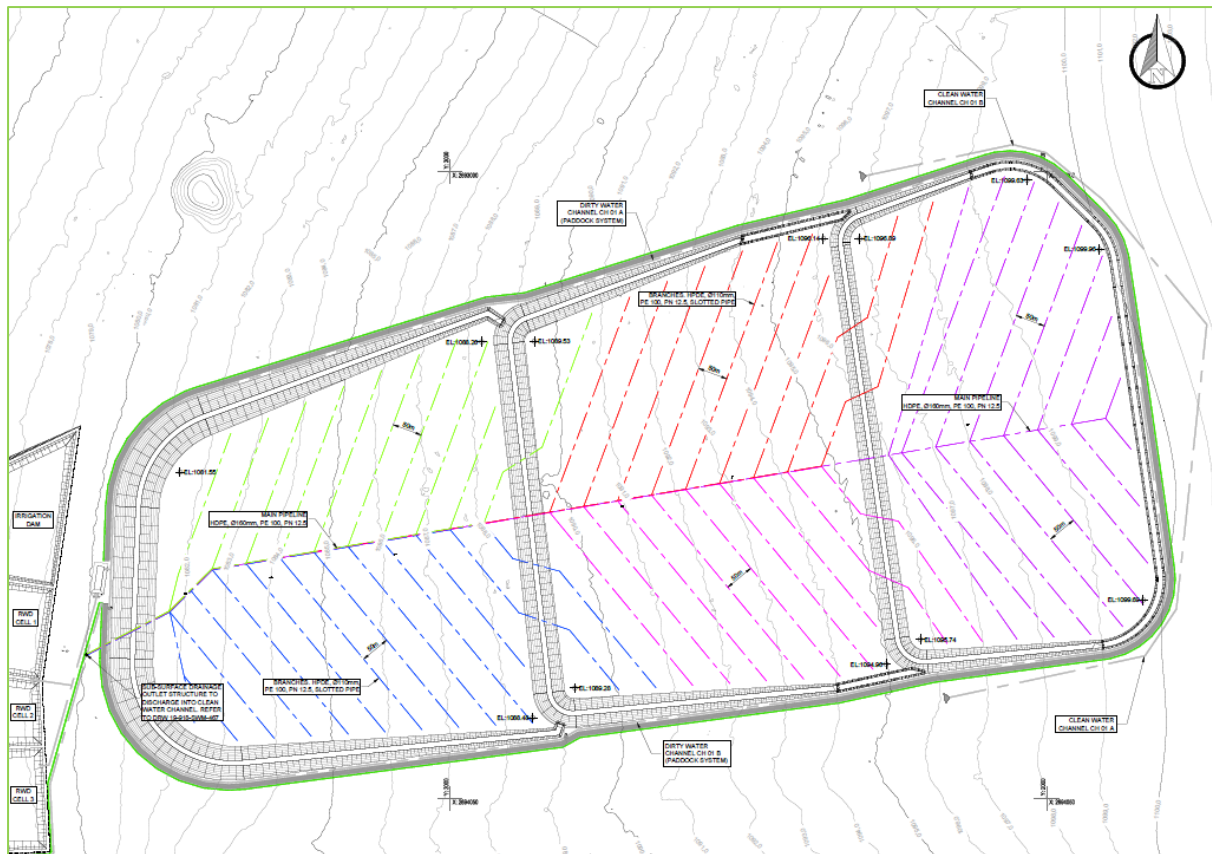


Figure 11-6: Sub-Surface Drainage Layout

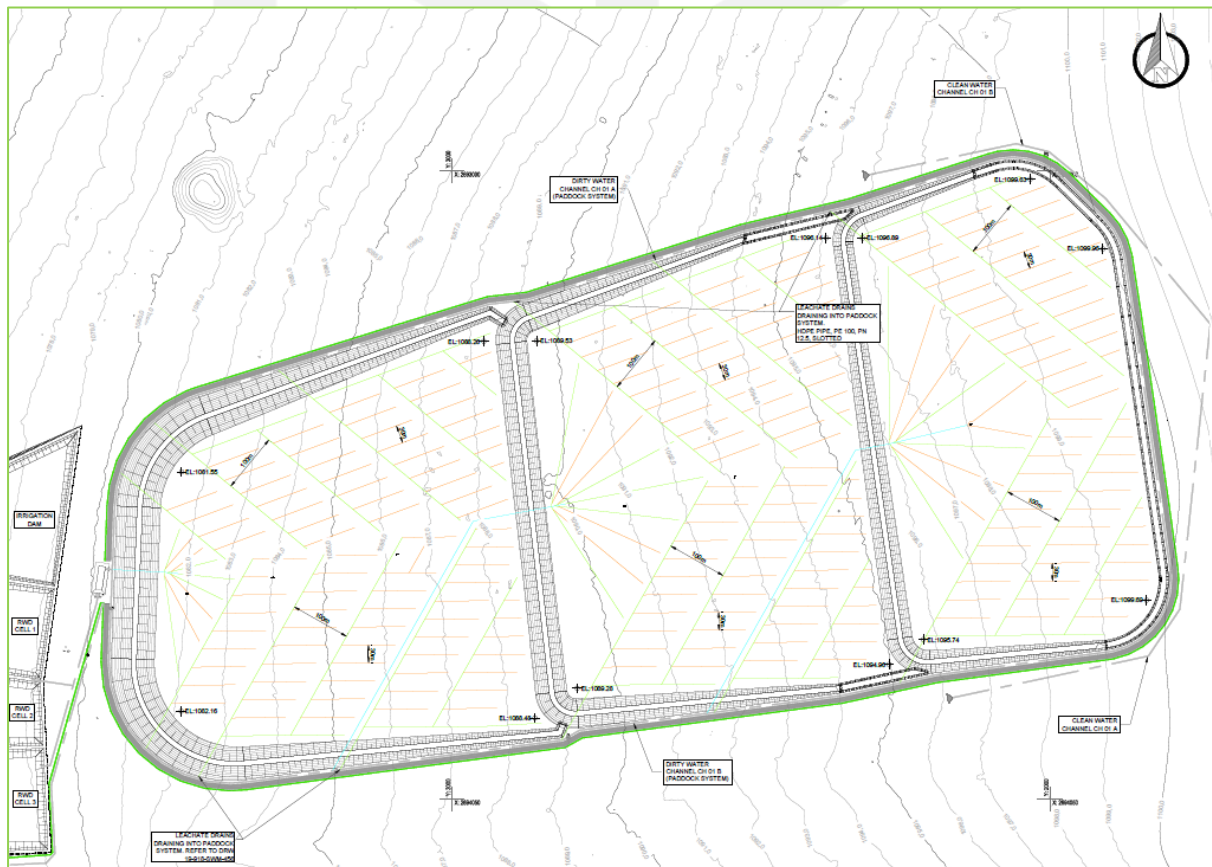


Figure 11-7: Basin Leachate Drainage Network

11.10 Perimeter Solution Trenches

A solution trench is constructed around the perimeter of the TSF to convey the collected seepage and drainage water to the RWD. The solution trench is concrete lined throughout its length, with its section and depth given on the channel long sections. The trench crosses the access road on the western side in one place by means of a pipe culvert. The lining and layerworks are those of the dirty water channels in Table 16-1 (19-918-SWM-469).

The trench is sized to convey, without overflowing, the 1:50-year flood peak from the TSF side slopes together with the outflow from the penstock system and the outflow from the TSF drains. The penstock pipeline is sized to draw down the stormwater storage from the 1:100-year, 72-hour event within 72 hours. The drain outflow has not been calculated and is taken as the full-bore capacity of the drain outfall pipes discharging to each reach of the trench, which is an upper limit on the flow the drains can deliver. The penstock and drain outflows enter the trench as point inflows where each outfall discharges. The three flows are assumed to coincide, which is conservative, as the penstock drawdown continues after the storm peak has passed. The design criteria are given in Chapter 9.

The solution trench is made up of DW CH 01A and DW CH 01B, which discharge through the RWD silt trap (Chapter 15) into the RWD (Chapter 14).

11.11 Penstock and Decant System

A permanent 500 mm HDPE PE100 PN16 outfall pipeline is installed in each cell, from the central concrete decant towers of that cell to the solution trench. The line is installed to fall within a trench that is filled with C30/19 concrete as a full encasement. The remainder of the trench is backfilled and compacted before the geomembrane is installed. Concrete sleepers and saddle straps are placed at 2 m intervals along the outfall pipe to hold it at line and level and stop it floating while the encasement is poured. At its outlet the pipe turns through a 90° long radius HDPE bend and discharges into the solution trench between C35/19 reinforced concrete splash walls with a minimum cover of 30 mm. The pipe connects to the bend through HDPE stub flanges with galvanised steel backing rings drilled to SANS 1123, table 1600/3, and passes through the splash wall with a fusion welded HDPE puddle flange (19-918-SWM-464).

Temporary intakes are located at intervals along the penstock outfall pipeline in each cell and are used as the pool migrates towards the centroid of the TSF. As the pool migrates the redundant temporary intakes are sealed appropriately. A cluster of reinforced concrete intake structures is located at the centroid of the TSF and functions as the final intake structure. A starter ring is cast on top of each reinforced concrete tower to form the base onto which the precast concrete rings are stacked as the tailings level rises.

The temporary and final intake structures are C35/19 reinforced concrete, founded on a 500 mm thick rockfill layer with a D50 of 26.5 mm, placed on the in-situ competent material ripped and recompacted to 98% of Standard Proctor density at OMC. The minimum cover to the reinforcement is 75 mm on faces cast against the rockfill and 50 mm on all other faces. The geomembrane is fixed to the intake

structures by the liner to concrete fixing detail on 19-918-SWM-907 (19-918-SWM-462 and 19-918-SWM-463).

The precast penstock rings control the inflow of the supernatant pool water, and the lower rings retain the slimes from the settled tailings. The stack of rings rises as the level of the tailings rises, with the safe height of a ring stack limited to 20 m based on historical safe practice. Once that height is reached the stacked precast penstock is sealed and the next ring stack is started on the second set of reinforced concrete towers. The two final penstock intake towers allow for the full development height of the TSF.

The supernatant pool for each cell starts forming against the starter wall on the western side of the TSF cells. Intermediate intake structures located along the main penstock outfall pipeline function as drawdown structures while the pool migrates from the starter wall to the final location around the final intake structures. The pool migration and location are controlled through the deposition process and the construction of pool walls during the operational phase.

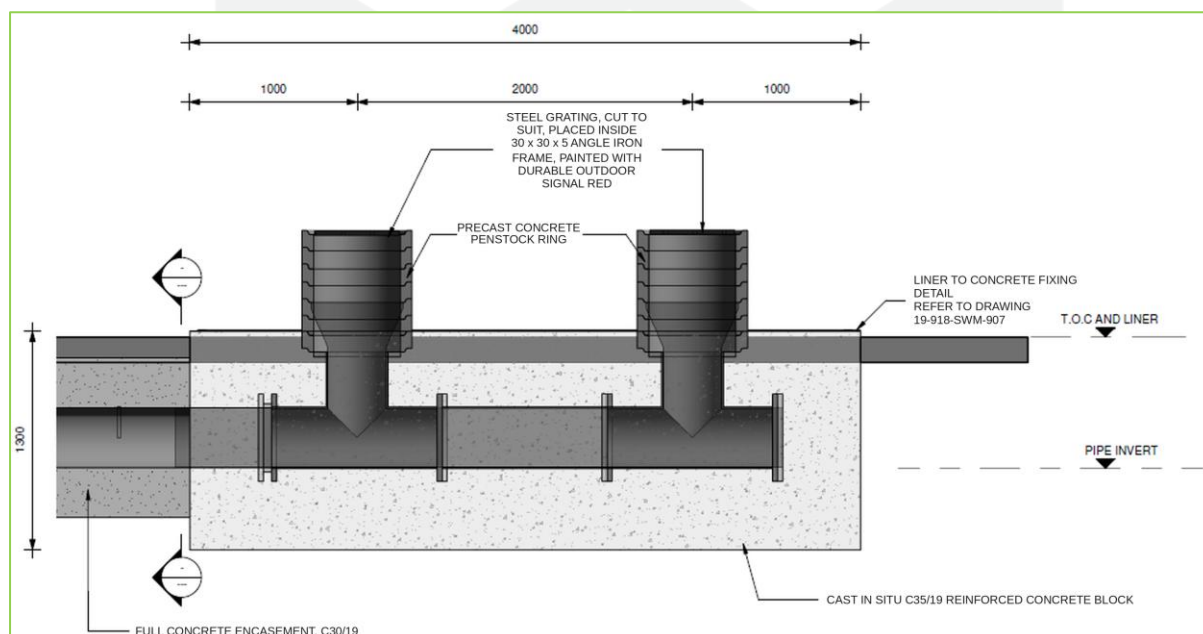


Figure 11-8: Final Penstock Intake Structures

A timber catwalk structure provides a walkway along the length of the penstock pipeline for each compartment (19-918-SWM-461). The timber is treated in accordance with clause 6 of SANS 457-1 with a copper chromium arsenic preservative complying with SANS 673, and the walkway rests on 210 litre plastic barrels filled with C20/19 mass concrete, bedded on a geotextile (Table 10-4) wrapped twice. All footings are C20/19 mass concrete. Access to the walkway is controlled by a galvanised steel swing gate with a warning sign. As the height increases a pool wall is developed leading from the east, and the catwalk then leads from the pool wall instead of over the full length from the outer wall of the TSF. Once the permanent penstock intake structures are commissioned the long timber catwalk can be replaced by a hydraulically placed pool wall. The installation of the pool wall forms part of the Operator's work requirements and is included in the Operation and Maintenance Manual.

11.12 Penstock Hydraulic Capacity

The penstock must decant the operating return water and, at the same time, draw down the stormwater storage within 72 hours (Table 9-2). Each cell is checked on its own, with the full tailings production rate in Table 9-6 delivered to it.

The operating return water is the slurry water less the water retained in the voids of the tailings at the target in-situ density in Table 11-1. Evaporation from the pool and the beach is ignored, which is conservative for the penstock.

The stormwater volume is the 1:100-year, 72-hour rainfall depth in Table 3-2 over the top surface of the cell, with no allowance for losses. The required decant rate is the daily operating return water plus the stormwater volume released over 72 hours.

Each intake is checked for weir flow over the precast ring ($Q = C_w \pi D H^{1.5}$) and for orifice flow through it ($Q = C_d A \sqrt{2gh}$), with a discharge coefficient C_d of 0.62 and a weir coefficient C_w of 1.83. The mechanism that needs the greater head governs. At the minimum operating pool depth of 300 mm the orifice governs, and the capacity of one intake at this depth sets the number of intakes required. The results for each cell are given in Table 11-4.

