CIVIL DESIGN: BRINE POND DESIGN AND STORMWATER MANAGEMENT REPORT

FOR

BANKABLE-FEASIBILITY STUDY FOR THE MINE ACID WATER TREATMENT: INFRASTRUCTURE ENGINEERING AND DESIGN

EXXARO

BCX-000017-12968-ENG-RPT-0038
REVISION NUMBER: 00

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DEFINITIONS

The definitions listed below apply to this document. For more details, please consult the Glossary.

TERM	DEFINITION
APPROVED	Approved by the engineer in writing
CLIENT	Exxaro
DRAWINGS	Fully dimensioned drawings and schedule prepared by the engineer, showing all members with their size, concrete grade and reinforcement layout, and any other information required for construction
ECSA	Engineering Council of South Africa
ENGINEER	The individual, or company, responsible for the design, for the preparation of the Drawings (or approval of Drawings prepared by others) and where applicable, an inspection of construction for conformity with design.
ISO	International Standards Organization
SANS	South Africa National Standards

REFERENCES

The following documents are either **Applicable Documents** - applicable to the extent specified herein and thus forming part of this document. The applicability will generally relate to the Project in terms of the policy, procedures, standards, qualification, etc.; or **Reference Documents** - where the information concerned has been fully extracted from the reference document and added to this document, or where the reference document contains information relevant to this document, or for information only.

DOCUMENT NAME	RELEVANCE
LEGAL REQUIREMENTS	
National Environmental Management Waste Act (Act No 59 of 2008)	
Regulation 634 of the National Environmental Waste Management Act (Act No 59 of 2008), Waste Classification and Management Regulations;	
Regulation 635 of the National Environmental Waste Management Act (Act No 59 of 2008), National norms and standards for the assessment of waste for landfill disposal	
Regulation 636 of the National Environmental Waste Management Act (Act 59 of 2008), National Norms and Standards for the disposal of waste to landfill	
Notice 737 of 2013 by the Department of Environmental Affairs, Explanatory Summary of the National Environmental Management: Waste Amendment Bill, 2013	
Gazette No 26187, Revision of General Authorizations in terms of Section 39 of the National Water Act, 1998 (Act No 36 of 1998)	
Government Gazette, 18 May 1984 No. 9225, Regulation No 991 Requirements for the purification of wastewater effluent	
National Water Act (Act 36 of 1998)	
Mine Health and Safety Act, 1996	
Mineral and Petroleum Resources Development Act, 2002 (Act 28 of 2002) Section 39 (4) of the Act	
Hazardous Substances Act No 15 of 1973	
Environment Conservation Act No 73 of 1989	
National Environment Management Act No 107 of 1998	
Government Notice No. 704, National Water Act, 1998 (Act No. 36 of 1998) deals with regulations on the use of water mining and related activities aimed at the protection of water resources	
The Dam Safety regulations (published in Government Notice R. 139 of 24 February 2012)	

DOCUMENT NAME	RELEVANCE
BCX-000017-12968-ENG-DCR-0001	Civil/structural design criteria
BCX-000017-12968-ENG-DCR-0004	Mechanical services design criteria

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1. INTRODUCTION

Exxaro's Belfast Coal Mine is in Mpumalanga province and is approximately 19 km southwest of the town of Belfast, on the farms Leeuwbank, Zoekop and Blyvooruitzicht. A locality plan of the mine is presented in Figure 1.

The Belfast Coal Mine has an existing plant with the capacity to process 3 MTPA of run-of-mine material over a Life of Mine of 17 years, producing thermal coal for both domestic and export markets.

The development of the Belfast Coal Mine will unfold in a phased approach, involving the incremental expansion of the mine. The initial phase, referred to as the Belfast Implementation Project (BIP), was initiated in 2018.

Exxaro is seeking to establish the optimal solution to meet their current and future water treatment needs. PROXA was awarded a PFS to assess the viability of these different options to identify the optimal solution. This study concluded that a 1.5-stage process will be optimal regardless of the product water specification.

A Plant Capacity trade-off study evaluated the capital, operating and lifecycle cost implications of different water treatment plant (WTP) capacities and expansion strategies. This trade-off study concluded that the WTP should proceed based on a WTP feed capacity of 2.5 MLD plant and that no provision should be made for incremental future expansion. A brine pond was selected as the optimal brine management option.

The report primary focus lies in the design and sizing of the brine pond and its associated infrastructure. Specific stormwater-related considerations include improvements to existing infrastructure and minor drainage features around the brine pond.

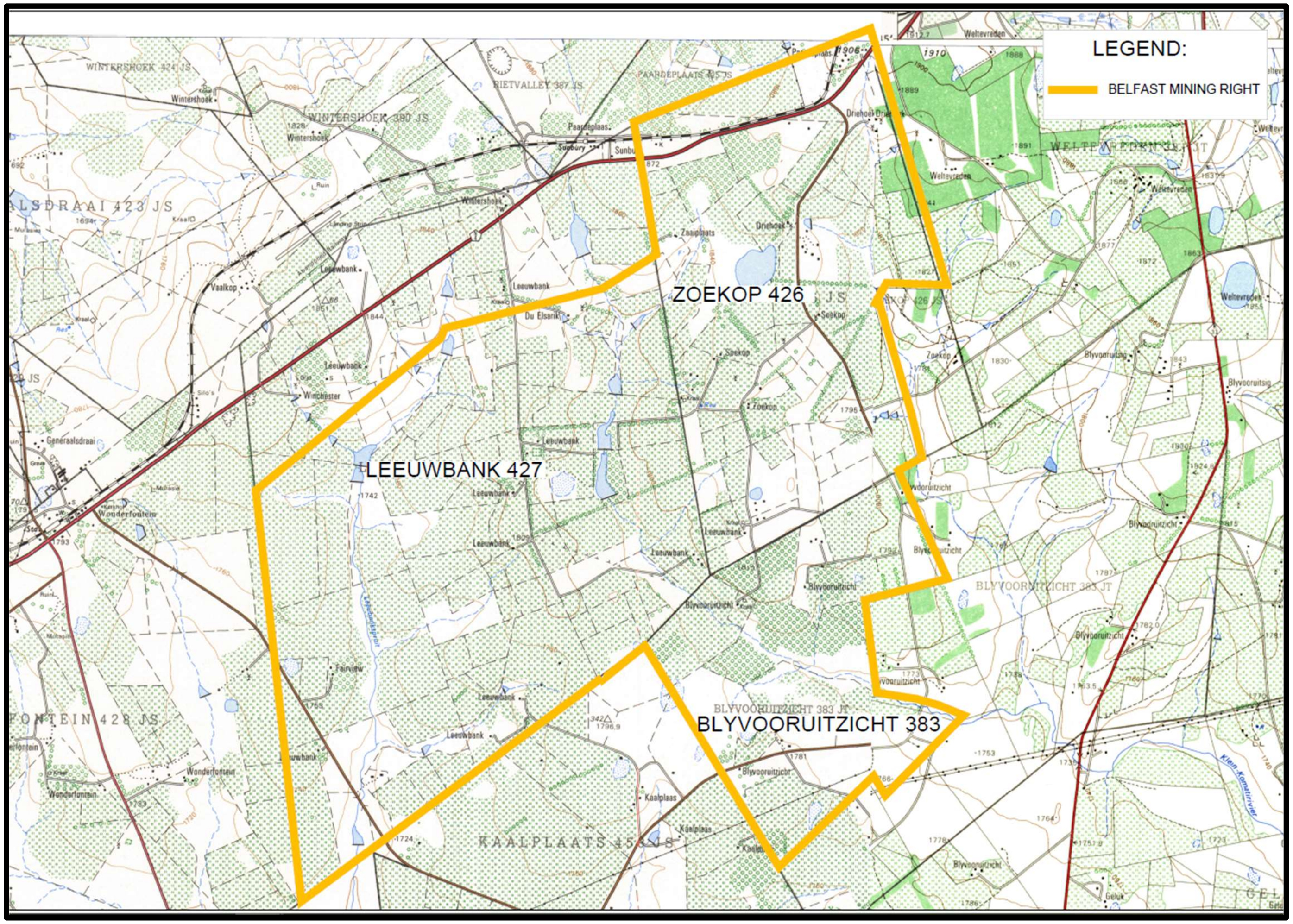


Figure 1: Locality Plan of Exxaro's Belfast Mining Right

2. PURPOSE OF REPORT

The objective of this report is to demonstrate compliance with the pollution barrier design for the brine pond, including the sizing of the pond, and the implementation of effective stormwater management and drainage systems. These measures aim to ensure adherence to the National Water Act (NWA) and other relevant environmental legislation and water management requirements within the mining industry.

These systems have been developed to support a successful Water Use License Application (WULA). The primary objectives of the report are summarised as follows:

- Satisfy the stormwater management and drainage requirements, which form a critical component of the integrated water and waste management of the mine;
- Eliminate the risk of contamination of water resources through the potential ingress of contaminants to the groundwater or by direct stormwater run-off;
- Keep the clean water clean and confined to a clean water system that eventually discharges into the environment under controlled conditions.
- Collect, contain and re-use dirty water in a confined dirty water system;
- Clean and dirty water systems not to spill over more than once in 50 years;
- Make provision for an 800 mm freeboard zone above the Full Supply Level (FSL) of the Pollution Control Dams (PCD);
- Ensure compliance with Dam Safety Regulation;
- Ensure sustainability over the mine life cycle.

3. DESIGN PHILOSOPHY

The following design philosophy has been adopted:

- Pollution Control: Design and implement effective pollution barrier systems for the brine pond to prevent contamination of surrounding areas.
- Stormwater Drainage: Develop drainage systems to manage runoff around the brine pond, ensuring that water is directed to designated areas (such as wetlands) to maintain natural flow patterns.
- Minor Drainage Systems: Include minor drainage features, such as trenches around the brine pond embankments, to manage local stormwater and prevent ponding.
- Erosion Protection: Incorporate erosion protection measures, such as Reno mattresses, at discharge points to prevent soil erosion and manage the transition from pipeline flow to sheet flow.
- Compliance: Ensure all design elements adhere to the National Water Act (NWA) and other applicable environmental legislation, focusing on water management and sustainability within the mining industry.

4. REGULATORY REQUIREMENTS

The design for this scope of work is primarily guided by the requirements of the National Water Act (NWA), 1998 (Act No. 36 of 1998), along with the associated regulations, including GN 704 of 4 June 1999 and the National Norms and Standards for the Disposal of Waste to Landfill (GN R.636 in terms of Section 7(1)(c) of the NEMWA).