Table 11-4: Penstock Hydraulic Capacity per Cell

PARAMETER	UNIT	CELL 1	CELL 2	CELL 3
Slurry relative density / solids specific gravity	–	1.2 / 2.6	1.2 / 2.6	1.2 / 2.6
Slurry water delivered	m ³ /month	654 231	654 231	654 231
Water retained in voids	m ³ /month	58 413	58 413	58 413
Operating return water	m ³ /month	595 817	595 817	595 817
Operating return water as a proportion of slurry water	%	91	91	91
Top surface area reporting to the penstock	m ²	155 969	259 621	188 887
Stormwater volume	m ³	29 338	48 835	35 530
Operating return water rate	m ³ /s	0.227	0.227	0.227
Stormwater drawdown rate over 72 hours	m ³ /s	0.113	0.188	0.137
Required decant rate	m ³ /s	0.340	0.415	0.364
Intake ring internal diameter	m	0.52	0.52	0.52
Capacity of one intake at 300 mm pool depth, orifice control	m ³ /s	0.319	0.319	0.319
Intakes required	No.	2	2	2
Intakes provided	No.	2	2	2
Outfall pipeline, HDPE PE100 PN16	mm	500	500	500
Combined discharge to DW CH 01B, three cells decanting	m ³ /s	1.119		

If one of the two intakes in a cell is blocked or out of service, the other passes the required decant rate at a higher pool level, and the stormwater is held in the storm storage until the drawdown is complete. Each cell discharges through its own 500 mm HDPE PE100 PN16 outfall pipeline. The pipelines are

sized in iDAS with Manning's equation for full-bore flow, for the required decant rate of their cell on the surface gradient. The design flow, velocity, maximum capacity, slope, pipe length and percentage of full flow for each pipeline are given on the iDAS long sections in Annexure J. The combined discharge from the three cells is included in the design of DW CH 01B in Chapter 16.

Each cell has two decant towers on its own outfall pipeline. Blockages occur mostly at the towers rather than the outfall, so the second tower is the main redundancy. If the outfall pipeline of a cell is compromised, the pool is decanted by a floating pump or barge until the pipeline is repaired. The storm storage and dry freeboard in Table 9-2 are provided above the maximum operating level without relying on the decant, so the pool can hold the design storm while the emergency decant is put in place.

11.13 Access, Fencing and Ancillary Features

A gravel or dump rock access roadway is constructed along the perimeter of the TSF and ties into the constructed access road from the plant area. No gates are provided since the access road ties into the access-controlled road.

A pipeline for the slurry delivery leads from the crossover valve station on the western side of the site and runs around the TSF. A culvert crossing is constructed at each of these valve stations to allow vehicle access to the TSF across the solution trench.

The entire TSF complex is to be fenced, with access control provided by the Mine.

11.14 Seepage and Slope Stability

The seepage and slope stability analyses of the facility, with the sections analysed and the results, are given in the slope stability report in Annexure G. The analyses address the starter wall, the upstream raised embankment and the foundation against the loading conditions and minimum factors of safety given in Chapter 9.

11.14.1 Ground Model and Material Parameters

The foundation model was compiled from the test pit profiles of the geotechnical investigation (Chapter 4). The strength parameters of the transported soil were derived from the shearbox tests of the 2021 geotechnical investigation (section 4.3), and those of the other foundation materials were taken from geotechnical literature (Look, 2014). No tailings samples were available for testing, so the tailings parameters were assumed from experience on other chrome TSFs and from the literature on tailings impoundment design, with the peak and residual undrained shear strengths chosen conservatively. The parameters adopted are given in Table 11-5.

Table 11-5: Material Parameters Adopted in the TSF Stability Analyses

MATERIAL	$\gamma / \gamma_{\text{sat}}$ (kN/m ³)	kh / kv (m/s)	c' (kPa)	ϕ' (°)	$s_u(\text{peak})/\sigma'_{\text{vo}}$	s_u , min PEAK (kPa)	$s_u(\text{liq})/\sigma'_{\text{vo}}$	s_u , min LIQUEFIED (kPa)
Transported clayey sand (TR-S)	17 / 18	$1 \times 10^{-7} / 1 \times 10^{-7}$	0	27	0.40	40	0.25	30
Nodular ferricrete (F-N)	18 / 19	$1 \times 10^{-6} / 1 \times 10^{-6}$	1	32	Not applicable			
Residual harzburgite (RES-S)	18 / 19	$1 \times 10^{-7} / 1 \times 10^{-7}$	0.5	32	0.40	60	0.25	50
Very soft rock harzburgite (R-VS)	21.5 / 21.5	$1 \times 10^{-7} / 1 \times 10^{-7}$	Generalised Hoek-Brown, UCS 1.8 MPa, GSI 24					
Soft to medium hard rock harzburgite (R-SM)	23 / 23	$1 \times 10^{-9} / 1 \times 10^{-9}$	Generalised Hoek-Brown, UCS 10 MPa, GSI 38					
Starter wall	20 / 21	$1 \times 10^{-7} / 1 \times 10^{-7}$	0	32	Not applicable			
Tailings, overflow	16 / 17	$5 \times 10^{-8} / 1 \times 10^{-8}$	0	28	0.18	10	0.05	5
Tailings, underflow	16 / 17	$5 \times 10^{-7} / 1 \times 10^{-7}$	0	30	Not applicable			
Textured HDPE geomembrane and tailings interface	9.5 / 9.5	$1 \times 10^{-11} / 1 \times 10^{-11}$	Interface strengths as given in Table 11-6					

Maximum undrained strengths of 100 kPa (peak) and 80 kPa (liquefied) for TR-S, and 150 kPa (peak) and 100 kPa (liquefied) for RES-S.

The interface shear strengths of the liner system were taken from the statistical analysis of historical test results by Howell and Kirsten (2016), who distinguish between granular and cohesive soils. The sandy soil beneath the geomembrane was taken as granular, and the tailings above it as cohesive because they are fine grained. The peak and residual friction angles are given in Table 11-6. The lower residual angle, for the tailings interface, was used for both interfaces in the analyses, because the strain the geomembrane will undergo is not known.

Table 11-6: Interface Shear Strength of the Liner System (Howell and Kirsten, 2016)

INTERFACE	PEAK FRICTION ANGLE (°)	RESIDUAL FRICTION ANGLE (°)	MODELLED FRICTION ANGLE (°)
Textured HDPE and granular base preparation layer	30.51	26.42	19.47
Textured HDPE and tailings	20.65	19.47	19.47

The interface shear strengths are to be confirmed by testing on the materials used in the works, and the analyses checked against the confirmed values before deposition starts (section 11.7). The peak and liquefied undrained strength ratios and the horizontal and vertical permeabilities of the tailings are also to be confirmed by laboratory testing on representative tailings samples before construction.

11.14.2 Analysis Method

The analyses use the Morgenstern-Price limit equilibrium method (Morgenstern and Price, 1965), with a non-circular search for the critical slip surface, for the loading conditions in Table 9-3. A seepage analysis is run on each section first to generate the phreatic surface. As the shear behaviour of the tailings could not be tested, both effective stress and undrained strength analyses were done. A minimum beach length of 50 m between the edge of the supernatant pool and the crest of the TSF was assumed, and both the underflow and overflow tailings, together with the fine-grained foundation soils below the phreatic surface, were assumed to liquefy during a trigger event.

11.15 Operation and Deposition Strategy

The TSF is developed as an upstream construction, hydraulically deposited cyclone facility. Deposition consists of a hybrid system which makes use of a hydro-cyclone with the overflow directed through a spray bar.

The main delivery pipeline runs around the perimeter of the TSF at ground level and feeds the hydro-cyclones and spray bars located on the heel wall. The spray bar distributes the overflow tailings in a more controlled manner, which promotes steeper beach slopes. The coarse underflow fraction is deposited between the toe and starter walls and the heel wall, over the toe drain system, and the finer tailings run off towards the central region of the basin. Material settles through segregation and the supernatant water remains central to form the supernatant pond.

Once deposition rises above the starter wall, the coarser fraction is used to form the outer wall of the facility.

11.16 Instrumentation and Monitoring

The facility is to be instrumented so that the phreatic characteristics and the movement behaviour of the structure are monitored over its operating life. The monitoring system will measure the phreatic surface and pore pressures within the tailings and the foundation, the horizontal and vertical movement of the confinement walls, and the flows discharging from the drainage outfalls, and will be read against trigger levels set for the facility.

The instrumentation type, number and positions are determined during the construction phase, once the foundation conditions have been confirmed on site and the construction sequence is fixed. The instrumentation layout and the installation specification are issued as part of the construction documentation and the readings are reported under the operation, maintenance and monitoring requirements.

11.17 Closure and Cover

Sylvania operates its TSFs for the life of the facility, leaves each facility to stand for one to two years after deposition has stopped, and then remines it. The Volspruit TSF is expected to be remined in the same way. It is required that a remining plan is in place before remining starts, and the remining plan must ensure that the installed barrier and drainage system are not compromised. During the standing period the drainage system and the solution trench stay in operation. The closure concept that follows applies where the facility is not remined.

At closure, the TSF is covered with a multi-layered cover over the top surface and the outer slopes. The cover limits the infiltration of rainfall into the tailings, controls erosion and supports a self-sustaining vegetation cover, so that the closed facility remains physically stable and blends into the surrounding landscape. The concept cover build-up is given in Table 11-7.

The outer slopes are rehabilitated progressively as each lift is completed, so that no slope is left exposed for long. The completed slope is trimmed to the design profile, loose material is removed, and the slope cover is placed and vegetated before the next lift is raised. The outer slopes are flatter than 1V:3H (Table 11-1) and do not need to be reprofiled for closure. Contour berms on the benches interrupt the runoff and limit its velocity, and chutes convey it to the solution trench. Hydroseeding or erosion matting protects the slopes until the vegetation is established.

Where the TSF is not remined, it is closed in the following steps:

- the decant system is sealed;
- the top surface is shaped to fall towards a designated spill point, so that no pool forms on the facility;
- the top surface cover is placed, and the outer slope cover is completed where it has not been placed progressively;
- the cover is vegetated with indigenous, drought tolerant species; and

- the drainage system and the solution trench remain in operation until the monitored seepage quantity and quality confirm that they may be decommissioned.

Table 11-7: TSF Closure Cover Concept

ELEMENT	TOP SURFACE	OUTER SLOPES
Final profile	Fall of about 0.5% towards the spill point	Slopes as constructed (Table 11-1), with contour berms on the benches
Low permeability layer	Compacted clay or similar low permeability material, at least 500 mm thick	Compacted clay or similar low permeability material, 300 mm to 500 mm thick
Drainage and protection layer	Coarse grained, free draining material, 200 mm to 300 mm thick, draining laterally to the spill point and the toe	Coarse grained, free draining material, 200 mm to 300 mm thick, protecting the clay from drying and cracking
Growth medium	Topsoil from the construction stockpiles, 300 mm to 500 mm thick	Topsoil from the construction stockpiles, 200 mm to 300 mm thick
Vegetation	Indigenous, drought tolerant species selected from revegetation trials during operation	Indigenous, drought tolerant species, with hydroseeding or erosion matting until established

12 WASTE ROCK DUMP

12.1 Overview

The WRD receives the waste rock hauled from the North Pit. Waste rock from the South Pit is used to partially backfill the North Pit and is not placed on the WRD. The WRD is a dry facility that holds no water, and the only hydraulic head on its foundation develops from rainfall that infiltrates through the waste rock during storm events.

The design criteria for the WRD are given in Table 9-1 and its deposition method in Table 9-6. The waste rock classifies as Type 4 waste (Chapter 5), and the WRD is placed on the Class D foundation given in Table 10-2.

12.2 Layout, Capacity and Development

The WRD is located on the farm Zoetveld 294 KR, east of the North Pit and west of the local road (19-918-SWM-100). It is built in three lifts, each completed and benched before the next lift is started, to a flat final platform (19-918-SWM-151). The geometry of the WRD is given in Table 12-1.

Table 12-1: WRD Geometry

PARAMETER	VALUE
Final platform level	1 115.00 m
Lift heights	20 m for the first and second lifts, 10 m for the top lift
Lift side slopes	1V:2H
Bench width	5 m
Bench crossfall	2%
Bench paddock berms	500 mm high, with paddock rib berms at 20 m centres
Overall outer slope	1V:2.2H
Maximum height above natural ground	50 m
Design capacity	15 166 145 m ³
WRD footprint	463 539 m ² (46.4 ha)

The mine plan is still subject to change. The design capacity in Table 12-1 is therefore confirmed against the final mine plan before construction, and the WRD design is revised if the plan requires more capacity.

12.3 Foundation Preparation

The footprint of the WRD is to be cleared of all vegetation and the topsoil stripped and stockpiled for re-use during rehabilitation. Any soft, compressible or otherwise unsuitable material identified during construction is to be removed under the direction of the Engineer.

The WRD footprint is covered by a 0.8 m to 2.3 m thick layer of sandy transported soils, underlain in places by nodular ferricrete and by sandy residual soils over harzburgite and pyroxenite bedrock, which is present below the entire WRD area at depths of not more than 3.1 m (Peregrine Geoconsultants, PG19-003/673 Rev02, June 2021). The transported soils are prone to collapse settlement, expected to

be in the order of a 10% volume reduction on saturation. Because of the compressible and pinhole voided nature of these soils, the geotechnical investigation recommends that the layer is improved by dynamic impact rolling, or alternatively removed, before the foundation is prepared. The impact rolling approach recommended is:

- strip and stockpile all topsoil;
- strip and stockpile the remaining soil material to the required level from which the barrier system will be constructed;
- in the late afternoon, wet the exposed area copiously and allow it to stand overnight;
- using a multi-sided impact roller, dynamically compact the exposed surface to ensure densification to a depth of 1.5 m below the platform level; and
- rip the uneven surface created by the impact rolling to a depth of 200 mm and recompact it, using a smooth-drum roller capable of generating a centrifugal force of at least 275 kN operating in high-frequency mode, to 93% of Mod AASHTO density at 0% to +2% of OMC.

The extent, depth and method of treatment are to be confirmed by the detailed geotechnical investigation and approved by the Engineer.

No groundwater seepage was encountered in any of the test pits within the WRD footprint, which reached refusal at 1.2 m to 3.1 m below ground, and the monitoring boreholes recorded levels of 5 m to 15 m below ground (Annexure F). The 2 m unsaturated zone below the base of the foundation required by Note 10 to regulation 3(1) of GN R636 is therefore maintained.

12.4 Barrier System

The WRD is founded on the Class D foundation of GN R636 as amended, which has no liner (section 10.2). After the ground improvement, the exposed in-situ soil is scarified, ripped to a depth of 150 mm and recompact to 98% of Standard Proctor maximum dry density within 2% of OMC to form the base preparation layer (19-918-SWM-152).

The sub-surface drainage system required in the in-situ foundation by Note 10 to regulation 3(1) comprises a herringbone network of sub-surface drains below the full footprint of the WRD and a sub-surface cut-off drain along its full perimeter. The cut-off drain is a 450 mm deep geocomposite panel drain, or similar approved, installed vertically in a 1 300 mm deep trench, wrapped in a non-woven, needle punched geotextile (Table 10-4), bedded on Class I selected aggregate and backfilled with Class II coarse grained granular material. The position and depth of the drains are set on site to suit the in-situ ground conditions. The cut-off drain and the herringbone network discharge into manhole sumps at the northern and southern ends of the WRD, where the outflow is monitored (19-918-SWM-100 and 19-918-SWM-906).