Furthermore, the Department of Water and Sanitation (DWS) developed a series of Best Practice Guidelines (BPGs) in order to aid the design engineer to ensure compliance with the regulations from Reg. GN704. A BPG was developed for each facet of the mining spectrum. The following BPG's are of specific relevance:

- G1 – Storm Water Management (DWAF-G1, Aug 2006);
- A4 – Pollution Control Dams (DWAF-A4, Aug 2007);
- A5 – Water Management for Surface Mines (DWAF-A5, Jul 2008)
- G2 - Water and Salt Balances (DWAF – G2, Aug 2006)

5. HYDROLOGY

5.1 INTRODUCTION

This section of the report summarises the key input data necessary for conducting the hydrological analysis of the project, performing the stochastic analysis for the brine pond sizing, and completing the associated civil design scope. For more detailed information regarding the site climatic conditions and hydrology, refer to the following reports prepared by WSP (2024):

- *Water Quality Assessment Update for the Bankable Feasibility Study: Water Treatment Solution*
- *Water Quality Modelling in Support of the Feasibility Study for the Belfast Water Treatment Plant*

5.2 REGIONAL HYDROLOGY

The study area is located within quaternary catchment X11D, part of the Inkomati Water Management Area (WMA). To accurately estimate the peak runoff rate from a catchment, it is essential to consider the temporal distribution of a storm event to account for variations in rainfall intensity during the event. Schulze (1984) developed four synthetic rainfall distributions specific to different regions in South Africa. Based on Schulze's (1984) regionalization, the SCS SA-Type 3 distribution will be applied to the study area as shown in Figure 2.

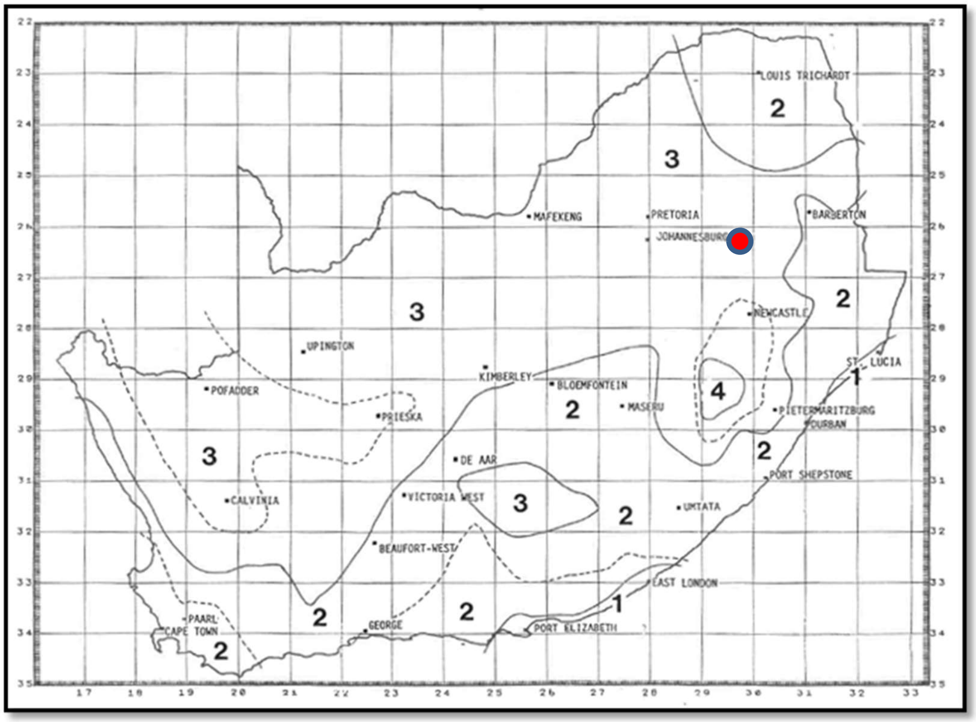


Figure 2: Regionalisation of Synthetic Rainfall Distributions in Southern Africa (Schmidt & Schulze, 1987:153)

5.3 RAINFALL RECORDS

The nearest active rainfall station to the project area is the Roodepoort rainfall station (SAWS No 0516554 W) located approximately 16 km from the study area. This station recorded a Mean Annual Precipitation (MAP) of 697 mm.

Table 1 provides the average monthly rainfall data recorded at the Roodepoort rainfall station.

Table 1: Average Monthly Rainfall recorded by Roodepoort Rainfall Station

Month	Rainfall (mm)	% of MAP
January	119.2	17.1%
February	88.4	12.7%
March	80.2	11.5%
April	44.9	6.4%
May	14.6	2.1%
June	7.0	1.0%
July	4.7	0.7%
August	6.4	0.9%
September	23.2	3.3%
October	70.5	10.1%
November	111.3	16.0%
December	126.8	18.2%
Mean Annual Precipitation (MAP)	**697.2**	**100%**

Design storm estimates for various recurrence intervals are available from the Design Rainfall Estimation Software for South Africa (DRESA), developed by the University of Natal in 2002 as part of a WRC project K5/1060 (Smithers and Schulze, 2002).

The DRESA program provides 1-day rainfall depths, which must be converted to 24-hour continuous maximum rainfall depths.

The distinction lies in the recording methodology: 1-day rainfall depths are based on data recorded between 00:00 and 24:00 of a single day, while 24-hour rainfall depths reflect the maximum rainfall recorded over any 24-hour period, potentially spanning two consecutive days. A correction factor of 1.11 (Adamson, 1981) is applied for this conversion.

The design rainfall depths obtained from the DRESA program, along with the adjusted 24-hour values are presented in Table 2.

Table 2: Design Rainfall Depths for Roodeplaat station obtained from DRESA

Duration	Rainfall Depth (mm)					
	1:2y	1:5y	1:10y	1:20y	1:50y	1:100y
1 Day	50.1	66.4	78.0	89.7	105.8	118.5
24 Hour	55.6	73.7	86.6	99.6	117.4	131.5

WSP obtained daily rainfall records from the South African Weather Service (SAWS) and performed a statistical analysis using the Log-Pearson Type 3 method to calculate peak 24-hour design rainfall depths for various return periods. The results are shown in Table 3. A comparison indicates good alignment between the values derived by WSP and those from the DRESA program.

To ensure consistency in input data across all consultants, the design rainfall values provided by WSP will be used for the hydrological analysis, hydraulic design, and stochastic analysis for brine pond sizing.

Table 3: Statistical Analysis of Roodeplaat rainfall station (WSP, 2024)

Duration	Rainfall Depth (mm)					
	1:2y	1:5y	1:10y	1:20y	1:50y	1:100y
24 Hour	49	67	85	105	138	170

5.4 EVAPORATION DATA

The nearest weather station with a reliable evaporation dataset near the Belfast site is Station X1E003, located at the Nooitgedacht Dam. This station is 16.6 km from the Belfast Mine and has a Mean Annual Evaporation (MAE) of 1,451 mm per annum (**S-Pan**). The record length spans 19 years, from 1961 to 1980. Refer to Table 4 for the mean monthly evaporation values (WSP, 2024).

Table 4: Average Lake Evaporation (WSP, 2024)

Month	Adopted Lake Evaporation (mm)	% of MAE
January	204.6	10.7%
February	177.6	9.3%
March	173.4	9.1%
April	135.6	7.1%
May	114.6	6.0%
June	94.7	5.0%
July	102.9	5.4%
August	138.1	7.3%
September	176.1	9.2%
October	193.7	10.2%
November	192.9	10.1%
December	199.7	10.5%
Mean Annual Evaporation (MAE)	**1904**	**100%**

5.5 SOIL PARAMETERS

The SCS Runoff Curve Number Method of hydrological soil classification and Water Resources of South Africa, 2005 Study (WR2005) has been implemented to obtain the relevant properties of the soils that are present in the project area. The parameter which provides the basis for a hydrological classification of soils in South Africa is a typical amount infiltration for the soil at likely moisture content to the point of maximum run-off rate. (Schulze and Arnold, 1979). This data is summarised in Table 5.

Table 5: Summary of Soil Parameters

SCS Soil Classification				Horton Infiltration Parameters			
Code	Soil Form	Soil Series	Textural Class	SCS Group	Anteceding Moisture Conditions	Max Infiltration Rate (mm/hr)	Min Infiltration Rate (mm/hr)
Hu26	Hutton A	Msinga	SCILm	A	Saturated	34	25

6. BRINE POND DESIGN

6.1 WATER BALANCE

6.1.1.1 PURPOSE

The brine pond sizing assessment was conducted using the GoldSim Modelling Software (GoldSim Technology Group, 2023) to provide a dynamic and comprehensive analysis of water flow and storage throughout the facility's lifecycle of 20 years. This analysis aimed to determine the required storage volume for a 2% Annual Exceedance Probability (AEP), equivalent to a 1:50-year spill frequency, as specified in the National Water Act (NWA) and regulation GN704 under the Water Act.

The GoldSim water balance model was developed in accordance with the Best Practice Guidelines (BPG) for Water and Salt Balances. It simulated site-specific hydrological, operational, and climatic data to model water inputs, outputs, and storage over time. The simulation employed a forecasting approach, incorporating a stochastic rainfall simulator specifically developed for the site. This simulator generates artificial rainfall sequences, which are used as the hydrological input for the water balance model on a daily basis throughout the facility’s lifespan. The model’s primary objective was to determine the required capacity of the brine pond, considering the water treatment plant (WTP) operational requirements and relevant hydrological parameters, in order to meet the legal requirements.

6.1.1.2 DESIGN CRITERIA

The main design criteria for the stochastic analysis of the brine pond are summarised below. For more details, please refer to the Civil Design Criteria document: BCX-000017-12968-ENG-DCR-0001.

- Brine Inflow: Calculated based on a 2.5 MLD plant's maximum output at a TDS of 2700 mg/l for a 95% climate scenario. Monthly brine volumes will be determined using flow rates provided by the overall integrated water balance model (developed by others).
- Operation: 24hrs, 365 days per annum;
- Operating Design Life: 20 years;
- Liner Serviceability Design Life : 50 years;

- Evaporation:
  - Mean Annual Evaporation (MAE) 1451 mm per annum (S-pan) (Station X1E003 with record length of 19 years);
  - Lake Pan Reduction Factor – Average of 80% (Adjusted Monthly);
  - Salinity Factor – 87%.
- Rainfall:
  - Site Rainfall Record (2018-2024);
  - Roodepoort (0516554 W Roodepoort);
  - MAP = 693 mm;
  - Average Monthly Brine Inflow (95th Percentile): 409 m³/month (over 20years);
- Required Capacity: Based on stochastic analysis. The Brine Pond will be sized to comply with a 2% Annual Exceedance Probability (AEP), ensuring that spillage occurs no more than once in 50 years, based on the monthly brine production rate.
- Brine Pond Water Balance:
  - Inflows: Direct Rainfall and Brine Discharge to Brine Pond;
  - Outflows: Evaporation (Any leakage will be intercepted by the leakage management system and pumped back into the pond).

Figure 3 below presents a schematic diagram representing the model, including the inputs, flow links, storage and outputs as applied in the water balance.

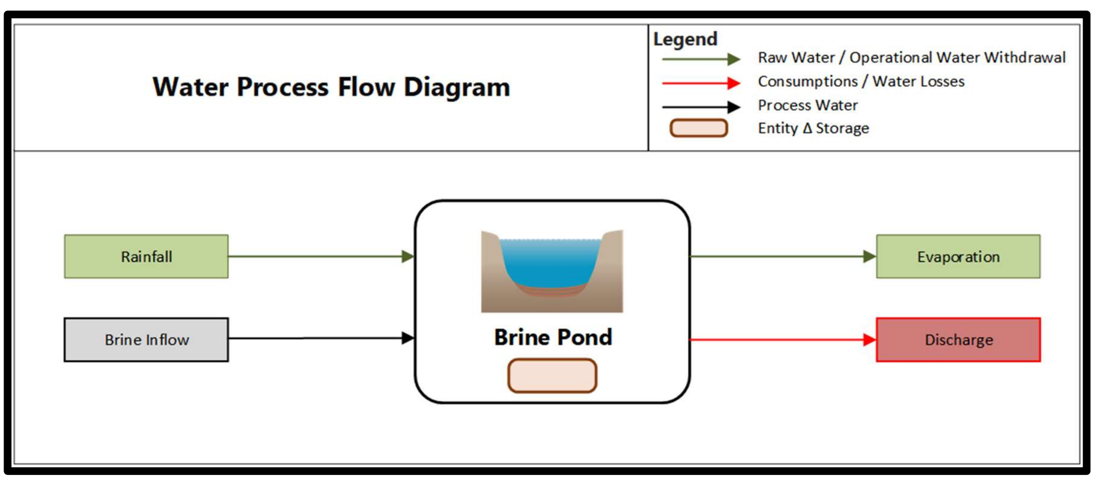


Figure 3: Schematic Water Balance Flow Diagram (No Flows)

6.1.1.3 *HYDROLOGICAL PARAMETERS*

The brine pond's required size will be influenced by two primary hydrological factors: rainfall and evaporation. The only other inflow to the pond is the brine itself, while the outflow consists of spillages (if any).

A stochastic rainfall simulator was set up in GoldSim using historical data from the Roodepoort rainfall station (Section 5.2). The simulated monthly rainfall distribution was then compared and calibrated against the measured monthly rainfall distribution from the same station. The results of both the historically recorded and the stochastic simulated rainfall are presented in Table 6. The calibration of the stochastic rainfall simulator was successful, with the simulated Mean Annual Precipitation (MAP) within ±0.1% of the historical MAP.

Table 6 : Average Monthly Historical vs. Simulated Stochastic Rainfall

Month	Historical Rainfall (mm/month)	Simulated Rainfall (mm/month)
January	120.0	121.4
February	89.9	90.7
March	80.8	80.7
April	43.1	42.8
May	14.7	14.6
June	7.2	7.1
July	5.2	5.4
August	7.3	7.4
September	22.1	23.1
October	70.1	70.4
November	108.5	108.9
December	122.5	124.0
Total	**691.4**	**696.5**

The MAE provided earlier (Section 5.4) is the total potential Symons Pan (S-Pan) evaporation which would not be correct to apply directly to the pond surface. Lake factors, assumed for this assessment are detailed in Table 7. Evaporation was then further reduced by 87% to account for the loss of evaporation due to the salinity the water within the brine pond.

Table 7: Monthly Evaporation Lake Factors

Month	Lake Factor (%)
January	84%
February	88%
March	88%
April	88%
May	87%
June	85%
July	83%
August	81%
September	81%
October	81%
November	82%
December	83%

6.1.1.4 Water Balance Inputs and Assumptions

The brine pond sizing scenario was based on the following parameters:

- Rainfall into the brine pond was calculated using the maximum surface area of the pond (i.e., at the crest elevation).
- Evaporation was calculated based on the dam volume/area stage curve, with calculations performed on a daily basis.
- The 95th percentile of the brine's daily inflow was assumed to enter the brine pond for storage (Figure 4).
- The volume/area stage curve used was derived from the estimated volume required.
- The required size of the brine pond was determined through iterative calculations, adjusting the dam sizing until the 2% AEP criteria were met. As a conservative approach, the 2% AEP was calculated only for the last five years of the brine pond's life, rather than over the entire 20-year period, since the probability of spills is zero in the initial stages when the dam is empty.

- For each iteration, the water balance was simulated for 500 realisations over the 20-year lifespan of the facility.

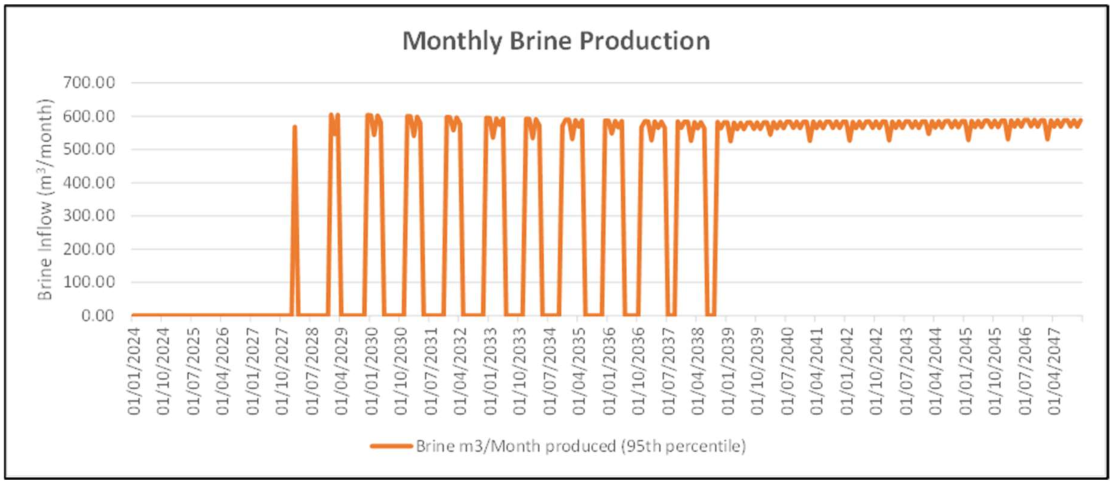


Figure 4: Brine inflow - 95th percentile (WSP,2024)

6.1.1.5 *BRINE POND SIZING ASSESSMENT*

The GoldSim model, using the average flow parameters, is shown in Figure 5.

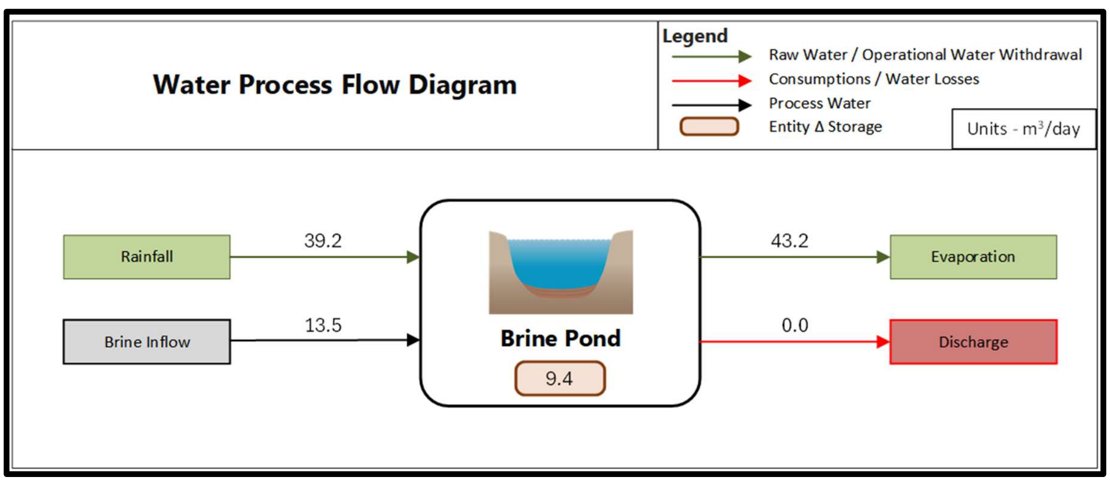


Figure 5: Schematic Water Balance Flow Diagram (Populated)

Various water balance model runs were conducted until the final volume of 80,488 m³ (excluding 800 mm free board), along with the associated surface areas, satisfied the 2% AEP requirement. Additionally, a dual system was considered with the aim of reducing the final footprint. This system would involve constructing a bay for the first 10 years of the facility's life, followed by the construction of a second facility for the remaining 10 years. Even assuming the first facility is rehabilitated immediately after its life and the second facility is immediately available, this approach did not provide a significant difference in the overall design. As a result, it was not studied further, due to additional considerations such as phased construction, contractor onboarding, and the need for ancillary infrastructure like safety measures and fencing

The stage volume, area, and depth curves used for the 80,488 m³ are presented in Figure 6. The simulated brine pond volume over time, for various probabilistic percentiles, is also shown in Figure 7. This figure demonstrates that the brine pond volume only approaches the spillway elevation in the last five years of the facility's life, with sufficient capacity prior to this period.

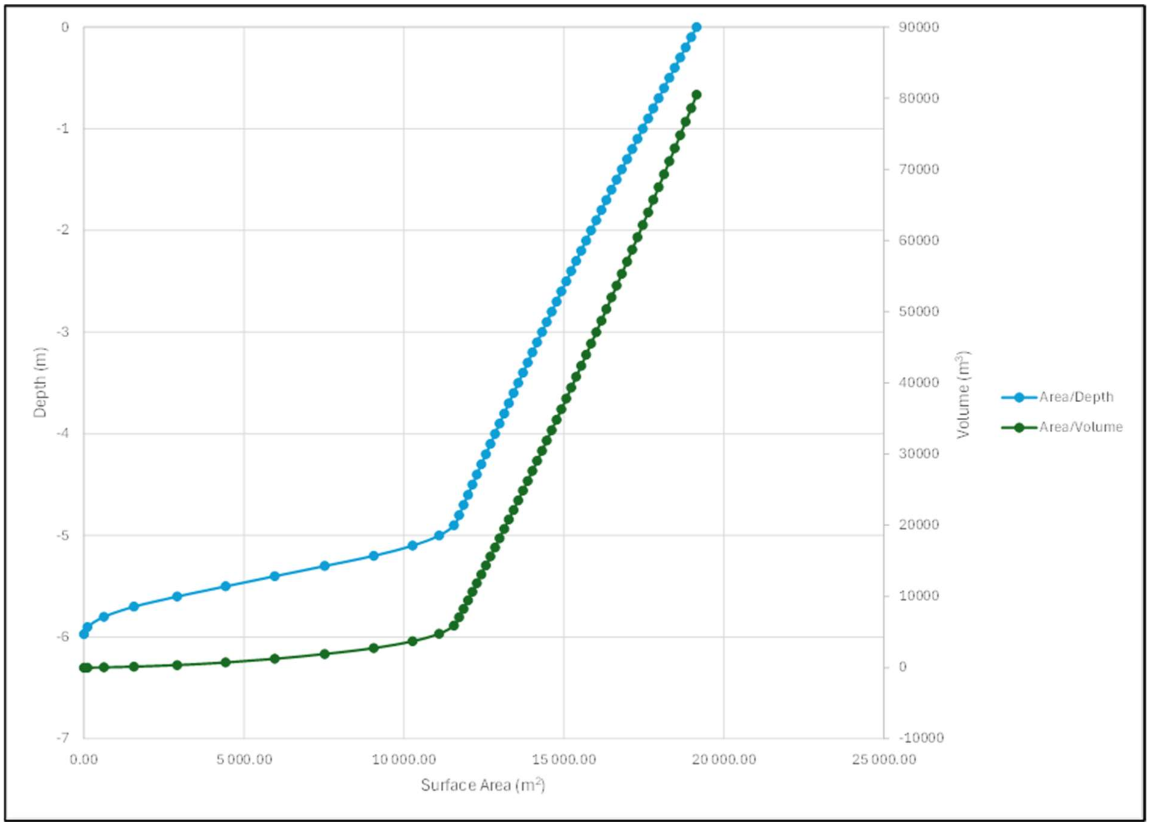


Figure 6: Stage volume, area and depth curve for 80 499 m3 required volume.