The herringbone network uses the same pipes and details as the TSF sub-surface drainage (Table 11-3). The branch drains are Ø110 mm corrugated slotted HDPE sub-surface drainage pipes, class SN16 (ring stiffness 16 kN/m² to ISO 9969) or similar approved, in Class I crushed stone aggregate

wrapped in a non-woven, needle punched geotextile (Table 10-4). The branches are joined with the pipe manufacturer's couplers and connected to the mains with HDPE Y-bends and adaptors. The mains are Ø160 mm HDPE PE100 PN12.5 solid pipes, butt welded, on a bedding cradle of selected granular sand with a fineness modulus of not less than 2 and a compactibility factor not greater than 0.4.

12.5 Barrier Performance Verification

The base preparation layer is tested for density and moisture content during construction under the construction quality assurance plan (Chapter 20), and the ground improvement is accepted by the Engineer before the base preparation layer is placed.

In operation, the performance of the foundation is monitored through the outflow and water quality of the sub-surface drainage network and the cut-off drain at the manhole sumps, and through the groundwater monitoring boreholes around the facility. The results are compared with the leachable concentration limits on which the Type 4 motivation for the waste rock rests.

12.6 Drainage and Solution Collection

The WRD footprint is a dirty water catchment. Runoff from the WRD is collected at its toe by the concrete lined dirty water channels DW CH 02A and DW CH 02B, which run inside the perimeter containment berm and carry the flow through a silt trap to the WRD PCD (19-918-SWM-100 and 19-918-SWM-152). Clean runoff from the surrounding catchment is diverted around the WRD by the clean water channels CW CH 02A, CW CH 02B and CW CH 02C. The channels are designed in Chapter 16 and Chapter 17, the silt trap in Chapter 15 and the WRD PCD in Chapter 14.

Each bench is formed with a crossfall and a bench paddock berm along its outer edge, and is divided into paddocks by rib berms, so that rainfall on the lift slope above is held on the bench and does not run down the full height of the WRD (Table 12-1 and 19-918-SWM-151). The water held in the paddocks evaporates or infiltrates into the waste rock.

A containment berm runs along the outer edge of the perimeter dirty water channel and separates the dirty water catchment of the WRD from the surrounding clean catchment (19-918-SWM-152).

12.7 Slope Stability

The slope stability of the WRD is analysed by the Morgenstern-Price limit equilibrium method for the loading conditions in Table 12-2. The sections analysed and the results are given in the slope stability report in Annexure G.

12.7.1 Ground Model

The test pits relevant to the WRD are summarised in Table 4-4. The typical ground profile adopted in the analysis is sandy transported soil, nodular ferricrete and sandy residual soil over very soft rock harzburgite. No groundwater was encountered in any of the WRD test pits. As for the TSF, the parameters of the foundation soils were assumed from previous experience with similar soils (Table 11-5).

12.7.2 Waste Rock Strength

The waste rock is expected to comprise mainly harzburgite and pyroxenite particles of varying shapes, sizes and degrees of weathering, with sand and silt sized particles also present within the rockfill matrix. No rock dumps are present on site, so no laboratory or in-situ strength testing was done on the waste rock, and its strength was taken from the literature. Hawley and Cunning (2017) consider the shear strength of rockfill to be the most important property affecting the stability of a dump, and also the most difficult to determine, because of the wide range of particle sizes and shapes and the change in strength over time with densification and degradation. From the results of large-scale triaxial tests, they present a series of relationships between normal stress and shear strength for rockfill. Because of the uncertainty in the strength and density of the waste rock and the finer particles within the matrix, the average strength curve was adopted.

Unlike the TSF, the WRD has no continuous geomembrane, so a geosynthetic interface check is not required. The contact between the waste rock and the foundation, and the transported soil, are represented in the ground model.

12.7.3 Loading Conditions

In line with the draft revision of SANS 10286 (2022) (section 2.5), two loading conditions were analysed for the WRD in its final landform at full design height, which is its most critical shape for slope stability (Table 12-2).

Table 12-2: WRD Loading Conditions and Minimum Factors of Safety

SCENARIO	LOADING CONDITION	MINIMUM FACTOR OF SAFETY
1	Long-term, drained, static	1.50
2	Earthquake, pseudo-static, drained, peak ground acceleration of 0.08 g	1.10

The coarse grained, voided waste rock is expected to drain freely with minimal build-up of pore water pressure, and drained strengths were used. A pore pressure ratio ru of 0.15 was nevertheless applied to the waste rock and to the foundation soils. In the earthquake condition, the peak ground acceleration of 0.08 g was applied as a pseudo-static force. The waste rock is considered coarse enough for excess pore pressures to dissipate rapidly, so drained strengths were also used, with the same groundwater conditions as the long-term condition.

12.8 Closure and Cover

The WRD is rehabilitated progressively, with the benches and lift slopes vegetated as each lift is completed (19-918-SWM-151). At closure, the final platform is covered with the topsoil stripped and stockpiled during construction and vegetated with an indigenous grass cover suited to the site. The perimeter channels, the silt trap and the WRD PCD remain in operation after closure until the monitored runoff quality confirms that they may be decommissioned.

13 RUN-OF-MINE PAD

13.1 Overview

The RoM pad stores the run-of-mine ore from the pits before it is crushed at the primary crusher and conveyed to the concentrator plant. The RoM pad is a dry facility, and the only hydraulic head on its barrier develops from rainfall runoff across the slab during storm events.

The design criteria for the RoM pad are given in Table 9-1. The ore is classified as Type 3 waste, and the RoM pad is a Class C facility (Table 10-2).

13.2 Layout and Layerworks

The RoM pad is located between the North Pit and the plant, west of the RWD (19-918-SWM-200). It is a four-sided platform that follows the natural ground (19-918-SWM-250). The geometry of the RoM pad is given in Table 13-1.

Table 13-1: RoM Pad Geometry

PARAMETER	VALUE
Platform area	16.2 ha
Platform levels	1 068.80 m to 1 073.80 m
Length on Alignment A-A	360 m
Length on Alignment B-B	500 m
Minimum fall of the slab	1%

The RoM area is covered by a 0.65 m to 1.6 m thick layer of sandy transported soils, underlain in one test pit by sandy residual soil, over poor quality harzburgite and pyroxenite rock that hardens rapidly with depth, with refusal at 0.9 m to 2.7 m below ground (Peregrine Geoconsultants, PG19-003/673 Rev02, June 2021). The transported soil in test pit ROM-TP01 showed a pinhole voided structure, which may be susceptible to collapse, and the foundation preparation must take this into account. No groundwater seepage was encountered in the test pits. The pad foundation is therefore treated in the same way as the WRD foundation (section 12.3) before the layerworks are placed.

The layerworks of the RoM pad are given in Table 13-2 (19-918-SWM-904).

Table 13-2: RoM Pad Layerworks

ELEMENT	SPECIFICATION
Concrete slab	200 mm reinforced concrete, water retaining, to the concrete specification in Table 19-1, laid to the minimum fall in Table 13-1 towards the perimeter channels. Minimum cover to the reinforcement is 50 mm at the bottom and 40 mm at the top
Joints	Keyed construction joints and dowelled expansion joints, with external polyvinyl chloride (PVC) waterstops and a polyurethane joint sealant over a closed cell polyethylene backing cord. The slab is cast on a 250-micron polyethylene sheet bond breaker, with C15/19 blinding below the waterstops
Subbase	150 mm C4 stabilised natural gravel, compacted to 98% of Mod AASHTO density
Selected layer	150 mm G7 natural gravel, compacted to 95% of Mod AASHTO density

ELEMENT	SPECIFICATION
In-situ soil	Wetted copiously, ripped to a depth of 200 mm and recompacted to 95% of Mod AASHTO density at OMC 0% to +2%
Sub-surface drainage	Ø110 mm corrugated slotted HDPE sub-surface drainage pipes, class SN16 (ring stiffness 16 kN/m ² to ISO 9969) or similar approved, in 300 mm by 300 mm trenches of 19 mm washed stone, wrapped in a non-woven, needle punched geotextile (Table 10-4), laid in a herringbone pattern at 20 m spacing beneath the pad and discharging into a sump manhole (19-918-SWM-250 and 19-918-SWM-906)

13.3 Barrier System

The reinforced concrete slab forms the barrier of the RoM pad. It replaces the geomembrane and CCL of the prescribed Class C barrier and is motivated as a Class C equivalent barrier under regulation 3(2)(d), on the same basis as the concrete lining of the Plant PCD (section 14.5). The pad holds no standing water, so the head on the slab is limited to runoff during storms, and the joints, where leakage could occur, are sealed and fitted with waterstops. The structural design of the slab is completed as part of the pre-construction design set. It covers the loads from end-tipping haul trucks, front-end loaders and the ore stockpile, the slab thickness and reinforcement, crack width control to tightness class 1 of BS EN 1992-3, and the joint spacing and bay layout.

The subbase and selected layer below the slab form the base preparation layer, and the sub-surface drainage system in the in-situ foundation relieves any uplift pressure and collects any leakage through the slab.

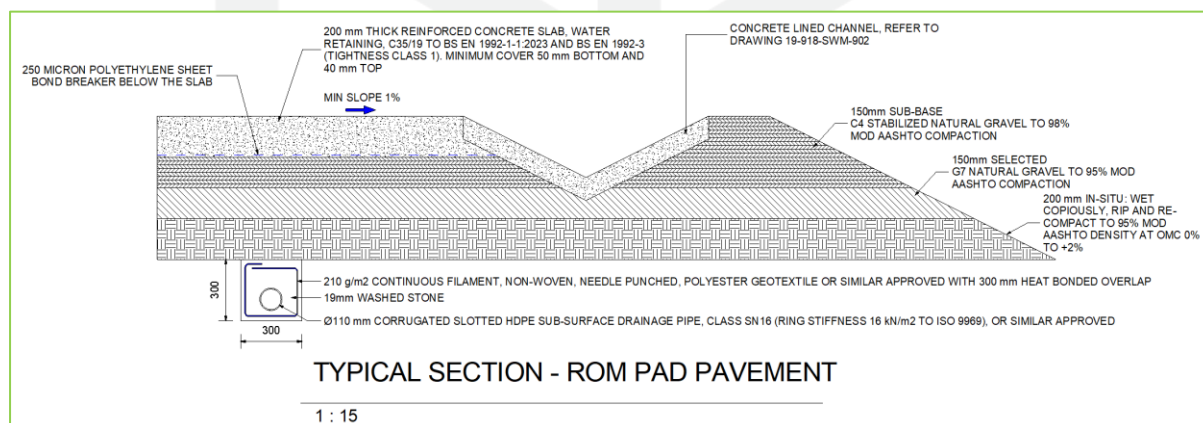


Figure 13-1: RoM Pad Layerworks

13.4 Drainage and Solution Collection

The RoM pad is a dirty water catchment. Runoff from the slab is shed to the concrete lined dirty water channels DW CH 05 and DW CH 05-1 along its edges, which carry the flow through a silt trap to the RoM PCD (19-918-SWM-200). The containment berms around the pad keep clean runoff off the slab. The channels are designed in Chapter 16, the silt trap in Chapter 15 and the RoM PCD in Chapter 14.

The sub-surface drains beneath the pad discharge into a sump manhole fitted with an ultrasonic water level meter, where the outflow is monitored (19-918-SWM-250 and 19-918-SWM-906).

13.5 Operation and Closure

Ore is end-tipped on the pad by the haul trucks and reclaimed by front-end loader for crushing. The slab, the joint sealant and the perimeter channels are inspected regularly and after every significant storm. Damaged concrete and failed joint sealant are repaired, and silt is removed from the channels and the silt trap so that the barrier and the runoff collection remain effective.

At closure, the pad is cleared of ore, and the slab and layerworks are removed and the footprint rehabilitated as set out in the mine closure plan.



14 RETURN WATER, POLLUTION CONTROL AND IRRIGATION DAMS

14.1 Overview

This chapter covers the water containment dams on the mine: the three cells of the RWD and the irrigation dam, which share one embankment, and the WRD, RoM and Plant PCDs. All the dams hold dirty water and are Class C facilities. The RWD, the irrigation dam and the WRD and RoM PCDs, referred to in this chapter as the HDPE lined dams, share one earth fill embankment design and one geomembrane barrier system (section 14.4). The Plant PCD is a reinforced concrete lined basin (section 14.5). Requirements common to all the dams are described once and apply to every dam unless a difference is stated.

The water in the irrigation dam classifies as Type 3 (Chapter 5), and the water in the RWD and the PCDs is treated as Type 3. Every dam therefore carries a Class C equivalent barrier. This includes the irrigation dam, which receives water from RWD Cell 3 through the RoM PCD as well as pit water, and carries the HDPE barrier described in section 14.4.

The dam capacities are based on the water balance (Chapter 6), which shows no spill from any dam over the simulation period with the dams operated below their maximum operating levels. On this basis the capacities are considered adequate for the 1:50-year storm. This is conditional on the re-run of the water balance on the updated production plan before construction, which sets the maximum operating level of each dam so that the 1:50-year, 24-hour storm is held between the maximum operating level and FSL (Table 9-1).

14.2 Dam Safety Registration and Classification

The RWD cells and the irrigation dam are compartments of one embankment with a maximum wall height of 4.50 m, and each compartment holds more than 50 000 m³. The WRD PCD holds more than 50 000 m³ behind a wall with a maximum height of 1.34 m. The RoM and Plant PCDs each hold less than 50 000 m³.

All the dams store polluted water. Apart from the size criteria in section 117(c)(i) of the National Water Act, a dam is a dam with a safety risk if it belongs to a category of dams declared under section 118(2), or if it is declared as such under section 118(3)(a). In making a declaration, the Minister must consider the need to protect the resource quality against the potential hazard posed by the dam (section 121(a)). Once a dam is declared, the owner must register it within 120 days (section 120(2)), and regulation 2(3)(b) of GN R139 requires its potential adverse impact on resource quality to be regarded as significant when its hazard potential is classified. The Dam Safety Office may therefore require the RWD, the irrigation dam and the PCDs to be registered on account of their pollution potential.

The dams are to be submitted to the Dam Safety Office before construction starts, with the information in Table 14-1, and the outcome is to be recorded in this section. A dam that is registered is classified and licensed in the same way as the TSF (section 11.2).