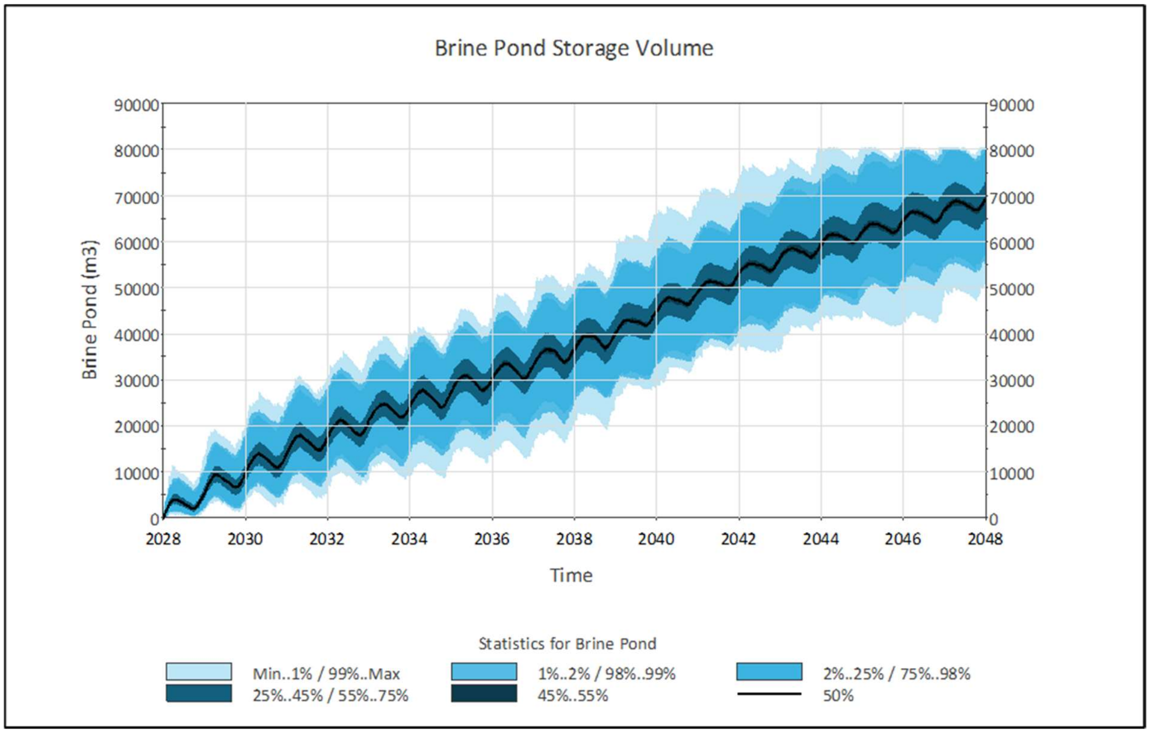


Figure 7: Simulated Probabilistic Brine Pond Volume.

6.1.1.6 DAM SAFETY REGULATIONS

The classification of the brine pond under dam safety regulations, as stipulated by the National Water Act (No. 36 of 1998), was evaluated. A dam can be classified based on the following criteria:

- **Safety Risk:**

  A dam is considered to pose a safety risk if it has a maximum wall height exceeding 5 meters and a storage capacity greater than 50,000 m³. Dam size classifications, based on size class and maximum wall height, are as follows:

  - Small: 5 m, not exceeding 12 m
  - Medium: 12 m, not exceeding 30 m
  - Large: 30 m or higher

- **Hazard Potential Rating:**

  The hazard potential rating evaluates the consequences of a catastrophic failure, including potential loss of life, economic loss, and adverse impacts on resource quality (Resource Quality Objectives). This is a risk-based assessment.

Based on these criteria, the Department of Water and Sanitation (DWS) would classify a dam into one of three categories: Category I, II, or III.

While the pond has a storage capacity of 80,500 m³ (96,381 m³ including 800 mm freeboard), its embankment height is less than 2 meters, as shown in Figure 8. According to dam safety classification criteria, a structure is considered a dam with a safety risk if it meets both of the following conditions:

- Storage capacity exceeds 50,000 m³, and
- Embankment height exceeds 5 meters.

Since the brine pond does not meet the height criterion, it is not classified as a dam with a safety risk under structural regulations.

However, a separate risk-based assessment was conducted as part of the Brine Pond Trade-Off Study (Document No. BCX-000017-12968-ENG-RPT-0007). This assessment used a risk matrix scoring system to evaluate potential hazards. The findings indicated that the brine pond has a very low hazard potential rating in terms of:

- Loss of life, and
- Economic loss.

Despite this, the assessment also considered the potential environmental impact in the event of failure. Specifically, it evaluated the likelihood of exceeding Resource Quality Objectives (RQOs) due to an uncontrolled release of brine. The results indicated that failure would exceed RQO thresholds, leading to a Category II classification based on hazard potential rather than structural safety risk.

This classification under hazard potential has regulatory implications, which will be addressed in the next project phase to ensure compliance with environmental and water management requirements.

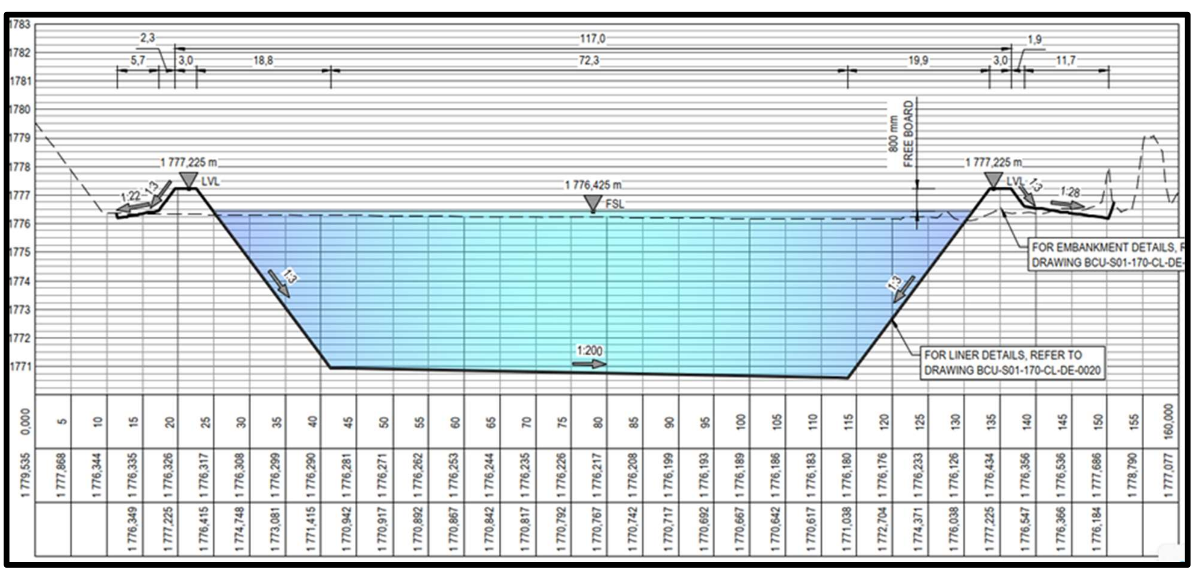


Figure 8 :Brine Pond Cross Section

6.1.2 SPILL WAY DESIGN

The brine pond spillway is designed to connect to the existing dirty water channel downstream of Stormwater Dam 4. This proximity was a key factor in selecting the location, as it allows integration with the mine's managed dirty water system.

By directing any overflow during a spill event into the dirty water system, the design minimises risks to existing infrastructure and mitigates potential environmental impacts. Overflow is ultimately routed to Dam 5, ensuring containment within the dirty water system.

The brine pond spillway arrangement relative to Dam 4 and the existing overflow channel is shown in Figure 9.

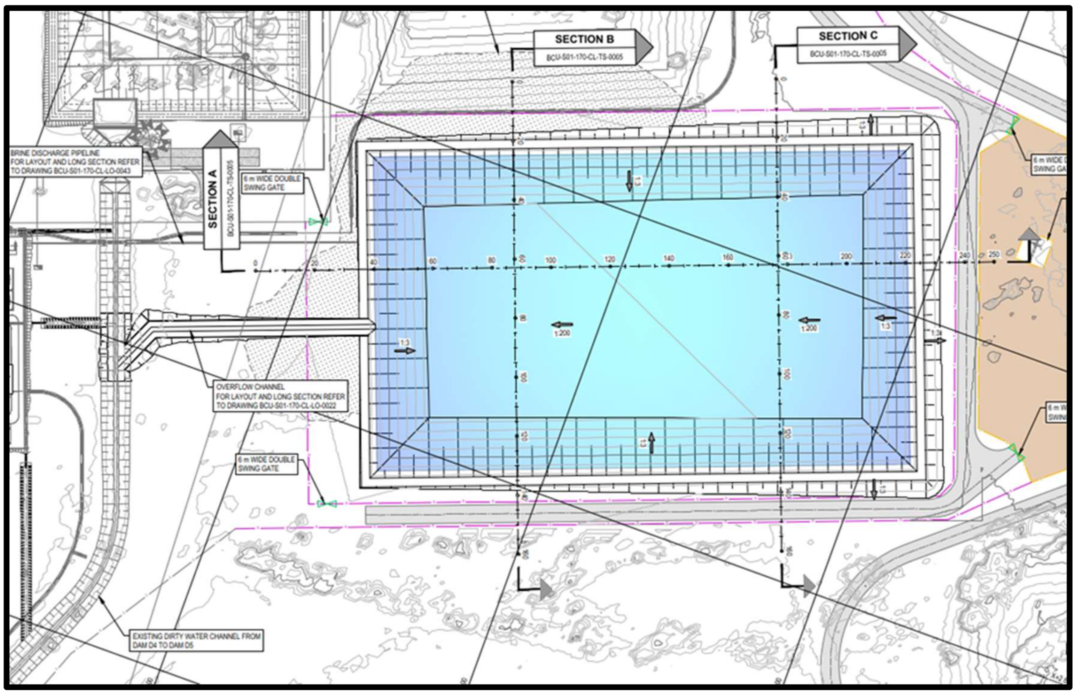


Figure 9: Brine Pond Spillway Arrangement

The design of the brine pond's spillway will adhere to the freeboard guidelines specified in BPG A1 which specifies a minimum freeboard of 800 mm. In this case, only the freeboard zone is applicable, as the brine pond is not designed to include an operating or flood buffer due to its nature as a storage facility for brine that will not be routinely pumped empty.

The freeboard zone, located above the dam's full supply level, is intended to provide additional capacity for managing inflow during extreme events, such as significant rainfall scenarios. This zone ensures the structural integrity and operational safety of the brine pond during such occurrences.

The brine pond spillway was analyzed under a 1:100-year, 24-hour storm event using the PCSWMM software program (Figure 10). This hydrological and hydraulic simulation tool utilises the kinematic method to compute runoff and analyze peak stormwater discharges.

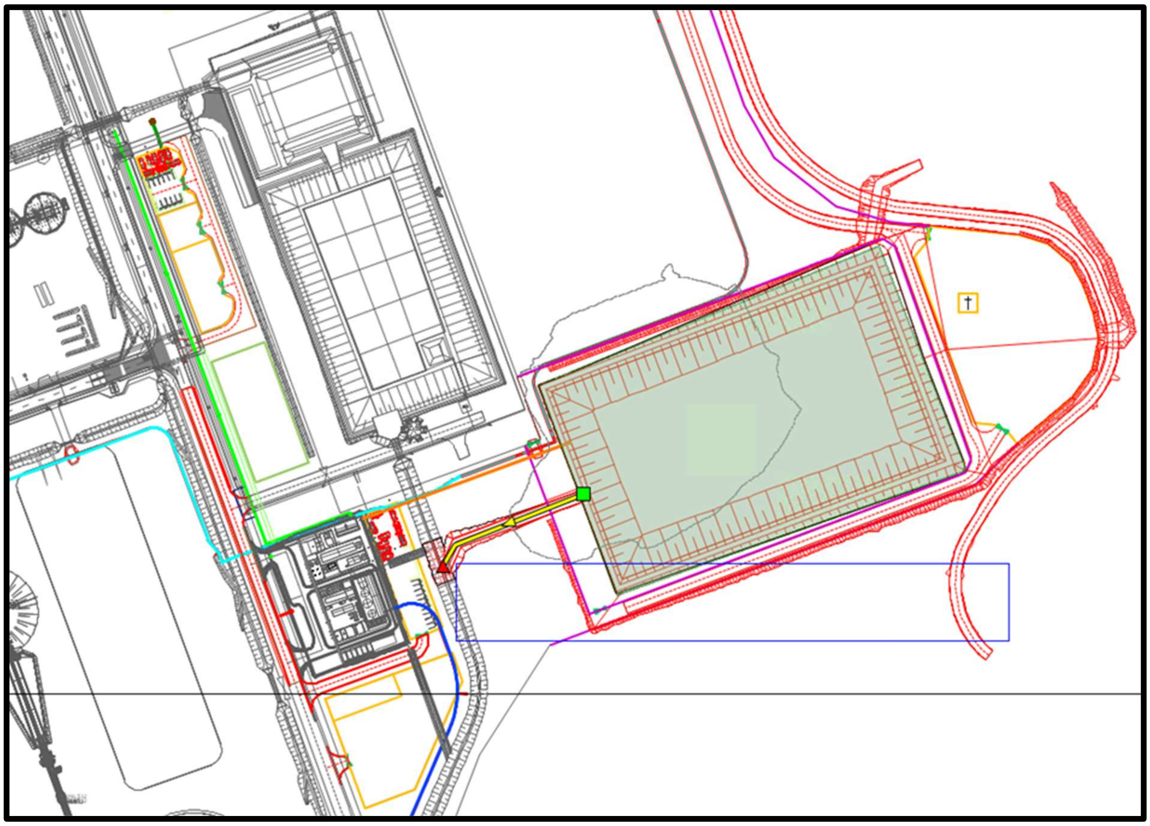


Figure 10: PCSWMM Model: Brine Pond Spillway

As outlined in Section 6.1.1.2, the brine pond does not receive inflow from external catchments; its catchment area consists solely of its maximum surface area, with 100% runoff and zero infiltration during major events. The modeling was performed with the water level in the brine pond at FSL (at spillway level) to simulate the worst-case scenario.

Table 8 summarises typical parameters for various land uses, but only the dam's attributes are applicable:

Table 8: PCSWMM Catchment Parameters for different Land Uses

Sub-Catchment Attributes		Land Use							
		Bush	Dams	Bare Ground	Paved Areas	Plant & Offices	Stockyard – Empty	Stockyard - Full	Surfaced Roads
Land Use Surface Parameters									
Imperv	**%**	20	100	30	100	65	40	40	100
N Imperv		0,13	0,01	0,02	0,02	0,02	0,02	0,02	0,02
N Perv		0,13	0,01	0,02	0,015	0,02	0,06	0,06	0,02
Dstore Imp	**mm**	5	0	5	2	2	5	10	2
Dstore Perv	**mm**	10	0	10	2	5	10	80	2
Zero Perv	**%**	0	100	0	0	35	0	0	0
Land Use Surface Parameters									
Max Infl. Rate	**mm/hr**	34	0	34	34	34	80	80	20
Min Infil. Rate	**mm/hr**	25	0	25	25	25	20	20	5
Decay Const.	**1/hr**	4	0	4	4	4	4	4	4
Drying Time	**days**	2	0	2	2	2	3	3	3
Max Volume	**mm**	0	0	0	0	0	20	60	0

The Key results from the analysis are displayed in Figure 11 and include the following:

- Peak Overflow Discharge: 0.26 m^3/s.
- Maximum Water Depth: 6.09 m, which is 0.68 m below the crest elevation, leaving a freeboard of 680 mm during the storm event.

This indicates that the freeboard volume is adequate to manage a major storm event even when the pond is at its F.S.L. Reducing the spillway dimensions would compromise the freeboard volume, increasing the risk of overtopping the crest during extreme events.

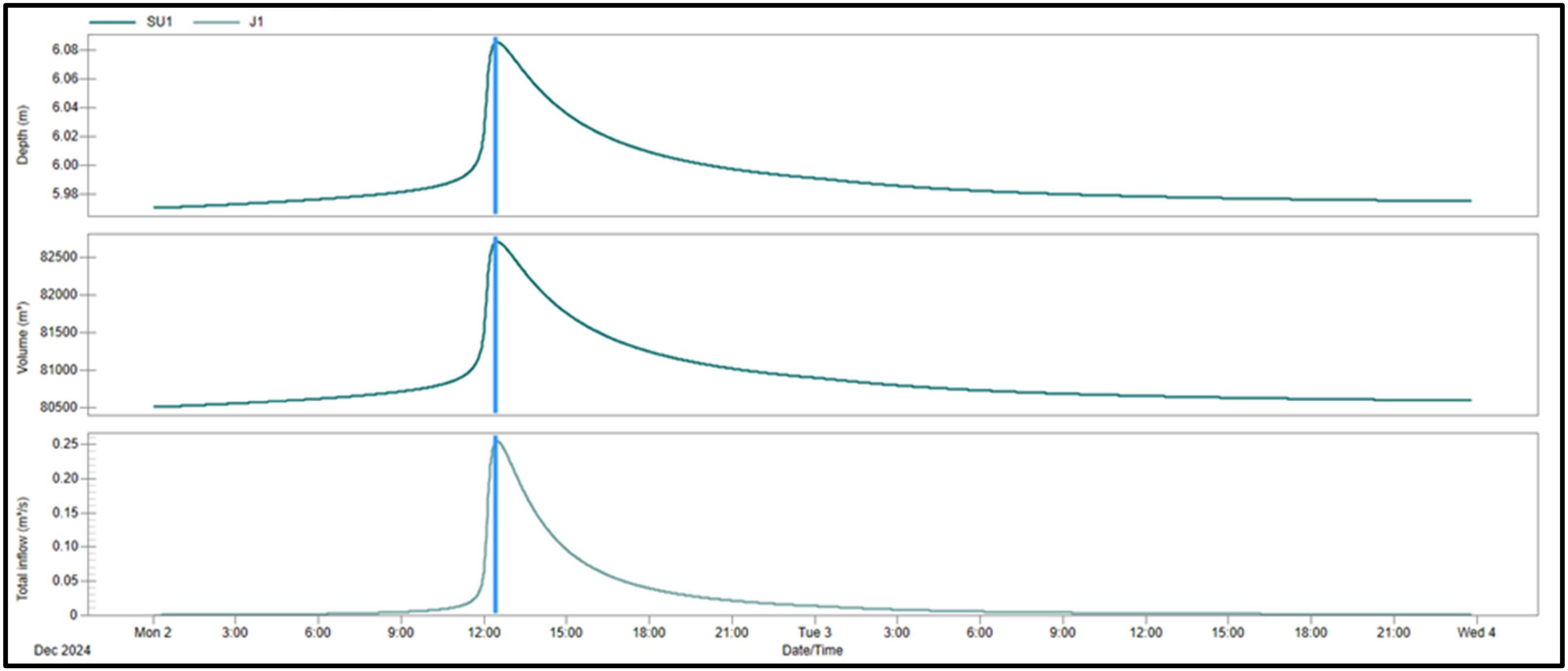


Figure 11: Brine Pond Spillway PCSWMM Results

7. STORMWATER MANAGEMENT INFRASTRUCTURE

The stormwater management scope for this project is limited. Specific stormwater-related considerations include improvements to existing infrastructure and minor trenches around the brine pond to manage water flow effectively. And will be discussed in the next sections.

7.1 WTP ACCESS ROAD UPGRADE

As part of the project, the existing gravel road providing access to the Water Treatment Plant (WTP) will be surfaced. The pavement details are provided in the Civil Design Report (BCX-000017-12968-ENG-RPT-0011), and the layout of the access road is shown in Figure 12.

The road will be sloped to direct runoff into the existing stormwater trench, ensuring effective drainage without requiring additional design modifications. Stormwater chutes, 1 meter wide and stone-pitched, will be installed every 30 meters and sloped towards the dirty water drain to manage runoff effectively. Kerbs on both sides of the road will further direct runoff into the chutes.

The surfaced section of the road, approximately 167 meters in length, will not impact the capacity of the existing stormwater trench due to the relatively short distance. The pre- and post-construction conditions are therefore effectively similar in terms of stormwater runoff.