Table 14-1: Dam Safety Information

PARAMETER	RWD AND IRRIGATION DAM	WRD PCD	RoM PCD	PLANT PCD
Type of dam	Earth fill embankment, four compartments	Earth fill embankment, two cells	Earth fill embankment	Reinforced concrete basin with vertical walls
Lining	Class C equivalent HDPE barrier (section 14.4)	Class C equivalent HDPE barrier (section 14.4)	Class C equivalent HDPE barrier (section 14.4)	Class C equivalent reinforced concrete lining (section 14.5)
Province and nearest town	Limpopo, Mokopane	Limpopo, Mokopane	Limpopo, Mokopane	Limpopo, Mokopane
Status	Proposed	Proposed	Proposed	Proposed
Purpose	RWD: storage of TSF return water and, in Cell 3, South Pit dewatering. Irrigation dam: storage of North Pit dewatering and excess water from RWD Cell 3 for irrigation supply	Containment of dirty runoff from the WRD	Containment of dirty runoff from the RoM pad	Containment of dirty runoff from the plant area
Water balance capacity (m³)	Cell 1: 119 091 Cell 2: 116 613 Cell 3: 118 840 Irrigation dam: 100 842	63 340	20 900	35 095
Gross capacity at FSL (m³)	Cell 1: 152 949 Cell 2: 188 092 Cell 3: 202 211 Irrigation dam: 102 907	63 345	20 900	38 097
Footprint (m²)	229 345	26 078	11 727	11 746
Footprint (ha)	22.9	2.6	1.2	1.2
Area at FSL (m²)	188 488	18 566	7 741	10 583
Crest level (m)	1 079.30	1 064.52	1 067.90	1 071.60
FSL (m)	1 078.10	1 063.50	1 066.70	1 070.40
Freeboard (m)	1.20	1.02	1.20	1.20
Maximum wall height above foundation (m)	4.50	1.34	0.95	1.70
Crest width (m)	5	5	5	Not applicable, vertical walls

PARAMETER	RWD AND IRRIGATION DAM	WRD PCD	RoM PCD	PLANT PCD
Crest length (m)	1 992.8	674	425	436.5
Embankment slopes	1V:3H upstream and downstream	1V:3H upstream and downstream	1V:3H upstream and downstream	Vertical reinforced concrete walls
Emergency spillway	Reinforced concrete lined, one per compartment, 21 m wide, with inter-cell overflow spillways	Reinforced concrete lined, 16 m wide, with an 8 m wide inter-cell spillway	Reinforced concrete lined, 30 m wide	Reinforced concrete lined, 15 m base width
Emergency spillway capacity (m³/s)	30.6 per compartment	16.7	43.7	21.8
Coordinates (centre of dam) (WGS84)	24°20'52.41" S, 28°57'43.83" E	24°20'01.59" S, 28°57'15.58" E	24°21'03.73" S, 28°57'09.18" E	24°21'19.91" S, 28°57'21.33" E
Drawing reference	19-918-SWM-500 to 19-918-SWM-508	19-918-SWM-110	19-918-SWM-210	19-918-SWM-310

14.3 Embankment Design

The embankments of the HDPE lined dams are homogeneous earth fill, built from selected material from the basin excavations. Their inside and outside slopes are 1V:3H. The RWD and RoM PCD crests are 5 m wide (19-918-SWM-500 and 19-918-SWM-210). The layout and sections of the Plant PCD are shown on 19-918-SWM-310.

The footprint is stripped of topsoil to a depth of 150 mm to 300 mm, and the topsoil is stockpiled for rehabilitation. The foundation level is approved by the Engineer before the in-situ material below the embankments is ripped and compacted to 93% of Mod AASHTO density. The embankment fill is placed in layers not exceeding 200 mm after compaction and compacted to 95% of Standard Proctor maximum dry density at OMC 0% to +2% (ASTM D698). Excavated faces in the basins are ripped and recompacted to a depth of 200 mm. The compacted density, permeability, cohesion and friction angle of the fill are to be confirmed by laboratory testing and submitted to the Engineer before construction.

14.4 HDPE Barrier System

The HDPE lined dams are lined with a Class C equivalent barrier. The geomembrane thickness is increased from the 1.5 mm of the prescribed Class C barrier to 2 mm, and the CCL is omitted. The geomembrane is placed on a receiving face of 200 mm of imported fine selected material prepared to SANS 10409 (2020), over a foundation preparation layer of 200 mm of in-situ soil ripped and recompacted. It is covered by soilcrete filled geocells on the slopes and a soilcrete ballast on the floor, which protect it from ultraviolet light, mechanical damage and uplift. A sub-surface drainage system in the foundation relieves groundwater pressure below the liner, and its outflow is monitored as an

indication of leakage through the geomembrane. The motivation for the Class C equivalent barrier and the material specifications is given in Chapter 10.

The available groundwater levels indicate that the water table lies well over 2 m below the base of the dam liners, as required by Note 10 to regulation 3(1) of GN R636. The rest water levels measured in the site boreholes were 8.75 m to 13.93 m below ground (Annexure C), the monitoring boreholes recorded levels of 5 m to 15 m (Annexure F), and no seepage was encountered in any of the test pits. The sub-surface drains beneath the dams relieve any uplift pressure, and their outflow is monitored.

The build-up of the barrier system, from the top down, is given in Table 14-2.

Table 14-2: HDPE Barrier System Build-up

ELEMENT	SPECIFICATION
Floor cover	150 mm layer of 5 MPa soilcrete ballast, maximum particle size 3 mm
Slope cover	150 mm deep geocells filled with 5 MPa soilcrete, maximum particle size 3 mm
Slope reinforcement	RockGrid KX 60/30 by Kaytech, or similar approved: a knitted polyester geogrid with a polymeric coating and an ultimate tensile strength of 60 kN/m in the machine direction. Laid with the machine direction downslope, in continuous lengths from the anchor trench to the toe of the slope, with no transverse joints. Provided for veneer stability of the slope cover only (section 14.6)
Geomembrane	2 mm smooth HDPE geomembrane to SANS 1526 (2015), with manufacturing quality control as per GRI-GM13 and welds as per GRI-GM19, installed in accordance with SANS 10409 (2020)
Receiving face	200 mm of imported fine selected material compacted to 98% of Standard Proctor maximum dry density at OMC 0% to +2%, in layers not exceeding 150 mm. The finished surface is smooth and free of protrusions that could puncture the geomembrane, with a maximum particle size of 3 mm, in accordance with SANS 10409 (2020)
Foundation preparation layer	200 mm of in-situ soil ripped and recompacted to 98% of Standard Proctor maximum dry density at OMC 0% to +2%
Subsurface drainage	<p>Ø110 mm corrugated slotted HDPE sub-surface drainage pipes, class SN16 (ring stiffness 16 kN/m² to ISO 9969) or similar approved, in the basin and along the perimeter of the dam, bedded in Class I aggregate and backfilled with 19 mm washed stone, in trenches lined with a 210 g/m² non-woven needle punched geotextile. Ø160 mm HDPE PE100 PN12.5 solid collector pipes, butt welded, report to the sub-surface drainage sump manhole (19-918-SWM-906 and 19-918-SWM-907).</p> <p>The sump manhole is built of factory-made concrete sections to SANS 1294 on a C35/19 concrete foundation, with any over-excavation filled with C20/19 mass concrete and is fitted with an ultrasonic water level meter.</p>
Anchorage	Anchor trench 850 mm wide and 1 000 mm deep on the embankment crest, 1.2 m back from the top of the inside slope, backfilled with selected material stabilised with cement to a minimum strength of 5 MPa (testing on 19-918-SWM-900)
Tie-in to concrete	Stainless steel flat bar and EPDM rubber strip fixing, as described for the TSF in section 11.6, with a 1 000 mm wide, 150 mm thick soilcrete strip at the make-off to limit strain from differential settlement
Leak location survey	Dipole survey in accordance with ASTM D8265 or ASTM D7007, after the 5 MPa soilcrete cover has been placed (section 10.7)

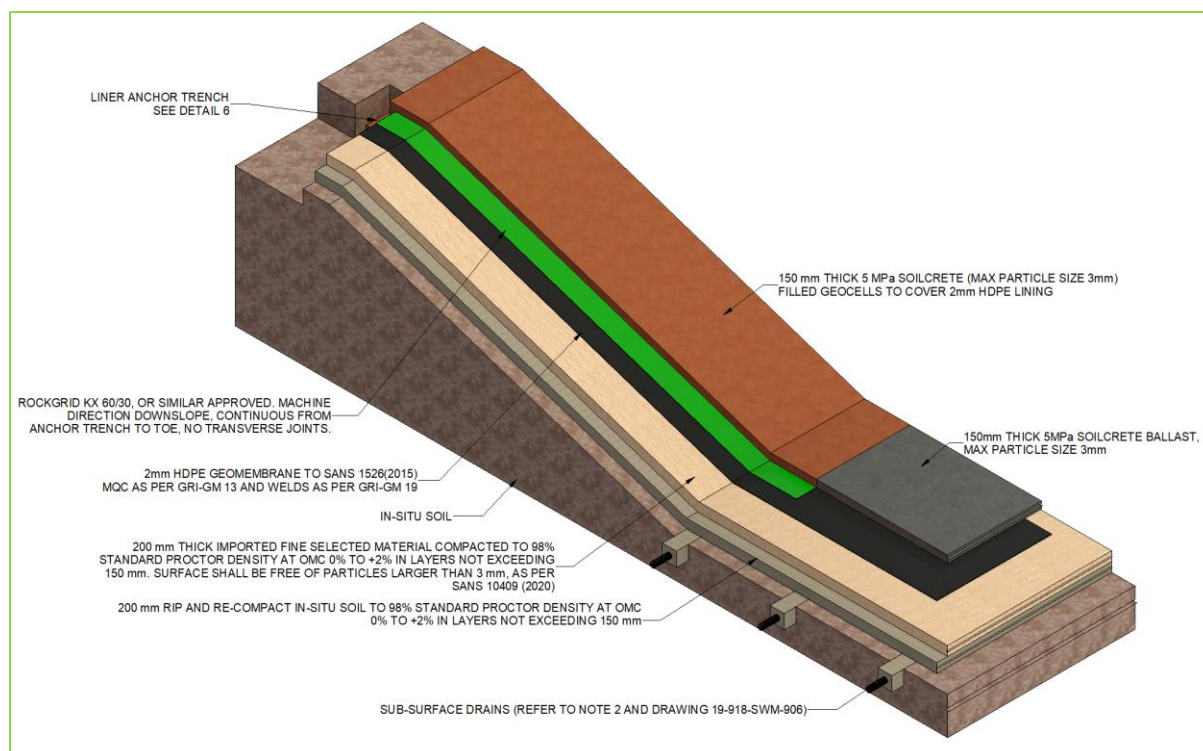


Figure 14-1: Typical Dam Lining Detail

The receiving face is inspected over its full area and accepted by the Engineer and the geomembrane installer before the geomembrane is placed.

The following are confirmed on the materials used in the works before the HDPE lined dams are filled:

- the interface shear strength between the dam geomembrane and the geogrid with its soilcrete infill (ASTM D5321), against the values assumed in section 14.6;
- the hydraulic conductivity of the compacted receiving face beneath the dam geomembranes, against the value assumed in section 10.9; and
- the unconfined compressive strength of the soilcrete and cement stabilised material, of at least 5 MPa at 7 days (SANS 3001-GR53).

Where a confirmed value is less favourable than the value assumed in the design, the analysis concerned is re-run, and the works do not proceed until the Engineer has accepted the result.

14.5 Plant PCD Concrete Lining

The Plant PCD lies within the plant area and is lined with reinforced concrete designed as a water retaining structure. The concrete lining forms the Class C equivalent barrier for the facility. A sub-surface drainage system of Ø110 mm corrugated slotted HDPE branch drains and Ø160 mm solid HDPE collectors beneath the basin, reporting to a sump manhole (19-918-SWM-311 and SWM-906), relieves groundwater pressure, and its outflow is monitored as an indication of leakage through the lining. The groundwater position given in section 14.4 applies equally to the Plant PCD. The structural design of the basin is completed as part of the pre-construction design set. It covers the water and earth pressures

on the walls, uplift when the basin is empty, the floor and wall thickness and reinforcement, crack width control to tightness class 1 of BS EN 1992-3, and the joint spacing and bay layout. The build-up of the lining is given in Table 14-3.

Table 14-3: Plant PCD Concrete Lining Build-up

ELEMENT	SPECIFICATION
Concrete lining	Reinforced concrete, water retaining to BS EN 1992-1-1:2023 and BS EN 1992-3 (tightness class 1), C35/19 with CEM I or CEM II 42.5N cement to SANS 50197-1, a water:cement ratio not exceeding 0.5 and a minimum cement content of 325 kg/m ³ , with a minimum cover of 40 mm (19-918-SWM-310). Slab thickness and mesh reference as given on the drawings
Blinding	C15/19 concrete
Layerworks	Imported G7 material compacted to 95% of Mod AASHTO density, in layers not exceeding 150 mm
Foundation	In-situ material of the side slopes and floor shaped to line and level and compacted in a 150 mm layer to 93% of Mod AASHTO density
Sub-surface drainage	Ø110 mm corrugated slotted HDPE sub-surface drainage pipes, class SN16 (ring stiffness 16 kN/m ² to ISO 9969) or similar approved, in trenches of 19 mm washed stone wrapped in a 210 g/m ² non-woven needle punched geotextile, joined to Ø160 mm HDPE PE100 PN12.5 solid collectors reporting to a sump manhole (19-918-SWM-311 and SWM-906)
Joints	Construction joints positioned and sealed as detailed on the drawings
Leak monitoring	Outflow from the sub-surface drainage system. An electrical leak location survey does not apply to the concrete lining

The concrete lining replaces the geomembrane and CCL of the prescribed Class C barrier and is motivated as a Class C equivalent barrier under regulation 3(2)(d). The same basis applies to the silt traps (Chapter 15), which are also water retaining concrete structures with sub-surface drainage. Equivalence is demonstrated on the following points:

- leakage: the concrete is designed as a water retaining structure to tightness class 1 of BS EN 1992-3. Crack widths are limited to between 0.2 mm and 0.05 mm, depending on the ratio of water head to section thickness, so that through cracks self-heal, and all joints are fitted with waterstops. Leakage is controlled by these limits rather than calculated (section 10.9), and is verified by the watertightness test and the sub-surface drainage monitoring described below;
- service life: the concrete mix, cover and cement type are selected for the exposure class set by the dirty water quality, so that the lining outlasts the 44.6-year polluting period;
- monitoring: the sub-surface drains beneath each structure report to a manhole sump, where the outflow is recorded. A baseline is set during the first wet season after commissioning. A sustained rise above the baseline, outflow that rises and falls with the water level in the structure, or outflow with the quality of the contained water triggers an inspection of the joints and cracks, repair and a repeat of the watertightness test. The prescribed Class C barrier has no equivalent means of detecting leakage below the liner; and

- construction quality: each structure is subjected to a hydrostatic watertightness test before commissioning, with a drop in water level of not more than 1/500 of the water depth, or 10 mm, over 7 days.

14.6 Veneer Stability

The cover on the embankment slopes must stay in place for the life of the dams without sliding on the smooth geomembrane. Its stability was assessed by the veneer stability method of Koerner and Soong (1998), as set out in Koerner (2012). The method compares the forces resisting sliding of the cover along the weakest interface with the forces driving it down the slope and includes the passive resistance of the wedge at the toe. The weakest interface in the system is between the smooth geomembrane and the overlying geogrid and soilcrete.

The analysis was run for the inside slopes of the RWD, the WRD PCD and the RoM PCD, each from the crest to the lowest floor level of the dam on the 1V:3H slope. The irrigation dam slopes are shorter than the RWD slopes and are covered by the RWD case. The parameters adopted are given in Table 14-4.