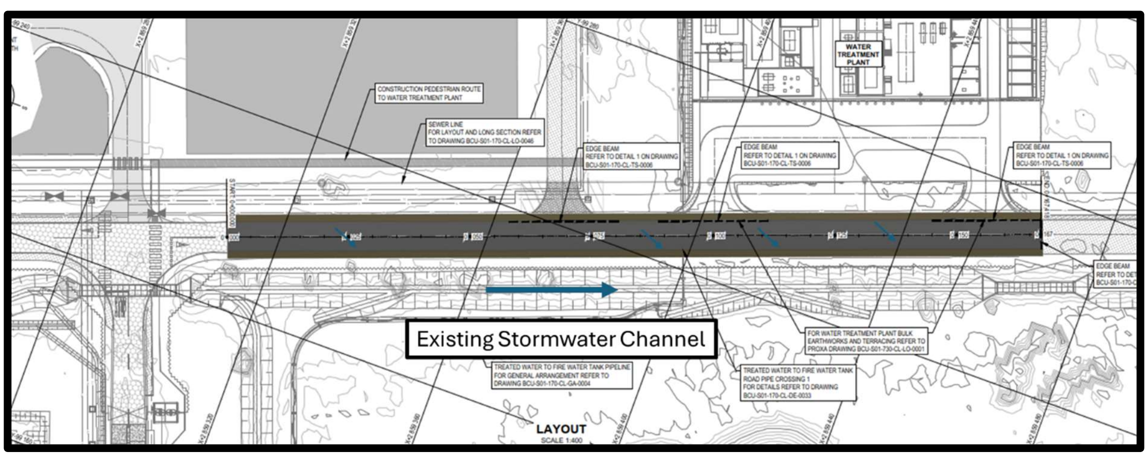


Figure 12: WTP Access road layout

7.2 MINOR SYSTEMS: DRAINAGE AROUND THE BRINE POND

To manage local stormwater around the brine pond, minor trenches will be constructed along the northern and southern embankments. These trenches are intended to prevent ponding at low-lying areas along the embankments and are classified as minor drainage systems without any well-defined catchments around the pond. Therefore it is not practical to perform any runoff calculations for the design.

The trenches are designed to direct runoff around the pond perimeter into the adjacent wetlands. The general drainage flow is towards the east, where the water will discharge into the wetlands as sheet flow, following its natural flow path. The discharge flow velocities are expected to remain below 1 m/s, ensuring no risk of erosion.

Localised low points will also be incorporated into temporary roads, including truck delivery routes and farmer access roads, to facilitate drainage. The layout of the minor drainage system is illustrated in the Figure 13 below.

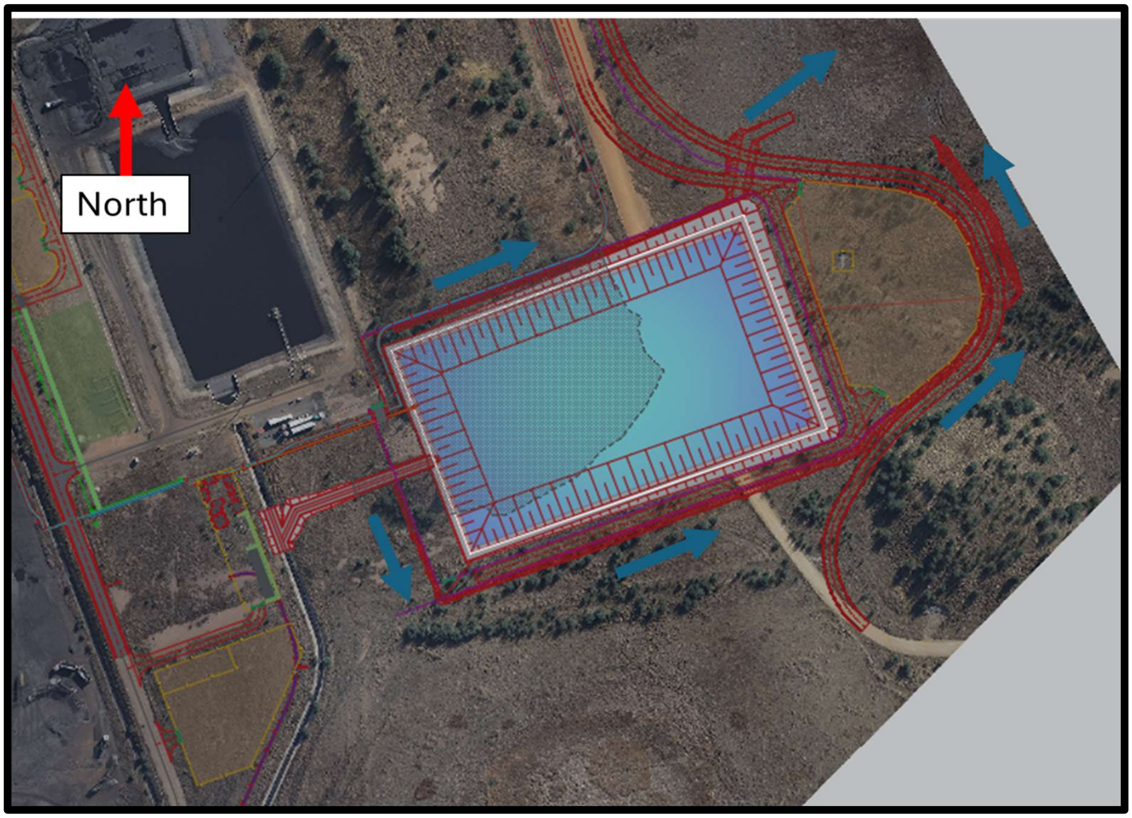


Figure 13: Brine Pond Minor Drainage System

7.3 SPOIL AREA ACCESS ROAD

A temporary construction road will be built to facilitate the hauling of spoil material to the designated stockpile area. As shown in the figure below, the road is located within open veld on the mine property.

The road alignment is primarily parallel to the natural contours of the terrain, minimising disruption to the existing flow patterns within the catchment. At the low point along the alignment, the road will incorporate a drift to allow for the passage of overland stormwater, ensuring that natural drainage is maintained.

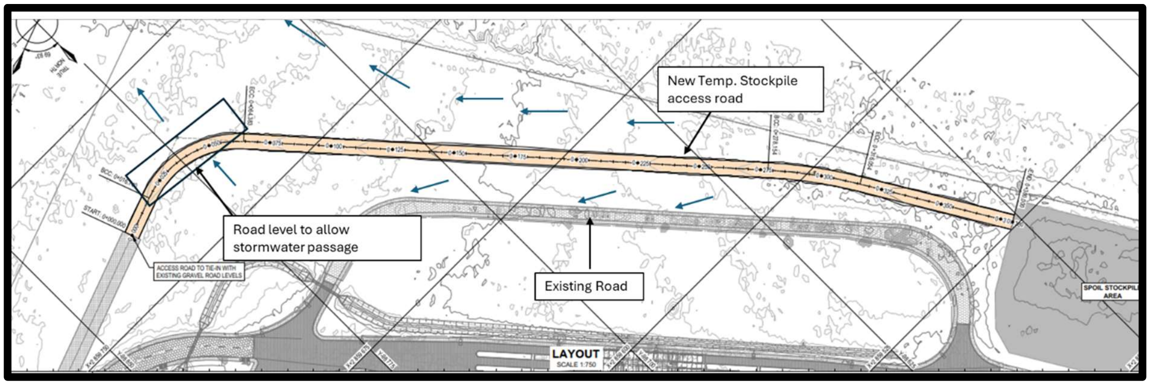


Figure 14: Temporary Construction Road Layout

7.4 EROSION PROTECTION

This section outlines the erosion protection measures for the permeate pipeline, a new overland pipeline designed to safely discharge treated water into the wetland.

The Permeate Pipeline, shown in yellow in Figure 15, consists of a 200 NB HDPE (PN 12.5) pipeline designed to handle a maximum flow rate of 115 m³/h, resulting in a velocity of approximately 1 m/s based on the water treatment plant's operational design.

The pipeline will primarily be installed on natural ground, with certain sections buried where necessary. It will cross multiple existing visible and buried services along its route, as illustrated in Figure 15. To accommodate these crossings, new culverts and supporting infrastructure, such as pipe bridges, will be installed where required.

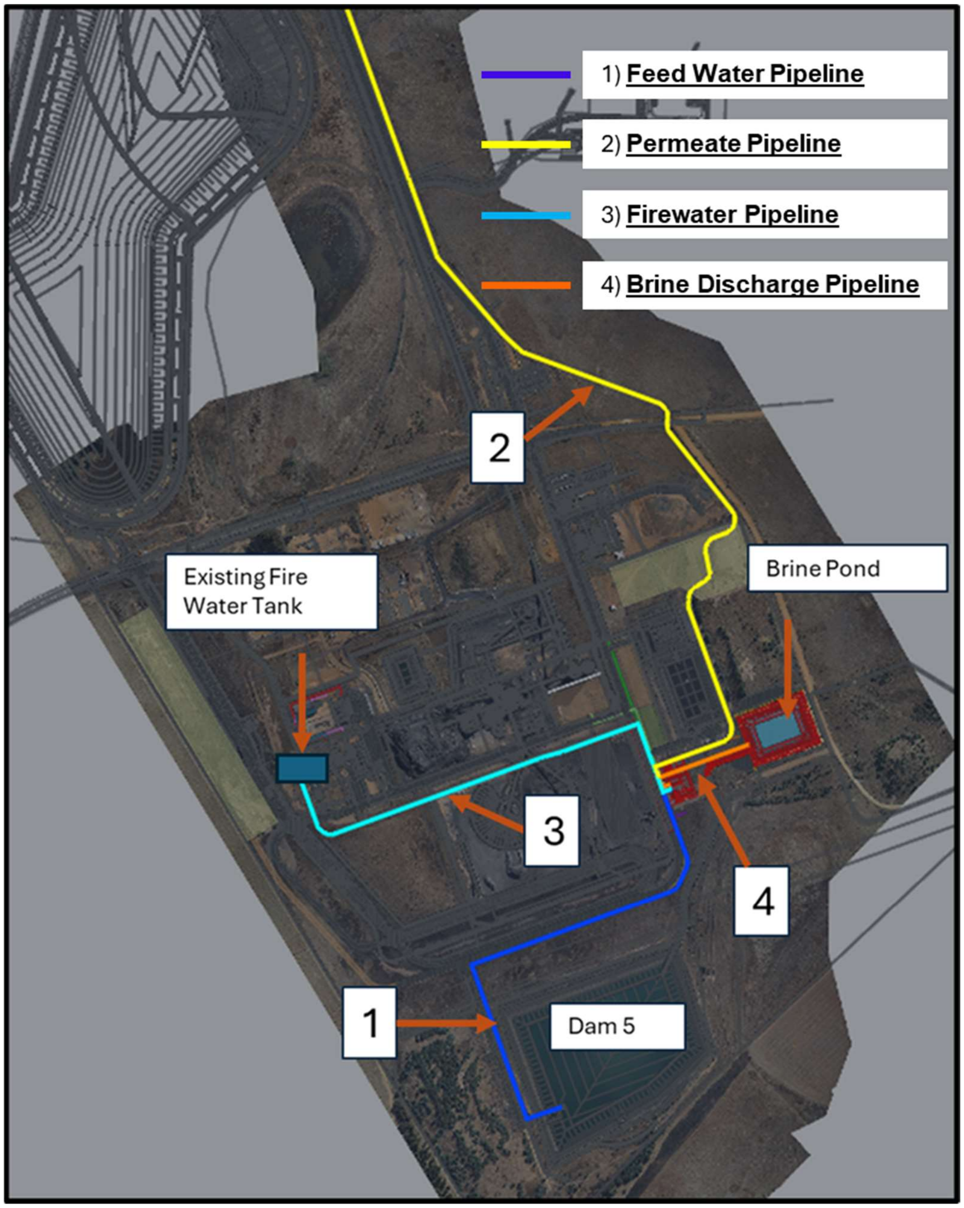


Figure 15:Overland Pipe Routes

To mitigate the risk of erosion at the outlet, a Reno mattress erosion protection structure will be installed. Reno mattresses are thin, flexible rectangular mesh cages made from double-twisted wire mesh, designed as a continuous panel to eliminate structural weak points. These structures can accommodate water velocities up to 6 m/s for prolonged periods, making them effective for this application. The structure will gradually flare open to promote sheet flow at the discharge location.