Table 14-4: Veneer Stability Parameters

PARAMETER	VALUE	BASIS
Unit weight of the cover	21 kN/m ³	Assumed typical value for soilcrete filled geocells
Friction angle of the cover	38°	Conservative value for 5 MPa cement stabilised soil
Cohesion of the cover	0 kPa	Conservatively ignored
Cover thickness	0.15 m	Depth of the geocells
Slope angle	18.4° (1V:3H)	Inside slope of the HDPE lined dams
Slope length	16.3 m to 24.1 m	Crest level (Table 14-1) to the lowest floor level of each dam in the current design models, with RWD Cell 2 governing for the RWD and irrigation dam (Table 14-5)
Interface friction angle	9°	Residual value for a non-woven needle punched geotextile on smooth HDPE (Koerner and Narejo, 2005), adopted as a lower bound for the geogrid and soilcrete on smooth HDPE until it is confirmed by the ASTM D5321 test (section 14.4)
Interface adhesion	0 kPa	Conservatively ignored
Reinforcement force, Tall	Allowable strength of the specified geogrid	Ultimate tensile strength of 60 kN/m divided by a cumulative reduction factor of 2.4 for installation damage, creep and durability, to be confirmed against the supplier's data for the specified geogrid. The force needed for the minimum factor of safety is also solved
Anchor trench	850 mm wide and 1 000 mm deep, with a 1.2 m runout under 0.15 m of cover	Table 14-2. Runout length and cover thickness assumed

PARAMETER	VALUE	BASIS
Interface friction angle in the anchor trench	20°	Assumed for the geogrid against the soilcrete backfill and the underlying soil
Earth pressure on the anchor trench walls	At rest, $K_0 = 1 - \sin \phi'$	Qian, Koerner and Gray (2002), with the friction angle of the cover
Minimum factor of safety	1.5	Adopted for a permanent cover

The factor of safety is the positive root of $a \cdot FS^2 + b \cdot FS + c = 0$, with the coefficients defined by Koerner and Soong (1998) for a cover of uniform thickness on a finite slope, and with the reinforcement force acting parallel to the slope. The factor of safety with reinforcement is calculated with the allowable strength of the specified geogrid, and the force needed for the minimum factor of safety is solved from the same equation. The pull-out capacity of the crest anchor trench is calculated by the method of Qian, Koerner and Gray (2002). The results are given in Table 14-5.

Table 14-5: Veneer Stability Calculations for the HDPE Lined Dams

TERM	RWD	WRD PCD	RoM PCD
Slope length, L (m)	24.13	17.74	16.25
Factor of safety without reinforcement	0.53	0.55	0.55
Allowable tensile strength, T_{all} (kN/m)	25.00	25.00	25.00
Factor of safety with reinforcement	> 5	> 5	> 5
Force needed for the minimum factor of safety of 1.5, T_{req} (kN/m)	15.56	11.21	10.20
Allowable strength over the required force, T_{all} / T_{req}	1.61	2.23	2.45
Pull-out capacity of the anchor trench, T_{MAX} (kN/m)	24.16	24.16	24.16
Pull-out capacity over the required force, T_{MAX} / T_{req}	1.55	2.16	2.37

The cover is not stable without reinforcement on any of the dams. It relies on the geogrid, anchored in the crest trench, to reach the minimum factor of safety (Table 14-5). At its allowable strength the geogrid can carry more than the net downslope force of the active wedge on each dam. The coefficient a then becomes negative, the equation has no positive root, and the geogrid holds the cover on its own. The design is therefore checked against the force needed for the minimum factor of safety. The allowable strength of the geogrid and the pull-out capacity of the anchor trench both exceed this force on every dam.

The geogrid carries the downslope component of the cover weight. Because the interface between the smooth geomembrane and the cover is the weakest in the system, little shear is transferred into the geomembrane, and the tensile strain induced in it by the cover is not significant. At the make-off to concrete structures, strain is limited by the soilcrete strip described in Table 14-2.

14.7 Spillways

Each compartment has an emergency spillway set at FSL and lined with 150 mm thick reinforced concrete (19-918-SWM-112, SWM-113, SWM-212, SWM-312 and SWM-504). On the HDPE lined dams the geomembrane is fixed to the spillway slab. The spillway outlets are protected by 300 mm thick Reno mattresses on a 210 g/m² non-woven geotextile. The RWD cells and the irrigation dam are connected by inter-cell overflow spillways (19-918-SWM-508), and the two WRD PCD cells by an 8 m wide overspill weir (19-918-SWM-114). The emergency spillway widths and capacities are given in Table 14-1. The capacity is calculated by the broad-crested weir equation at a head that leaves 0.3 m freeboard to the embankment crest, and each spillway passes the 1:100-year peak inflow from its catchment with a wide margin.

14.8 Stability

The slope stability of the dam embankments of the RWD and irrigation dam, the WRD PCD, the RoM PCD and the Plant PCD, with the sections analysed and the results, is given in the slope stability report in Annexure G.

The analyses use the Morgenstern-Price limit equilibrium method. The foundation profile was taken from the test pits of the geotechnical investigation (Chapter 4), in which no groundwater seepage was encountered. The strength parameters of the in-situ soils and rock were based on and adapted from the 2022 stability assessment of the TSF and WRD (section 11.14), with the transported soil parameters derived from the shearbox tests (section 4.3) and the berm parameters taken from geotechnical literature (Look, 2014). The embankment fill was taken as a clayey gravelly sand. Its parameters are confirmed by the laboratory testing of the fill required in section 14.3 before it is placed (section 19.1). The parameters adopted are given in Table 14-6.

Table 14-6: Material Parameters Adopted in the Dam and Flood Diversion Berm Stability Analyses

MATERIAL	BULK UNIT WEIGHT (kN/m ³)	ϕ' (°)	c' (kPa)	PEAK s_u/σ'_{vo}	PEAK s_u , min / s_u , max (kPa)	RESIDUAL s_u/σ'_{vo}	RESIDUAL s_u , min / s_u , max (kPa)
Transported clayey sand and silty sand with gravel (TR-S)	17	27	0	0.40	40 / 100	0.25	30 / 80
Nodular ferricrete, gravelly clayey sand (F-N)	18	32	1	Not applicable			
Residual clayey gravelly sand (RES-S)	18	32	0.5	0.40	60 / 150	0.25	50 / 100
Dam embankment fill, clayey gravelly sand	18	32	0	Not applicable			
Flood diversion berm, compacted rock fill (D50 = 300 mm)	21	38	0	Not applicable			
Flood diversion berm clay core	16	28	1	0.20	15 / –	–	–
Very soft to soft rock harzburgite (R-VS)	21.5	Generalised Hoek-Brown: UCS 5 MPa, GSI 24, mi 20, D 0.7					
Soft to medium hard rock harzburgite (R-SM)	23	Generalised Hoek-Brown: UCS 10 MPa, GSI 38, mi 20, D 0.7					

The loading conditions and minimum factors of safety were taken from the Dam Safety Guidelines of the Canadian Dam Association (CDA, 2013) and are given in Table 9-4. No site-specific seismic hazard assessment has been carried out. The peak ground acceleration of 0.08 g (section 3.7) was halved to a pseudo-static coefficient of 0.04 g, following Hynes-Griffin and Franklin (1984), as none of the conditions under which the acceleration should not be halved applies at Volspruit: the area is not subject to earthquakes of magnitude 8.0 or greater, the embankment and foundation materials are not considered susceptible to liquefaction under the design cyclic loading, and the dams do not have a small freeboard or safety features vulnerable to small deformations.

The assessment applies to the current geometry, material properties and loading conditions, and is to be reviewed if any of these change.

Rapid drawdown was not analysed. It is often not considered for dams lined with a geomembrane, because of the low permeability of the liner. It becomes a realistic condition if holes form in the liner during construction or through damage during desilting, so that water seeps into the embankment and develops a steady-state phreatic surface, which a rapid emptying of the dam would then leave behind. The integrity of the geomembrane is confirmed by the dipole survey after the soilcrete cover has been placed (section 10.7), and the 150 mm soilcrete cover protects it from damage during the operational life of the dams. Together these limit the possibility of a substantial leak that would lead to a phreatic surface within the embankment, and without a phreatic surface in the embankment, rapid drawdown is not a feasible loading condition.

14.9 Operation and Monitoring

The dams are operated below the maximum operating levels confirmed by the re-run of the water balance (section 14.1), so that the storm storage and the freeboard above FSL are available at all times. Monitoring of the dams comprises:

- the water level in each dam, read against a gauge plate marked with the maximum operating level and the FSL;
- the water level in each sub-surface drainage sump manhole, read on its ultrasonic level meter and recorded so that an increase that may indicate leakage through the geomembrane is picked up early;
- visual inspection of the embankments, spillways, geocell cover and ballast, and of the Plant PCD concrete lining after every significant storm and at least monthly; and
- the silt level in the basins, with silt removed by methods that do not damage the ballast, the geomembrane or the concrete lining.

Any dam registered with the Dam Safety Office is also inspected by the approved professional person at the intervals required by GN R139.

15 SILT TRAPS

15.1 Overview

Each dirty water collection system enters its receiving dam through a concrete silt trap. The silt traps settle out the sediment carried by the runoff from the dirty areas, which limits the loss of storage in the RWD and the PCDs and protects the dam barrier systems from abrasion and from the load of accumulated silt. Four silt traps are provided, upstream of the RWD, the WRD PCD, the RoM PCD and the Plant PCD.

The settling area of each silt trap is sized to retain particles of 0.2 mm and larger at the 1:10-year peak inflow, in line with the design criteria in Table 9-1. Above that flow the traps continue to operate at a lower removal efficiency, and the overflow weir passes the 1:50-year peak inflow into the receiving dam. The silt traps form part of the dirty water system and are constructed as water retaining concrete structures. Their structural design is completed as part of the pre-construction design set. It covers the water and earth pressures on the walls, the load of the TLB on the access ramp and floor during cleaning, uplift when a trap is empty, the floor and wall thickness and reinforcement, crack width control to tightness class 1 of BS EN 1992-3, and the joint layout. A sub-surface drainage system beneath each silt trap relieves groundwater pressure below the slab and reports to a manhole sump, where its outflow is monitored together with that of the receiving dam. The silt traps are motivated as a Class C equivalent barrier on the same basis as the Plant PCD concrete lining, set out in section 14.5.

15.2 Silt Trap Design

The design of the silt traps is described in Table 15-1, and the dimensions and settling area of each silt trap are given on the drawings referenced in the table (Annexure J). The inflows and settling areas are given in

Table 15-2.

Table 15-1: Design of Silt Traps

DESCRIPTION	DESIGN
Location	<p>RWD silt trap: receives the TSF solution trenches, DW CH 01A and DW CH 01B, and overflows into DW CH 01-1 and DW CH 01-2.</p> <p>WRD PCD silt trap: receives the WRD dirty water channels.</p> <p>RoM PCD silt trap: receives the RoM pad dirty water channels.</p> <p>Plant PCD silt trap: receives the plant area dirty water channels.</p>
Configuration	Reinforced concrete settling basin with an access ramp for mechanical cleaning and an overflow weir discharging to the receiving dam.
Foundation	<p>The sandy transported soil on site is collapsible, and the silt trap base and side slopes are not founded on in-situ transported soil.</p> <p>All transported soil is excavated below the silt trap and for at least 1.0 m beyond the footprint, down to medium dense to dense residual soil or bedrock, and stockpiled for re-use as backfill. A competent geotechnical engineer inspects the excavation and confirms that the founding layer is not susceptible to collapse before any backfill or layerworks are placed.</p> <p>Where the founding layer is soil, the exposed surface is wetted copiously and compacted to 93% of Mod AASHTO density at OMC 0% to +2%.</p> <p>Backfill to the underside of the layerworks with the stockpiled transported or residual soil in layers not exceeding 250 mm, compacted to 93% of Mod AASHTO density at OMC 0% to +2%, free of topsoil, organic matter and excavated rock.</p> <p>The silt traps are designed to tolerate a total settlement of up to 15 mm and a differential settlement of up to 1:500.</p>
Construction	<p>Layerworks from the bottom up: the top 150 mm of the backfill or founding soil ripped and recompact to 93% of Mod AASHTO density at OMC 0% to +2%; 150 mm of imported G6 material compacted to 95% of Mod AASHTO density at OMC 0% to +2%; a 50 mm blinding layer of C15/19 concrete; and a 250-micron polyethylene sheet bond breaker.</p> <p>250 mm thick reinforced concrete, water retaining, to the concrete specification in Table 19-1. Minimum cover to the reinforcement of 50 mm on faces cast against the blinding and 40 mm on all other faces.</p> <p>Construction joints with a 250 mm wide PVC external waterstop, dowelled construction joints and saw cut expansion joints, with joint fillers and sealants as shown on the concrete joint details.</p>
Sub-surface drainage	Ø110 mm corrugated slotted HDPE sub-surface drainage pipes, class SN16 (ring stiffness 16 kN/m ² to ISO 9969) or similar approved, beneath the silt trap footprint, joined to a Ø160 mm HDPE PE100 PN12.5 solid collector reporting to a sump manhole (19-918-SWM-111, SWM-211, SWM-311, SWM-502 and SWM-906).
Design guidelines	<p>Settling area sized by Stokes' law to retain particles of 0.2 mm and larger at the 1:10-year peak inflow.</p> <p>Settling depth 0.7 m to 1.0 m below the weir crest, sediment specific gravity 2.2 and a correction factor of 1.3 on the settling area.</p> <p>Overflow weir sized to pass the 1:50-year peak inflow into the receiving dam.</p>
Drawing reference	19-918-SWM-115, 19-918-SWM-213, 19-918-SWM-313 and 19-918-SWM-503.

Table 15-2: Silt Trap Inflows and Sizing

SILT TRAP	INFLOW Q 1:10 (m³/s)	INFLOW Q 1:50 (m³/s)	REQUIRED SETTLING AREA (m²)	PROVIDED SETTLING AREA (m²)	WEIR LENGTH (m)	HEAD OVER WEIR AT Q 1:50 (m)	HEIGHT FROM WEIR CREST TO TOP OF TRAP (m)
RWD	6.19	9.01	308	800	8.4	0.70	1.00
WRD PCD	3.01	4.69	150	750	16.4	0.29	0.87
RoM PCD	2.73	4.01	136	400	7.7	0.43	1.14
Plant PCD	3.73	5.20	186	400	17.3	0.30	1.20

Inflows are the sum of the catchment peaks upstream (section 7.2). At the RWD silt trap the inflow is the peak of the TSF side slope catchments D1a and D1b taken as whole catchments (Annexure D), because the peaks of their sub-catchments do not coincide at the trap, plus the combined penstock discharge (Table 11-4). Provided settling areas are the water surface at weir level, measured from drawings 19-918-SWM-115, SWM-213, SWM-313 and SWM-503 and rounded down.

15.3 Cleaning and Maintenance

The silt traps are to be inspected after every significant rainfall event and cleaned before the accumulated sediment encroaches on the settling depth. Sediment is removed with a TLB using the access ramp, and the silt is loaded, hauled and disposed of on the TSF.

16 DIRTY WATER COLLECTION SYSTEM

16.1 Overview

The dirty water areas served by the collection system are the TSF, the WRD, the RoM pad and the plant area. Runoff from these areas is intercepted by concrete lined channels and conveyed through silt traps into the RWD and the PCDs, as shown in Figure 7-3. Runoff from the North Pit and the South Pit is contained within the pits.

The dirty water collection system is designed on the following basis:

- the channels comply with regulation 6 of GN 704 and convey the 1:50-year flood peak of the contributing catchments without overtopping;
- the TSF solution trenches, DW CH 01A and DW CH 01B, form part of the TSF design (section 11.10) and discharge through a silt trap into DW CH 01-1 and DW CH 01-2, which carry the flow to the RWD;

for the remaining channels, the design flows are the 1:50-year flood peaks of the dirty water catchments given in

- Table 7-2; and
- each channel system enters its receiving dam through a silt trap.

16.2 Channel Design and Linings

The design of the dirty water channels is described in Table 16-1. The channels are sized in iDAS with Manning's equation for uniform flow. Each reach is sized for the sum of the 1:50-year peaks of the catchments upstream of it (



Table 7-2), on the surface gradient. The design flow, flow depth, freeboard, velocity, maximum capacity, slope and length of each reach are given on the iDAS long sections in Annexure J.

Table 16-1: Design of Dirty Water Channels

DESCRIPTION	DESIGN
Channel classification	Dirty water channels.
Channel configuration	Trapezoidal or V-drain with 1V:2H side slopes, or rectangular with vertical walls, with the sections given on the channel long sections.
Channel lining	<p>Excavate and shape to line and level and compact the in-situ material of the side slopes and bottom in a 150 mm layer to 93% of Mod AASHTO density.</p> <p>300 mm thick layer of imported G7 material compacted to 95% of Mod AASHTO density at OMC 0% to +2%, in layers not exceeding 150 mm.</p> <p>100 mm thick reinforced concrete C35/19 to BS EN 1992-1-1:2023, with mesh ref 245 in the centre and a minimum cover over and below the mesh of 30 mm. Minimum lap length of the mesh 400 mm.</p> <p>Construction joints at 2 m intervals, with the mesh discontinued across the joints, as per the construction joint detail.</p> <p>Cut-off berms, where shown on the channel layouts, of selected excavated material in layers not exceeding 150 mm, compacted to 93% of Mod AASHTO density at OMC 0% to +2%.</p> <p>The same lining applies to all dirty water channels, including the TSF solution trenches.</p>
Manning roughness	0.015 s/m ^{1/3} (concrete lined).
Channel outlet	Discharge into the silt trap upstream of the RWD or PCD.
Design guidelines	<p>Channels are designed to convey the 1:50-year flood peak without overtopping (GN 704).</p> <p>TSF solution trenches DW CH 01A and DW CH 01B are sized for the 1:50-year flood peak from the TSF side slopes plus the penstock and drain outflow. The penstock is sized for the 1:100-year, 72-hour TSF drawdown. DW CH 01-1 and DW CH 01-2, downstream of the silt trap, are sized for the combined flow of DW CH 01A and DW CH 01B.</p>
Drawing reference	19-918-SWM-469 and 19-918-SWM-902.

16.3 Culvert Crossings

Where access roads cross the dirty water channels, the flow is carried under the road in culverts sized for the 1:50-year channel design flow. They are checked in iDAS, and the design flow, headwater level against the road level, inlet or outlet control and outlet velocity of each culvert are given on the long sections in Annexure J. The design of the dirty water culvert crossings is described in Table 16-2.

Table 16-2: Design of Dirty Water Culvert Crossings

DESCRIPTION	DESIGN
Culvert crossings	Precast concrete pipe culverts at DW CH 04-2, DW CH 05 and DW CH 06. The number of barrels and the pipe diameters are given on the channel layouts and long sections.

DESCRIPTION	DESIGN
Construction	<p>Precast interlocking joint concrete pipe culverts, class 100D, to SANS 677, with precast headwall and wing wall units to suit the pipe diameter. Pipe diameters are given on the channel layouts and long sections.</p> <p>Foundation excavated to a competent founding layer, wetted copiously, and the top 150 mm ripped and recompact to 93% of Mod AASHTO density at OMC 0% to +2%.</p> <p>300 mm of natural gravel (G6) compacted to 95% of Mod AASHTO density at OMC 0% to +2%.</p> <p>Pipes encased in G6 material stabilised with cement to a minimum strength of 5 MPa, compacted in layers not exceeding 200 mm to 95% of Mod AASHTO density at OMC 0% to +2%, up to 300 mm above the pipe soffit.</p> <p>Road layerworks giving a minimum cover of 600 mm above the pipes, finished with a 300 mm natural gravel (G6) wearing course with a CBR of at least 80%, compacted to 98% of Mod AASHTO density at OMC 0% to +2% in layers not exceeding 150 mm.</p> <p>Inlet and outlet concrete lined as for the dirty water channels (19-918-SWM-902).</p> <p>The culvert class is confirmed for the cover and the mine vehicle loading in the structural design, which is completed as part of the pre-construction design set.</p>
Design guidelines	Culverts are sized to pass the 1:50-year channel design flow.
Drawing reference	19-918-SWM-905.

17 CLEAN WATER DIVERSION SYSTEM

17.1 Overview

In terms of GN 704 the clean water runoff arising within the catchment of the Volspruit Mine is required to be diverted and returned to the natural watercourse. The clean water channels intercept runoff upstream of the mine infrastructure and divert it around the dirty water areas to the environment, as shown in Figure 7-2.

The clean water diversion system is designed on the following basis:

- the channels comply with regulation 6 of GN 704 and convey the 1:50-year flood peak of the contributing catchments without overtopping;
- the design flows are the 1:50-year flood peaks of the clean water catchments given in Table 7-1; and
- each channel returns to the natural watercourse through an energy dissipating outlet structure.

17.2 Channel Design

The design of the clean water channels is described in Table 17-1. The channels are sized in iDAS with Manning's equation for uniform flow. Each reach is sized for the sum of the 1:50-year peaks of the catchments upstream of it (Table 7-1), on the surface gradient. The design flow, flow depth, freeboard, velocity, maximum capacity, slope and length of each reach are given on the iDAS long sections in Annexure J. The velocities in the earth lined channels are kept below the permissible velocity of 0.8 m/s for a grass cover on the site soils (SANRAL, 2013, Table 5.6).

Table 17-1: Design of Clean Water Channels

DESCRIPTION	DESIGN
Channel classification	Clean water channels.
Channel configuration	Trapezoidal or V-drain with 1V:3H side slopes, with the sections given on the channel long sections.
Channel lining	Excavate and shape the channel and berm foundation to line and level. Wet the exposed in-situ material copiously, then rip and recompact the top 150 mm to 93% of Mod AASHTO density at OMC 0% to +2%. Cut-off berms, where shown on the channel layouts, of selected excavated material in layers not exceeding 150 mm, compacted to 93% of Mod AASHTO density at OMC 0% to +2%. Grass to be established in the channel in accordance with the mine requirements.
Manning roughness	0.035 s/m ^{1/3} (earth lined).
Channel outlet	To the environment through an energy dissipating outlet structure, as described in Table 17-2.
Design guidelines	Channels are designed to convey the 1:50-year flood peak without overtopping.

DESCRIPTION	DESIGN
Drawing reference	19-918-SWM-600 to 19-918-SWM-633 and 19-918-SWM-901.

17.3 Erosion Protection

The clean water channels are earth lined and grassed. Grass is to be established in accordance with the mine requirements before the channels carry flow. Where the velocity on a reach exceeds the permissible velocity for bare soil (SANRAL, 2013, Figure 5.15), the reach is covered with an erosion control blanket until the grass is established. The outlets are protected by the energy dissipating structures described in Table 17-2.

17.4 Crossings and Discharge Structures

Each clean water channel returns to the natural watercourse through an energy dissipating outlet structure. The outlet design is described in Table 17-2.

Runoff from catchment C4 is not taken to a channel. It is held by a berm along the C4 and C6 boundary and released to the environment at the lowest point of the berm, through the energy dissipating outlet structure in Table 17-2, with a minimum width of 6 m.

Table 17-2: Design of Clean Water Channel Outlets

DESCRIPTION	DESIGN
Outlet structure	<p>Energy dissipating outlet structure at the end of each channel.</p> <p>Foundation excavated and shaped to line and level, wetted copiously, and the top 150 mm ripped and recompact to 93% of Mod AASHTO density at OMC 0% to +2%.</p> <p>150 mm thick reinforced concrete slab, C35/19, with mesh ref 395 in the bottom.</p> <p>300 mm thick dump rock with a D50 of 150 mm downstream of the slab, on a 210 g/m² geotextile (Table 10-4).</p> <p>200 × 200 mm concrete edge beam, C35/19, across the full width at the downstream end of the dump rock.</p> <p>The dump rock is sized for the outlet velocity of the channel given on the long section.</p>
Discharge requirement	The channel enters the natural system at the same elevation as the receiving watercourse, through the energy dissipating structure, so that erosion and incision of the watercourse are prevented.
Maintenance	Inspected and cleared at regular intervals and after every significant rainfall event.
Drawing reference	19-918-SWM-903.

Where access roads cross the clean water channels, the flow is carried under the road in culverts sized for the 1:50-year channel design flow. They are checked in iDAS, and the design flow, headwater level against the road level, inlet or outlet control and outlet velocity of each culvert are given on the long sections in Annexure J. The crossing design is described in Table 17-3.

Table 17-3: Design of Culvert Crossings

DESCRIPTION	DESIGN
Culvert crossings	Precast concrete pipe culverts at CUL CW 02A, CUL CW 04, CUL CW 04-1, CUL CW 04-1-1, CUL CW 04-2 and CUL CW 05-2. The number of barrels and the pipe diameters are given on the channel layouts and long sections.
Construction	As for the dirty water culvert crossings (Table 16-2), except that the inlet and outlet are protected with 300 mm thick Reno mattresses and 500 × 500 mm gabion boxes of Zn/Al 5% coated wire, on a 210 g/m ² geotextile (Table 10-4).
Design guidelines	Culverts are sized to pass the 1:50-year channel design flow.
Drawing reference	19-918-SWM-905.

18 FLOOD DIVERSION BERM

18.1 Overview

The flood line study (Annexure E) shows that the 1:50 and 1:100-year flood lines of the Nyl River extend into the North Pit area. The flood diversion berm is constructed along the eastern bank of the river so that the 1:100-year flood cannot enter the North Pit. The berm does not store water. It carries load only while the river is in flood, and the water level against it falls with the river, so it is not designed as a dam.

The geohydrological study (Annexure F) regards the berm as essential to ensure that no flood can overtop to the east of the Nyl River in the North Pit area and recommends that it be as impermeable to water as possible. The berm is therefore built with a low permeability core, keyed into the clay layer that underlies the floodplain.

18.2 Geometry and Freeboard

The berm is designed to the 1:100-year flood level given in Chapter 7, with 1.0 m freeboard to the non-overspill crest. It is built as two embankment types, a homogeneous earth fill embankment (Type 1) at the ends of the alignment where the berm is low, and a clay core rock fill embankment (Type 2) over the central, higher section. The design is described in Table 18-1 and the typical sections are shown in Figure 18-1.

Table 18-1: Design of the Flood Diversion Berm

DESCRIPTION	DESIGN
Alignment	Along the eastern bank of the Nyl River, between the river and the North Pit, starting and ending at natural ground above the 1:100-year flood level.
Non-overspill crest	1 064.13 mamsl, giving 1.0 m freeboard above the 1:100-year flood level of 1 063.13 mamsl.
Geometry	Minimum crest width of 3.5 m, with a 150 mm gravel capping layer. Maximum height of approximately 7 m.
Type 1 embankment	Homogeneous earth fill embankment with a semi-pervious core. Rip-rap slope protection on the river side, D50 = 300 mm. Chainage 0 m to 427 m and 1 100 m to the end of the berm, with 40 m transition zones to Type 2. 5 m construction buffer.
Type 2 embankment	Clay core rock fill embankment. Two filter zones on each side of the core: filter zone 1 of gravel, D50 = 40 mm, 300 mm thick, and filter zone 2 of river sand, 300 mm thick. Compacted rock fill shells, D50 = 300 mm. Chainage 467 m to 1 060 m. 7 m construction buffer.
Foundation	Box cut to an assumed competent founding layer, with a clay key into the low permeability clay layer of the floodplain, tied into jet grout columns that cut off seepage beneath the berm (19-918-SWM-701).

DESCRIPTION	DESIGN
Materials	Clay core and homogeneous fill selected from the transported and residual soils stripped from the North Pit footprint (section 19.1). Rock fill and filter material selected from the North Pit rock, subject to testing before construction.
Clean water crossing	750 mm diameter class 50D concrete pipe culvert through the berm, fitted with an anti-flood non-return flap valve on the river side so that the river cannot back up through the culvert.
Drawing reference	19-918-SWM-700 to 19-918-SWM-703.

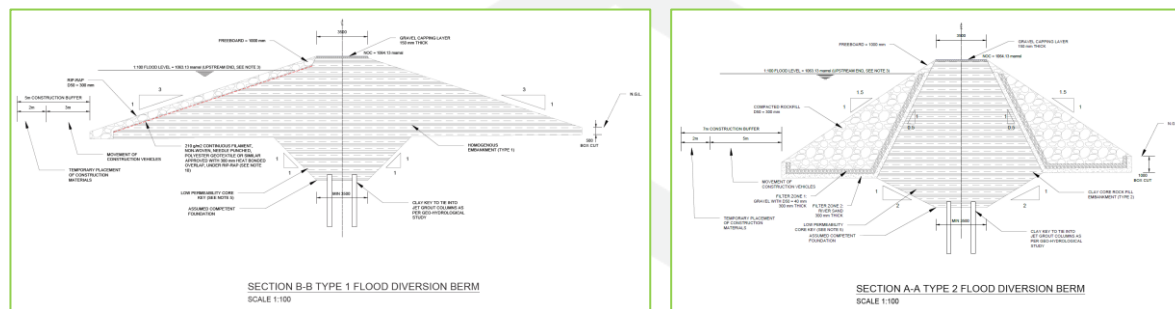


Figure 18-1: Flood Diversion Berm Typical Sections

18.3 Erosion Protection

The Type 1 embankment is protected on the river side by riprap with a D50 of 300 mm, and the Type 2 embankment by its compacted rock fill shell of the same grading. The crest is capped with gravel to limit erosion from rainfall and vehicle traffic. The berm is to be inspected after every flood that reaches it, and displaced riprap replaced.

18.4 Stability

The slope stability of the flood diversion berm was assessed in the same report as the dams (Annexure G), with the method, foundation parameters and seismic coefficient described in section 14.8, and the loading conditions in Table 9-4. The berm was modelled as compacted rock fill with a D50 of 300 mm and a clay core, with the parameters in Table 14-6 taken from geotechnical literature (Look, 2014). The section analysed and the results are given in the slope stability report in Annexure G. Two further conditions were analysed for the berm: flood loading, with the 1:100-year flood level 1 m below the crest, and rapid drawdown of the river side slope from that level (Table 9-4). The factors of safety meet the minimum values for all the loading conditions.