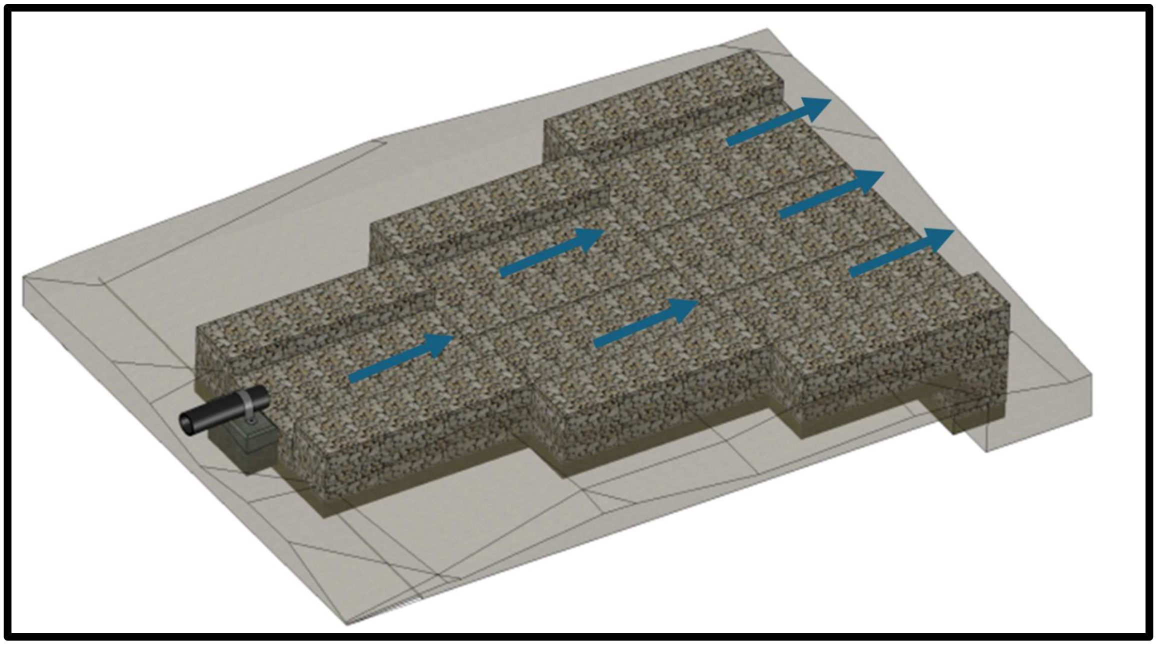


Figure 16: Permeate Discharge Line: Erosion Protection Structure

The Reno mattress outlet structure will include a non-woven geotextile fabric (Class 1, per GRI-GT13(a)) as a filter layer to prevent finer material displacement. The Reno mattress will be placed directly on the geotextile fabric, as illustrated in the design detail below. A 0.5 m toe section will be constructed at the outlet to prevent washout and scouring.

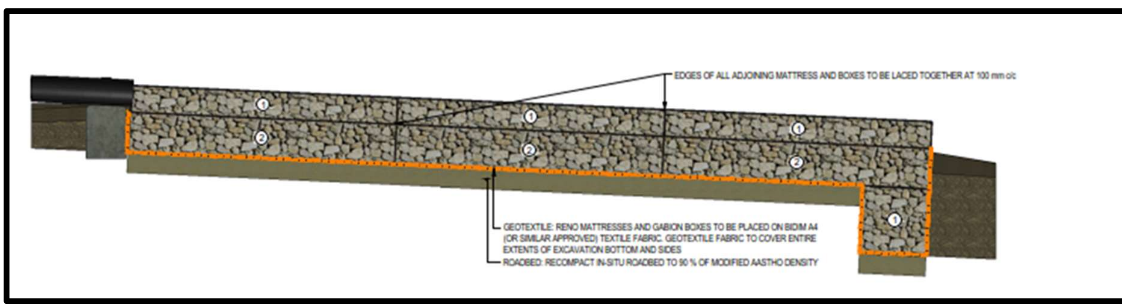


Figure 17: Reno Mattress Separation Geotextile

Using Manning's equation (with a roughness coefficient $n = 0.03$), the outlet velocity is calculated at 0.25 m/s with a flow depth of approximately 0.0425 m (refer to . After discharging, the water will transition to sheet flow. According to the SANRAL Drainage Manual, this velocity is suitable for even fine sand material and will not cause erosion within the wetland, which is assumed to be lightly vegetated.

Additionally, the low flow velocity mitigates the risk of silt deposition, as the permeate water is clean and treated by the Water Treatment Plant (WTP). This ensures the integrity of the wetland ecosystem is preserved.

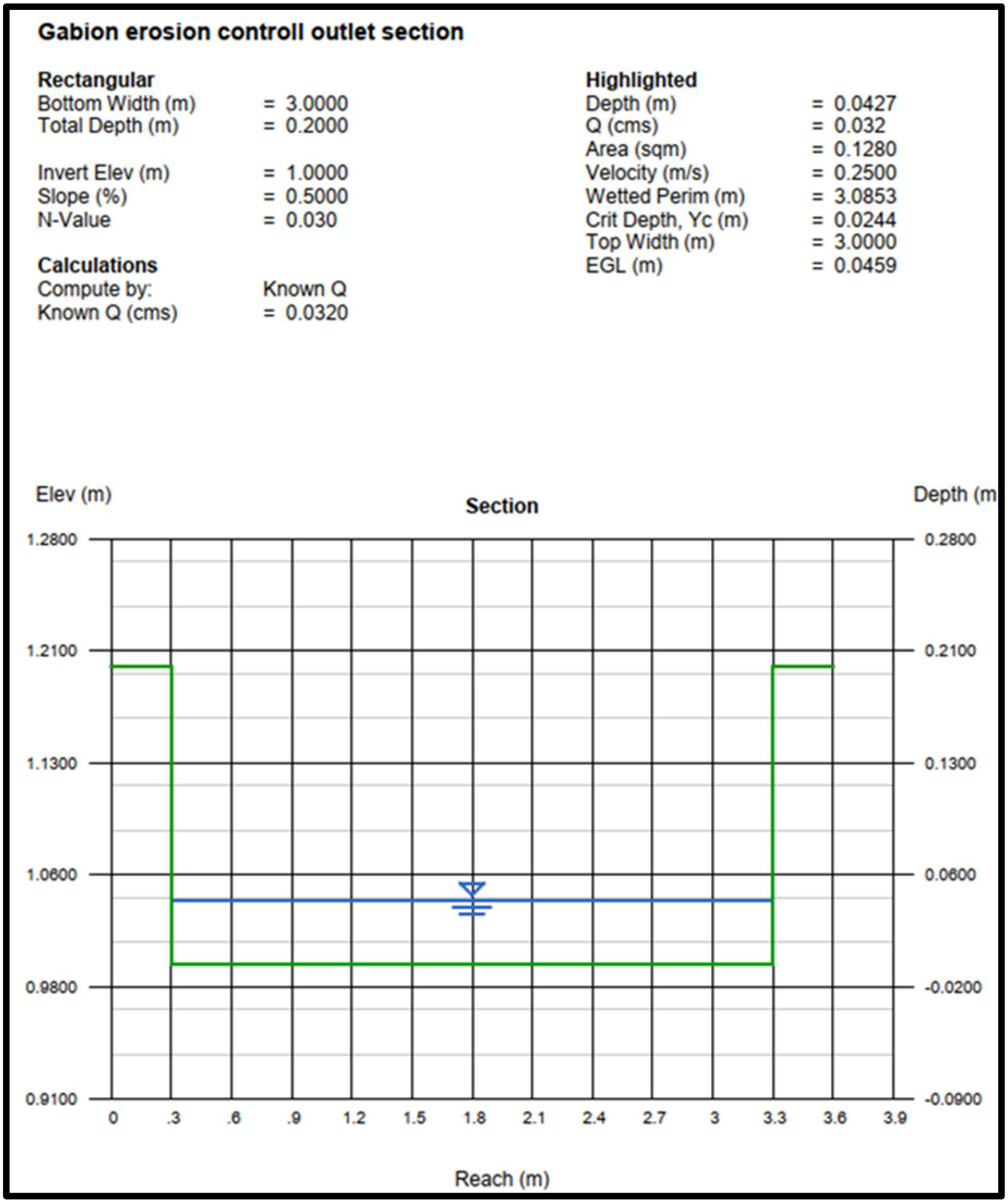


Figure 18: Channel hydraulic Calculation: Erosion Protection Structure

8. LINER DESIGN

8.1 WASTE ASSESSMENT

GCS Water and Environment (Pty) Ltd was appointed by Exxaro Coal Mpumalanga (Pty) Ltd to conduct a waste classification assessment for the liquid brine associated with the proposed Belfast Brine Pond. This study supplements the Waste Management License (WML) and Water Use License (WUL) applications for the facility, ensuring compliance with the National Environmental Management: Waste Act (NEM:WA) and the following regulations:

- Waste Classification and Management Regulations (GN 634 of 2013),
- National Norms and Standards for the Assessment of Waste for Landfill Disposal (GN 635 of 2013), and
- National Norms and Standards for Disposal of Waste to Landfill (GN 636 of 2013).

Brine Sample Characteristics:

A representative liquid brine sample, taken from the liquid waste stream associated with the Belfast water treatment process, was submitted to Talbot Laboratories for analysis. The sample represents the leachable concentrations from the initial phase of brine treatment. Key findings from the assessment include:

- The brine is classified as Type 2 Waste, primarily due to Total Dissolved Solids (TDS) exceeding the LCT1 threshold (>12,500 mg/L).
- Other detected constituents, including Boron (B), Chromium (Cr), Manganese (Mn), Molybdenum (Mo), Nickel (Ni), and Sulphate (SO_4), remained below the LCT1 thresholds but contribute to the waste profile.
- Disposal at a conventional landfill is prohibited under Regulation 5(1)(s) of GNR 636 (2013) due to the high salt content and elevated leachable concentrations.

Liner Design Implications

- Based on these findings, a Class B barrier system is recommended for the brine pond, incorporating the following elements:
- Primary Liner: High-Density Polyethylene (HDPE) geomembrane with a minimum thickness of 1.5 mm.
- Secondary Barrier: A compacted clay layer or Geosynthetic Clay Liner (GCL) with a permeability of less than 1 x 10^{-9} m/s.
- Leachate Collection System: Drainage layer designed to capture and manage leachate effectively.
- Protective Layer: Soil or geotextile layer to safeguard the liner system during construction and operation.

The waste classification assessment confirms the suitability of a **Class B liner** system for managing the liquid brine. This design aligns with regulatory requirements and addresses environmental risks associated with high-TDS waste storage. The results are consistent with similar brine classifications conducted for other industrial facilities.