PART D — IMPLEMENTATION AND ASSURANCE



19 CONSTRUCTION MATERIALS AND SPECIFICATIONS

19.1 Material Sources and Availability

The construction materials come from three sources: material won on site from the facility excavations and the North Pit, natural and commercial material imported to site, and manufactured products supplied to specification. The main materials and their intended sources are:

- waste rock from the North Pit, of the size and grading specified, for the body and toe zone of the TSF starter walls;
- locally sourced material for the upstream zones of the starter walls and for the toe and heel walls;
- selected material from the basin excavations for the homogeneous earth fill embankments of the HDPE lined dams;
- clay core and homogeneous fill selected from the transported and residual soils stripped from the North Pit footprint, and rock fill and filter material selected from the North Pit rock, for the flood diversion berm;
- imported fine selected earth fill for the receiving faces beneath the geomembranes;
- tailings reclaimed from the operating TSF cells, as an alternative receiving face in TSF cells 2 and 3 where they meet the acceptance criteria in section 11.5;
- imported natural gravel of G6 and G7 quality, and C4 stabilised gravel, for the layerworks beneath the silt traps, the Plant PCD, the RoM pad, the sump manholes and the culverts;
- commercial aggregate, filter sand and washed stone for the drains, the concrete and the soilcrete; and
- manufactured products, including the geomembranes, geotextiles, geogrid, geocells, HDPE pipes, precast concrete culverts, manhole sections and penstock rings, supplied to the specifications in Chapter 10 and on the drawings.

Material won on site is used only once its grading, plasticity and compaction characteristics, and where relevant its permeability and shear strength, have been confirmed by testing. This includes the laboratory testing of the dam embankment fill required in section 14.3. Material that does not meet the specification is processed, blended or replaced with imported material approved by the Engineer. Topsoil stripped from the footprints is stockpiled separately and kept for rehabilitation.

The transported and residual soils meet the permeability required of a low permeability core when compacted to 95% of Standard Proctor density (section 4.6). Because their grading and plasticity are variable, only material that meets the grading, plasticity and permeability specified for the core is placed in the berm core and the homogeneous fill. The quantity available from the North Pit footprint is confirmed by the material sourcing investigation recommended in section 4.6 before construction. The Nyl River floodplain and the wetland along it are excluded as borrow sources, and no construction material is taken from them.

19.2 Earthworks and Layerworks

The earthworks and layerworks for each facility are specified on its drawings and summarised in the facility chapters. Earthworks are carried out to SANS 1200 D, DM and ME. The compaction targets refer to the Mod AASHTO maximum dry density (SANS 3001-GR30) or the Standard Proctor maximum dry density (ASTM D698), as stated on each drawing, and the in-place density is tested by nuclear gauge (SANS 3001-NG5).

The Engineer confirms the excavation levels on site. Excavation taken below acceptable founding material is backfilled with mass concrete or soilcrete, and no foundation is placed on backfilled soil unless the drawings allow it.

19.3 Geosynthetics

The geomembrane and geotextile specifications are given in Table 10-3 and Table 10-4, the dam cover and geogrid in Table 14-2, and the leak location survey in section 10.7. Geosynthetics are delivered with the manufacturer's quality control certificate for each roll, and are stored off the ground, covered and clear of traffic until they are placed.

19.4 Concrete and Pipework

Concrete works are carried out to SANS 2001-CC1, with the grades and exposure requirements in Table 19-1 and on drawing 19-918-SWM-900. Water retaining structures comply with BS EN 1992-1-1:2023 and BS EN 1992-3, tightness class 1. Loose material is removed from each excavation before concrete is cast, and concrete is placed in lifts of no more than 1 m unless the Engineer authorises otherwise. Structural steelwork is to SANS 2001-CS1, with hot-dip galvanising to SANS 121 (ISO 1461) where specified.

Table 19-1: Concrete Specification

ITEM	SPECIFICATION
Water retaining structures	BS EN 1992-1-1:2023 and BS EN 1992-3 (tightness class 1) for the silt traps, the Plant PCD, the RoM pad slab, the emergency spillways, the connection channel and the TSF sub-surface drainage outlet sump.
Grades	Unless the drawings state otherwise: blinding C15/19; mass concrete, mass concrete backfill and pipe encasements C20/19, except the TSF penstock pipe encasement at C30/19; reinforced concrete, including foundations, slabs, walls, channels and all water retaining structures, C35/19.
Concrete below natural ground level	Designed for the severe exposure condition: C35/19 with CEM I or CEM II 42.5N cement to SANS 50197-1, a maximum water:cement ratio of 0.5 and a minimum cement content of 325 kg/m ³ . The mix design is submitted for acceptance.
Workmanship	Concrete works, curing and protection to SANS 2001-CC1.
Reinforcement	As given on the drawings (19-918-SWM-900).

The HDPE pipe materials and jointing are given in Table 10-3, and the pipe diameters and pressure classes on the drawings.

20 CONSTRUCTION QUALITY ASSURANCE

Construction quality assurance is required by regulation 3(2)(g) of GN R636. The construction quality assurance (CQA) plan for the barrier systems is attached as Annexure I. The plan is site specific and applies only to this design report and its drawings. It adopts the example CQA plan of the Department of Water and Sanitation (November 2019) as its point of departure. Where the design report or the drawings set a standard or a performance requirement, that requirement takes precedence over the example plan.

The plan is a living document. Before the works go to tender, it is developed to include the hold points, test frequencies and acceptance criteria for each facility, without reducing any requirement of this report or the drawings.

Quality control and quality assurance are kept separate. The manufacturers control the quality of the geosynthetics, and the contractor and the geomembrane installer control the quality of the works they build. The design engineer is appointed for full-time construction monitoring and approves each stage of the works before it is covered. An independent CQA monitor audits the quality control records and the works, reports to the Employer with copies to the Engineer, and confirms on completion whether the facilities were built in accordance with the design report and drawings.

The materials and workmanship are tested against the specifications in this report and on the drawings. The barrier performance requirements in sections 11.7 and 14.4 are confirmed on the materials used in the works before deposition starts on the TSF and before the dams are filled. No facility is commissioned until the leak location survey (section 10.7) is complete and all defects found have been repaired.

On completion of each facility, the Engineer compiles a completion report with the as-built drawings and the test records, and the independent CQA monitor issues a separate report. Both reports are kept on the mine's records and submitted to the Department as the licence conditions require.

21 OPERATION, MAINTENANCE AND MONITORING REQUIREMENTS

The facilities perform as designed only if they are operated within the limits set in this report and are maintained and monitored over their life. This chapter sets out the minimum requirements. The mine is to develop them into an operation, maintenance and surveillance manual for the facilities before the TSF is commissioned.

21.1 Responsibilities

The mine is responsible for operating, maintaining and monitoring the facilities, and appoints a responsible person for the TSF and the dams, supported by a tailings engineer. The TSF, and any dam registered with the Dam Safety Office, is inspected by the approved professional person at the intervals required by GN R139.

21.2 Tailings Storage Facility

The TSF is operated within the following limits:

- the pool is kept between the minimum and maximum operating ponds in Table 9-6, and is held against the penstock intakes, away from the outer walls;
- the freeboard in Table 9-2 is maintained at all times, including during commissioning, and after a storm the pool is drawn down through the penstock within 72 hours;
- the rate of rise does not exceed the maximum in Table 9-6;
- the coarse underflow is placed over the drainage corridor, as set out in section 11.15;
- penstock rings are added ahead of the rising tailings, redundant temporary intakes are sealed as the pool migrates, and no ring stack is raised beyond the safe height in section 11.11; and
- no external water is directed onto the TSF (section 9.5).

The instrumentation installed under section 11.16 is read at the frequencies set in the surveillance manual, and the readings are compared with trigger levels. The tailings engineer sets the trigger levels once baseline readings are available, with an alert level that calls for closer surveillance and an action level that calls for the Engineer to assess the facility and for remedial measures. Surveillance of the TSF comprises:

- the pool position, the freeboard and the beach length, recorded daily by the operator;
- the phreatic surface and pore pressures in the walls and the foundation, from the piezometers;
- the horizontal and vertical movement of the confinement walls, from the survey beacons;
- the flow and water quality at the drainage outfalls and the sub-surface drainage outlet sump, so that a blocked drain or a leak in the barrier is picked up early;
- visual inspection of the walls, the outer slopes, the penstock, the catwalk and the solution trench for cracking, seepage, erosion and damage, at least weekly and after every significant storm; and

- an annual topographical survey, used to check the rate of rise, the stored volume and the remaining capacity against the stage capacity curve (Figure 11-2).

21.3 Dams

The dams are operated and monitored as set out in section 14.9. In addition:

- the storm storage above the maximum operating level is restored after every storm, in line with the operating rules of the water balance (Chapter 6);
- the outflow recorded at each sub-surface drainage sump manhole is compared with the action leakage rate for the dam (section 10.9), or, for the Plant PCD, with the triggers in section 14.5, and an exceedance triggers an inspection of the barrier and, where needed, a leak location survey and repair;
- damage to the geocell cover, the ballast or the concrete lining is repaired so that the geomembrane is not left exposed; and
- trees and shrubs are kept off the embankments and the spillways, and burrowing animals are controlled.

21.4 Stormwater Infrastructure

The stormwater infrastructure is inspected before each wet season, after every significant rainfall event and at least monthly during the wet season. Inspection and maintenance cover:

- the dirty water channels and solution trenches, for silt, debris, cracked concrete and failed joints, with silt removed and damaged lining repaired so that the channels keep their design capacity;
- the clean water channels, for erosion, loss of grass cover and blockage, with eroded sections reinstated and regrassed;
- the culverts and their inlets and outlets, for blockage and scour;
- the silt traps, which are cleaned before the sediment encroaches on the settling depth and in any case before each wet season, with the silt disposed of on the TSF (section 15.3);
- the energy dissipating outlets, Reno mattresses and gabions, for scour and displaced stone; and
- the flood diversion berm, after every flood that reaches it, for erosion, displaced riprap, settlement and seepage on the pit side, and to confirm that the flap valve on the culvert through the berm closes.

The flood diversion berm keeps the Nyl River out of the North Pit up to the 1:100-year flood level (Chapter 18). A larger flood can overtop the berm without the berm itself failing. The operation, maintenance and surveillance manual and the mine's emergency procedures are therefore to include monitoring of the river level at the berm, with trigger levels for evacuating the North Pit before the river reaches the berm crest.

21.5 Waste Rock Dump and Run-of-Mine Pad

On the WRD, the bench paddocks, the perimeter channels and the silt trap are inspected after every significant storm. Damaged paddock berms are reinstated, silt is removed so that the channels and the silt trap keep their design capacity, and the outflow and water quality at the manhole sumps of the sub-surface drainage network and the cut-off drain are recorded. On the RoM pad, the slab, the joint sealant and the sub-surface drainage sump are inspected as set out in section 13.5.

21.6 Water Quality and Groundwater Monitoring

Water quality is monitored in the dams, at the TSF drainage outfalls, at the sub-surface drainage sumps and in the groundwater monitoring boreholes around the facilities. The results are compared with the baseline and with the leachable concentrations on which the waste classification rests, so that a failure of a barrier is detected. The monitoring points, parameters and frequencies are set in the environmental management programme, and the water use licence conditions.

21.7 Reporting and Review

The monitoring results are reported monthly to the responsible person and reviewed at least once a year by the tailings engineer, who compares the performance of the facilities with the design and recommends changes to the operation where needed. The exceedance of an action level, a spill from a dirty water system or a leak detected in a barrier is reported at once to the mine and the Engineer, and to the Department where the licence conditions require it. The stage capacity curve, the water balance and the surveillance manual are updated whenever the mine plan or the production rate changes.

22 DAM BREAK ANALYSIS

A tailings dam breach analysis (TDBA) was carried out for the Volspruit TSF to estimate the consequences of a flow failure of the facility (Eco Elementum, 2026). The results confirm the zone of influence and the safety classification (Chapter 8) and are the basis for the Emergency Response and Preparedness Plan (ERPP). The TDBA report is attached as Annexure K, and the inundation, velocity and arrival time maps are given on drawings 19-918-DDR-DBA-801 to 19-918-DDR-DBA-818.

22.1 Basis of the Analysis

The analysis was carried out at the final height of the TSF, with the three compartments treated as one facility. Breaches were placed on the northern, eastern, southern and western walls, which together cover every area that could be inundated. Two scenarios were simulated at each location, giving eight simulations:

- a sunny-day failure during normal operation, caused by internal erosion, piping, an earthquake or poor operation; and
- a rainy-day failure, in which the 1:100-year 24-hour storm adds to the pool before the wall is breached.

Overtopping was simulated as the breach mechanism, as a piping failure eventually develops into an overtopping failure. The rainy-day scenario follows Case 1A of Small et al. (2017), in which the tailings liquefy, and the breach intercepts the pool, and the sunny-day scenario follows Case 2A.

The volume of liquefied tailings was estimated from a valley-shaped failure surface that runs into the facility from the breach at 2° with side slopes of 4°, adopted from the range of 2° to 6° observed in failed facilities by Blight and Fourie (2003). Rust and Dressler (2016) recommend a slope of 1° into the facility. Where the slopes were too shallow for the breach to stay within the TSF, they were increased. The pool was taken as 10% of the upper surface area at a depth of 0.3 m, and the 24-hour storm adds 91 924 m³ to the pool in the rainy-day scenario. The breach outflow volumes range from 881 486 m³ for the eastern sunny-day breach to 1 966 853 m³ for the northern rainy-day breach, which is 4% to 7% of the impounded tailings (Table 22-1). A breach formation time of 16 minutes was adopted, calculated by the method of Froehlich (2008).

No rheological testing has been done on the Volspruit tailings. The rheological parameters were taken from a study on similar tailings, at solids concentrations by mass of 65% for the rainy day and 73% for the sunny day. The simulations were run in HEC-RAS on a terrain model built from the site survey, the design models of the TSF, the RWD, the WRD and the North and South Pits, and 1 m contour data from the National Geo-spatial Information.

Table 22-1: Tailings Breach Outflow Volumes

BREACH	SUNNY-DAY OUTFLOW (m³)	RAINY-DAY OUTFLOW (m³)	IMPOUNDED TAILINGS RELEASED (%)
Northern	1 874 930	1 966 853	7
Eastern	881 486	973 409	4
Southern	1 854 376	1 946 299	7
Western	1 354 897	1 446 820	5

22.2 Analysis Limitations

The simulation software cannot model fluids with different rheological properties in one simulation, so flow in the Nyl River was not modelled. Once the tailings enter the river they mix with the runoff, and the inundation extent, velocities and arrival times in the river will differ from the results presented. The results remain valid for the area around the TSF, and the sunny-day outflows, which do not reach the river, are not affected by the river flow.

22.3 Inundation Zones

The rainy-day breaches inundate a much larger area than the sunny-day breaches, as they release more material and the material is far less viscous. A rainy-day outflow runs into the RWD, the North Pit and the South Pit before it reaches the Nyl River. The sunny-day outflows stay in the area directly downstream of the TSF, including the RWD, and do not reach the pits. The southern sunny-day outflow comes to rest about 1.9 km from the breach. The combined inundation zone of all eight simulations is shown on drawing 19-918-DDR-DBA-801. It extends along the Nyl River to the N1 at the river bridge.

22.4 Flow Velocities and Arrival Times

The highest velocities occur on the TSF walls at the breach, where they exceed 10 m/s, and where the tailings flow over the sidewalls into the pits. Velocities fall with distance from the breach. In a western rainy-day breach, the tailings reach the RWD in about 6 minutes, the North and South Pits in about 16 minutes, and the Nyl River, about 3.5 km away, in about 46 minutes. Personnel in the pits would therefore have about 16 minutes to evacuate after a rainy-day breach.

22.5 Sensitivity to the Outflow Volume

The eastern rainy-day breach was re-run at 0.75 and 1.25 times the base outflow volume. At 0.75 times the base volume the tailings no longer reach the Nyl River to the north-west, and the southern and western limits of the zone pull back by several hundred metres. At 1.25 times the base volume the zone closely follows the base case. The extent of inundation is therefore more sensitive to a reduction in the outflow volume than to an increase.

22.6 Implications for the Design and Operation

The combined inundation zone on drawing 19-918-DDR-DBA-801 confirms the zone of influence of the TSF and the safety classification in Chapter 8. The ERPP is to be developed from it, with particular

attention to personnel in the North and South Pits and at the RWD, and to warning road users on the N1 at the Nyl River bridge. As the river flow was not modelled, the ERPP is to treat the inundation extent and arrival times along the Nyl River as indicative.



23 CONCLUSION

This report presents the detailed design of the mine residue facilities and the water management infrastructure for the Volspruit Mine, consolidated into one report in support of the water use licence and waste management licence applications.

The TSF is lined with a Class C equivalent inverted barrier. The RWD, the irrigation dam and the WRD and RoM PCDs are lined with a 2 mm HDPE geomembrane under a soilcrete cover, and the Plant PCD, the silt traps and the RoM pad with water retaining reinforced concrete. The WRD is founded on the Class D foundation. Each barrier has a sub-surface drainage system in its foundation, as required by the 2024 amendment of GN R636, and its outflow is monitored. Clean and dirty water are kept separate in accordance with GN 704, and the flood diversion berm keeps the 1:100-year flood of the Nyl River out of the North Pit.

The layout, geometry and specification of the facilities are complete, and the design addresses the barrier, water management and safety requirements of GN R636 as amended, GN 704 and SANS 10286 within the scope of this report. The remaining assumptions are confirmed by the investigations, tests and approvals in Table 23-1 before construction starts, and the drawings in Annexure J are updated to match this report before they are issued for construction.

The investigations, tests and approvals in Table 23-1 are completed before construction starts. The design is reviewed against their results, and any change is issued as a revision of this report and the drawings.

Table 23-1: Pre-construction Programme

NO.	ITEM	PURPOSE	SECTION	BY
1	Geophysical survey and confirmatory drilling of the TSF and WRD footprints	Faulted, fractured or deeply weathered zones	4.2.1, 4.2.2	Geotechnical engineer
2	Test pits at the RoM PCD and the western RoM area	Founding and excavation conditions	4.2.3	Geotechnical engineer
3	Detailed investigation of the collapsible soils beneath the TSF and WRD	Extent, depth and method of ground improvement, approved by the Engineer	11.5, 12.3	Geotechnical engineer
4	Material sourcing investigation, including the North Pit footprint soils	Source and quantity of the materials won on site, including the berm core and fill	4.6, 19.1	Geotechnical engineer
5	Laboratory testing of the dam embankment fill	Density, permeability and shear strength against the design values	14.3	Geotechnical engineer
6	Grading tests on the North Pit core, filter and rock fill	Filter compatibility of the berm zones	10.10, 18.2	Engineer
7	Grading tests on the founding soils, receiving face, filter sand and aggregate, and gradient ratio tests on the	Filter bands, receiving face limits and geotextile properties, before materials are ordered	10.10	Engineer

NO.	ITEM	PURPOSE	SECTION	BY
	transported soils			
8	Laboratory testing of representative tailings samples	Undrained strength ratios, permeabilities and grading, with the TSF stability reassessed on the measured values	10.10, 11.14	Geotechnical engineer
9	Re-run of the water balance on the updated production plan	Maximum operating levels and 1:50-year storm storage of each dam	Chapter 6, 14.1	Mine, with the water balance specialist
10	Confirmation of the WRD design capacity against the final mine plan	Capacity of the WRD	12.2	Mine, with the Engineer
11	Structural design of the RoM pad slab, the Plant PCD, the silt traps and the culverts	Loads, thickness and reinforcement, tightness class 1 crack control, joint layout and culvert class	13.3, 14.5, 15.1, 16.3	Structural engineer
12	Dam safety classification of the TSF and the dams, and the licence to construct	Registration with the Dam Safety Office	11.2, 14.2	Mine, with the Engineer
13	CQA plan developed for tender	Hold points, test frequencies and acceptance criteria	Chapter 20	CQA monitor, with the Engineer

24 REFERENCES

The documents cited in this report are listed below. The legislation, regulations, standards and guidelines applied in the design are listed in section 2.5.

African Environmental Development, 2010. Flood Line Report: 50- and 100-year flood lines for the Nyl River at the Sylvania Resources Volspruit Mine. Report AED0151, Revision 00.a, 22 July 2010. Prepared by G. Krige and verified by P. Bond.

Australian National Committee on Large Dams (ANCOLD), 2012. Guidelines on Tailings Dams: Planning, Design, Construction, Operation and Closure. May 2012, with Addendum of July 2019.

Australian National Committee on Large Dams (ANCOLD), 2012. Guidelines on the Consequence Categories for Dams. October 2012.

Benson, C.H., Zhai, H. and Wang, X., 1994. Estimating hydraulic conductivity of compacted clay liners. Journal of Geotechnical Engineering, ASCE, 120(2), pp. 366 to 387.

Blight, G.E. and Fourie, A.B., 2003. A review of catastrophic flow failures of deposits of mine waste and municipal refuse. International Workshop on Occurrence and Mechanisms of Flow in Natural Slopes and Earthfills.

Brink, A.B.A. and Bruin, R.M.H., 2002. Guidelines for Soil and Rock Logging in South Africa. 2nd impression. Proceedings of the Geoterminology Workshop organised by AEG, SAICE and SAIEG.

Canadian Dam Association (CDA), 2013. Dam Safety Guidelines 2007 (2013 edition). Canadian Dam Association.

Chou, Y.-C., Brachman, R.W.I. and Rowe, R.K., 2022. Leakage through a hole in a geomembrane beneath a fine-grained tailings. Canadian Geotechnical Journal, 59, pp. 372 to 383.

Committee of Land Transport Officials (COLTO), 1998. Standard Specifications for Road and Bridge Works for State Road Authorities. Pretoria.

Council for Geoscience, 1978. 2428 Nylstroom, 1:250 000 Geological Series. Pretoria.

Department of Water Affairs and Forestry, 2007. Best Practice Guideline A4: Pollution Control Dams. Pretoria.

Department of Water Affairs and Forestry, 2008. Best Practice Guideline A2: Water Management for Mine Residue Deposits. Pretoria.

Department of Water and Sanitation, 2016. Guideline for the Development and Implementation of Water Conservation and Water Demand Management Plans for the Mining Sector. Pretoria.

Eco Elementum (Pty) Ltd, 2022. Volspruit MRD Design Report. Reference 19-918-SW, Version 0.0, 2 December 2022. Superseded by this report.

Eco Elementum (Pty) Ltd, 2022. Volspruit Stormwater Management Plan. Reference 19-918-SW, Version 0.0, 2 December 2022. Superseded by this report.

Eco Elementum (Pty) Ltd, 2026. Sylvania Volspruit Waste Classification, 2026 Update. Reference 19-918, Version 1.0, 9 October 2026.

Eco Elementum (Pty) Ltd, 2026. Volspruit Mine: PCD, RWD and Diversion Berm Slope Stability Assessment. Report 19-918 DDR-RPT-001, Version 0.0, 9 October 2026.

Eco Elementum, 2026. Volspruit TSF Tailings Dam Breach Analysis Report. Report 19-918-WMF, Version 0.0, 30 September 2026.

EcoPartners, 2024. Environmental Impact Assessment Report (Part A) and Environmental Management Programme (Part B) for the Volspruit Mine. 29 September 2024.

Fan, J. and Rowe, R.K., 2022. Seepage through a circular geomembrane hole when covered by fine-grained tailings under filter incompatible conditions. Canadian Geotechnical Journal, 59, pp. 410 to 423.

Froehlich, D.C., 2008. Embankment dam breach parameters and their uncertainties. Journal of Hydraulic Engineering, 134(12), pp. 1708 to 1721.

GEM-GPT, 2026. Groundwater Specialist Study Update for Proposed Volspruit Mining Project. Report SYVOL-26-14406, Draft, June 2026.

Giroud, J.P., 1997. Equations for calculating the rate of liquid migration through composite liners due to geomembrane defects. Geosynthetics International, 4(3-4), pp. 335 to 348.

Giroud, J.P. and Bonaparte, R., 1989. Leakage through liners constructed with geomembranes, Part I: Geomembrane liners. Geotextiles and Geomembranes, 8(1), pp. 27 to 67.

Global Tailings Review, 2020. Global Industry Standard on Tailings Management. August 2020.

Hawley, M. and Cuning, J., 2017. Guidelines for Mine Waste Dump and Stockpile Design. CSIRO Publishing, Melbourne.

Holtz, R.D., Christopher, B.R. and Berg, R.R., 2008. Geosynthetic Design and Construction Guidelines. Report FHWA-NHI-07-092, Federal Highway Administration, Washington DC.

Howell, G.C. and Kirsten, A.H., 2016. Interface Shear: Towards Understanding the Significance in Geotechnical Structures. South Africa.

Hynes-Griffin, M.E. and Franklin, A.G., 1984. Rationalizing the Seismic Coefficient Method. Miscellaneous Paper GL-84-13, US Army Engineer Waterways Experiment Station, Vicksburg, Mississippi.

Jennings, J.E. and Knight, K., 1975. A guide to construction on or with materials exhibiting additional settlement due to "collapse" of grain structure. Proceedings of the 6th Regional Conference for Africa on Soil Mechanics and Foundation Engineering, Durban, September 1975, Volume 1, pp. 99 to 105.

Kijko, A., Graham, G., Bejaichund, M., Roblin, D. and Brandt, M.B.C., 2003. Probabilistic Peak Ground Acceleration and Spectral Seismic Hazard Maps for South Africa. Report 2003-0053, Council for Geoscience, Pretoria.

Knight Hall Hendry, 2012. Volspruit Mine Geotechnical Investigation: Feasibility Phase Report. Report KHH1931/3010035701, Revision 0, March 2012.

Koerner, G.R. and Narejo, D., 2005. Direct shear database of geosynthetic-to-geosynthetic and geosynthetic-to-soil interfaces. GRI Report 30, Geosynthetic Institute, Folsom, Pennsylvania.

Koerner, R.M., 2012. Designing with Geosynthetics. 6th edition. Xlibris Corporation.

Koerner, R.M. and Soong, T.-Y., 1998. Analysis and design of veneer cover soils. Proceedings of the Sixth International Conference on Geosynthetics, Atlanta, pp. 1 to 23.

Legge and Associates, 2024. Alternative Barrier System Layout to a Class C Single Composite Barrier: Inverted Barrier Having Superior Containment Performance and Cost Effectiveness. Prepared through Geotheta (Pty) Ltd for Harmony Gold. Undated.

Legge, K.R., 2024. The Inverted Barrier as Superior Alternative to a Class C Barrier for TSFs. Correspondence to the Department of Water and Sanitation, 15 July 2024.

Look, B.G., 2014. Handbook of Geotechnical Investigation and Design Tables. 2nd edition. Taylor & Francis, London.

Mills Water, 2026. Simulation of Water Quality from North Pit, Volspruit Mine. Updated Report, Final, 28 September 2026. Prepared by M. Mills.

Mnisi, K.K. and Bester, K.J.P., 2018. Achieving cost effective compliant pollution control performance standard through upturned barrier systems. Proceedings of the 6th International Mining and Industrial Waste Management Conference, Limpopo, 29 to 31 October 2018.

Morgenstern, N.R. and Price, V.E., 1965. The analysis of the stability of general slip surfaces. Géotechnique, 15(1), pp. 79 to 93.

Peregrine Geoconsultants, 2021. Geotechnical Investigation Report for the Volspruit Mine. Report PG19-003/673, Revision 02, 30 June 2021.

Qian, X., Koerner, R.M. and Gray, D.H., 2002. Geotechnical Aspects of Landfill Design and Construction. Prentice Hall, New Jersey.

Rowe, R.K., 2012. Short- and long-term leakage through composite liners. The 7th Arthur Casagrande Lecture. Canadian Geotechnical Journal, 49(2), pp. 141 to 169.

Rowe, R.K., Joshi, P., Brachman, R.W.I. and McLeod, H., 2017. Leakage through holes in geomembranes below saturated tailings. Journal of Geotechnical and Geoenvironmental Engineering, 143(2), 04016099.

Rust, M. and Dressler, S., 2016. Prediction of potential tailings storage facility inundation zones. ICOLD 2016 Annual Meeting, International Symposium on Appropriate Technology to Ensure Proper Development, Operation and Maintenance of Dams in Developing Countries.

Small, A., James, M. and Al-Mamun, M., 2017. Advancing the state of practice for tailings dam breach assessment using empirical correlations. Proceedings of the Canadian Dam Association Annual Conference, Kelowna, BC.

Smithers, J.C. and Schulze, R.E., 2003. Design Rainfall and Flood Estimation in South Africa. WRC Report K5/1060, Water Research Commission, Pretoria. Design Rainfall Estimation in South Africa software.

South African Institution of Civil Engineering (SAICE), 2010. Site Investigation Code of Practice. 1st edition. Geotechnical Division, SAICE.

South African National Roads Agency SOC Ltd (SANRAL), 2013. Drainage Manual. 6th edition. Pretoria.

Tognon, A.R., Rowe, R.K. and Moore, I.D., 2000. Geomembrane strain observed in large-scale testing of protection layers. Journal of Geotechnical and Geoenvironmental Engineering, 126(12), pp. 1194 to 1208.

United States Department of Agriculture (USDA), 2017. Chapter 26: Gradation Design of Sand and Gravel Filters. Part 633, National Engineering Handbook. Natural Resources Conservation Service, Washington DC.

Water Hunters, 2024. Volspruit Mining Company (Pty) Ltd Computerized Water Balance Simulation Model. Technical Report 1023 (SX-TR0124), Version 1.2 Final, May 2024.

Water Research Commission, 2011. Guidelines on Freeboard for Dams. Volumes I and II. WRC Reports 1759/1/11 and 1759/2/11, Pretoria.

Wissa, A.E. and Fuleihan, N.F., 1993. Design and reclamation of phosphogypsum disposal sites. AIChE Symposium Series, Vol. 89. American Institute of Chemical Engineers.

ANNEXURES



ANNEXURE A: MATERIALS WASTE CLASSIFICATION



ANNEXURE B: WATER BALANCE REPORT



ANNEXURE C: GEOTECHNICAL INVESTIGATION REPORT

To be provided by the geotechnical team.



ANNEXURE D: HYDROLOGY AND CALCULATIONS



ANNEXURE E: FLOOD LINE STUDY



ANNEXURE F: GEOHYDROLOGICAL STUDY

To be provided by the environmental specialist.



ANNEXURE G: SEEPAGE AND SLOPE STABILITY ANALYSIS RESULTS

[Placeholder: insert the seepage and slope stability analysis results.]



ANNEXURE H: ADDITIONAL INFORMATION FOR INVERTED BARRIER SYSTEMS



ANNEXURE I: CONSTRUCTION QUALITY ASSURANCE PLAN



ANNEXURE J: DETAIL DESIGN DRAWINGS



ANNEXURE K: TAILINGS DAM BREACH ANALYSIS REPORT