Based on the above, brine sample has been classed as a **Type 2** waste and should be disposed of or stored at a facility with a **Class B** liner or a system with similar properties.

A conceptual design for a Class B liner as given by the NEM: WA guidelines (DEA, 2013) is shown below in Figure 19.

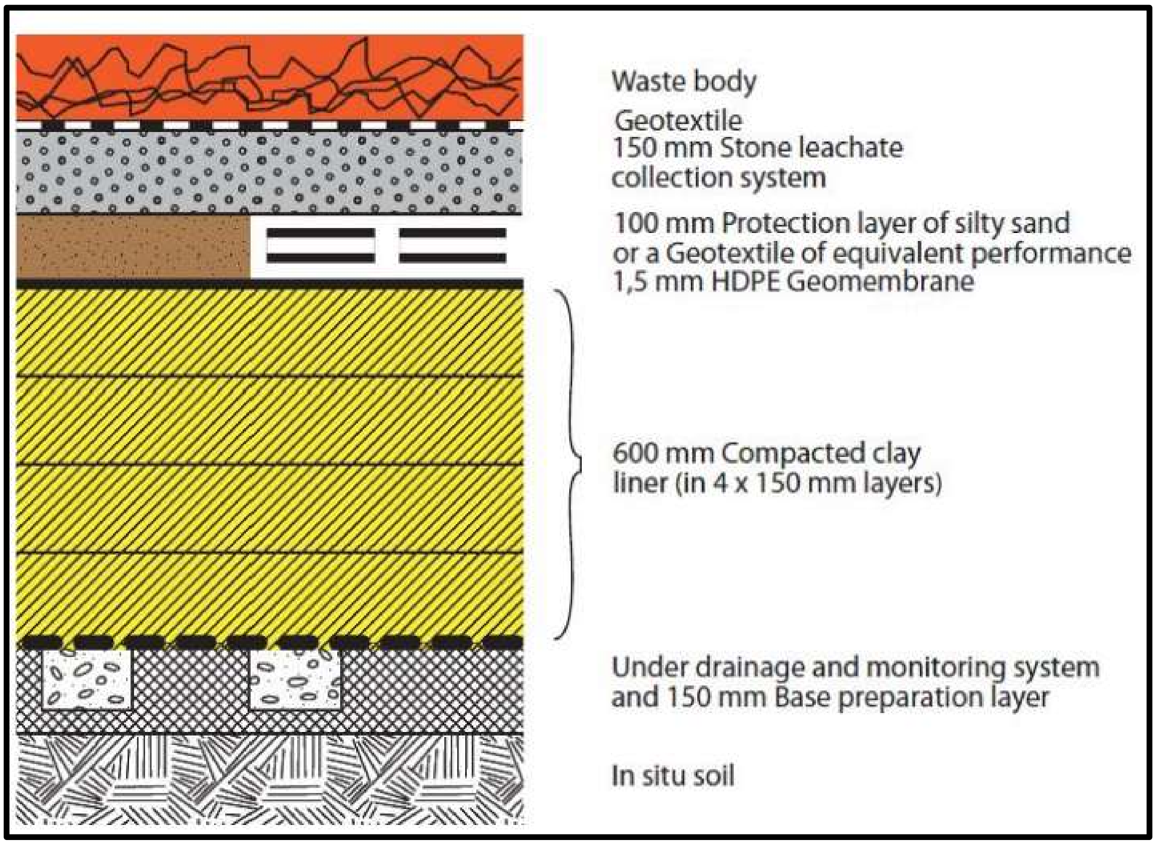


Figure 19: Typical Class B Liner Composition (NEMA reg 636)

8.2 PROJECT COMPOSITE LINER SYSTEM

The following section of this report will detail the liner system specifically designed for the proposed brine pond, demonstrating its compliance with the regulatory Class B liner requirements and its ability to achieve equivalent performance in mitigating environmental risks.

8.2.1 THEORETICAL OVERVIEW OF MAIN LINER LAYERS

8.2.1.1 CELLULAR CONFINEMENT SYSTEM

Stiff-walled HDPE geocells play a vital role in enhancing the performance of liner systems, particularly in applications like brine ponds where high chemical loads and mechanical stress are common. These geocells, made from durable high-density polyethylene, provide structural support and improve load distribution across the liner, preventing deformation or settlement of the underlying materials. Their rigid walls help confine the infill material, such as soil or geosynthetic layers, preventing lateral movement and enhancing the overall stability of the liner system. Moreover, their honeycomb structure contributes to erosion control, safeguarding the liner from mechanical forces or water runoff that could compromise its integrity. While the initial cost and installation complexity may be higher, the long-term benefits, including increased durability, enhanced load distribution, and improved environmental protection, make stiff-walled HDPE geocells an essential component in achieving reliable and effective containment in liner systems.

8.2.1.2 GEOTEXTILE

The continuous filament, nonwoven, needle-punched geotextile performs two functions when incorporated as part of the drainage system within a composite liner, i.e. that of a filter and a separator. Fine soil particles are prevented from entering the drainage structure while water passes through, thus lowering the phreatic surface in the surrounding soil.

Furthermore, thick geotextile grades are resistant to abrasion and piercing. Sandwiching the elements of the composite liner between these geotextiles during installation will ensure that minimal damage is caused to the composite liner components.

8.2.1.3 HDPE GEOMEMBRANE LINING

High-Density Polyethylene (HDPE) geomembranes are among the most widely used materials for lining various facilities. HDPE geomembranes are a cost-effective choice for lining projects.

8.2.1.4 GEOSYNTHETIC CLAY LAYER

Geosynthetic Clay Liners (GCL) are bentonite-based liners used for hydraulic barriers for water and waste containment sites. A GCL layer is an alternative to the 300 mm Compacted Clay Liner (CCL) and is proposed due to limited available clay in the area. The geotechnical investigation conducted by Jeffares & Green in 2011 found that most of the soils in the area are predominately sandy with a low Plasticity Index (PI) and heave potential (low clay content).

The GCL layer is also a cost-effective solution for replacing the 300 mm thick clay layers as specified in the typical DWS Class C liner system detail. The GCL is produced by needle punching a uniform layer of sodium bentonite between two layers of geotextile. GCLs have combined properties of low permeability and high internal shear strength, making them versatile hydraulic barriers.

8.2.2 Brine Pond Liner Details

The composite liner system for the brine pond, adhering to the Class B specifications, is illustrated in Figure 20 and will consist of the following layers:

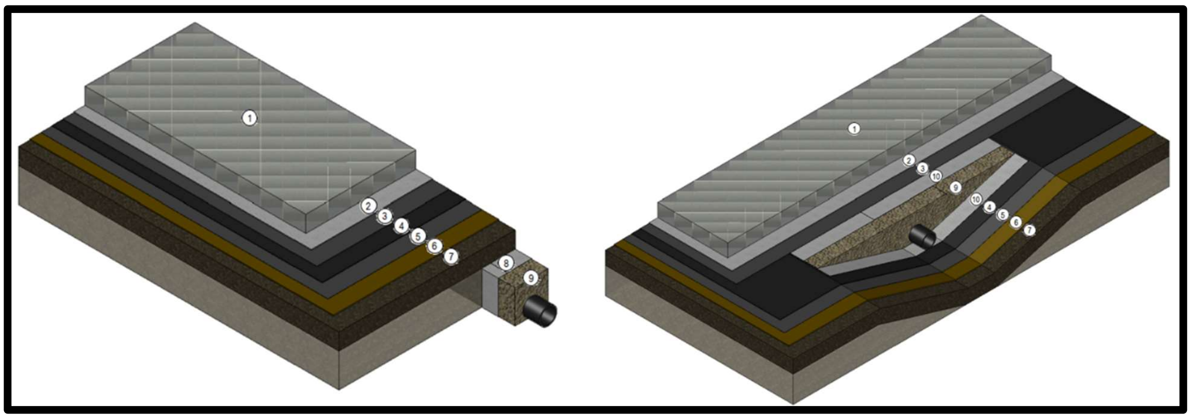


Figure 20: Brine Pond Composite Liner System (with leakage detection and sub-soil drainage system)

1. Mechanical Protection layer (Geocells filled with 6% stabilised cement soil);
2. Liner Protection geotextile: Non-woven, needle punched, continuous filament, polyestergeotextile, 1,32 m standard width (1 000 g/m² puncture-resistant);
3. Primary liner :2 mm thick HDPE smooth geomembrane (According to GRI-GM 13);
4. Geocomposite drainage core, cuspated side top (Min 6 mm deep);
5. Secondary Liner :1.5 mm thick HDPE smooth geomembrane (According to GRI-GM 13);
6. Geosynthetic Clay Liner (GCL) (reinforced, 3700 g/m^2 mass sodium bentonite, according to GRI-GCL 3);
7. Engineered soil subgrade with the surface finish to lining requirements: 150 mm thick G7 quality levelling layer from free-issue or commercial sources, compacted to 95 % of modified AASHTO density;
8. Non-woven, needle punched, continuous filament, polyester geotextile, 1,32 m standard width (180 g/m² puncture-resistant);

9. 19 mm washed crushed stone wrapped inside a geotextile;
10. Non-woven, needle punched, continuous filament, polyester geotextile, 1,32 m standard width (180 g/m² puncture-resistant).

A subsoil drainage system has been provided below the liner system at the pond floor level. The subsoil system drains to a small pump sump with a submersible pump which will be pumped back into the brine pond. (Refer to 8.5 for more details)

The purpose of the subsoil Drainage Geocomposite layer beneath the geocomposite liner is to intercept and drain away any groundwater seepage that may occur beneath the brine pond and so prevent hydrostatic uplifting of the installed liner systems.

The composite liner system is protected from loads and potential damage during maintenance activities by a geocell layer filled with cement-stabilized soil.

The mechanical protection layer is also necessary to provide a confining pressure (approximately 4 kN/m²) for the GCL and to improve the contact interface between the GM and GCL.

8.3 GEOTECHNICAL CONSIDERATIONS

8.3.1 INTRODUCTION

The location of the Brine Pond was established based on a trade of study as summarised in the site brine pond location trade of study report. A geotechnical investigation was done at the proposed location. Mukona was responsible for the site work and ARQ geotechnical engineers used the information for the slope stability and liner strain assessment. This section will outline the main findings of the report, specifically related to the liner design. For the detailed geotechnical report, please refer to the following report (BCX-000017-12968-ENG-RPT-0017).

8.3.2 GEOTECHNICAL INVESTIGATION AND SOIL PROFILE

Mukona Geotechnical Engineers excavated four test pits at the proposed brine pond location, which appeared to be virgin soil, as shown in Figure 21. The test pit profiles were provided to ARQ for further assessment. Based on the test pit data, the following general soil profile was derived and is summarised in Table 9

Table 9: General soil profile of Test pits

Depth (m)	Description
0 – 1	Soft rock hardpan ferricrete
>1	Stiff residual sandstone/siltstone

Note: A G7 fill material will be utilised for the construction of the pond embankment – a 1m thick layer of this material was therefore modelled above the in-situ material.

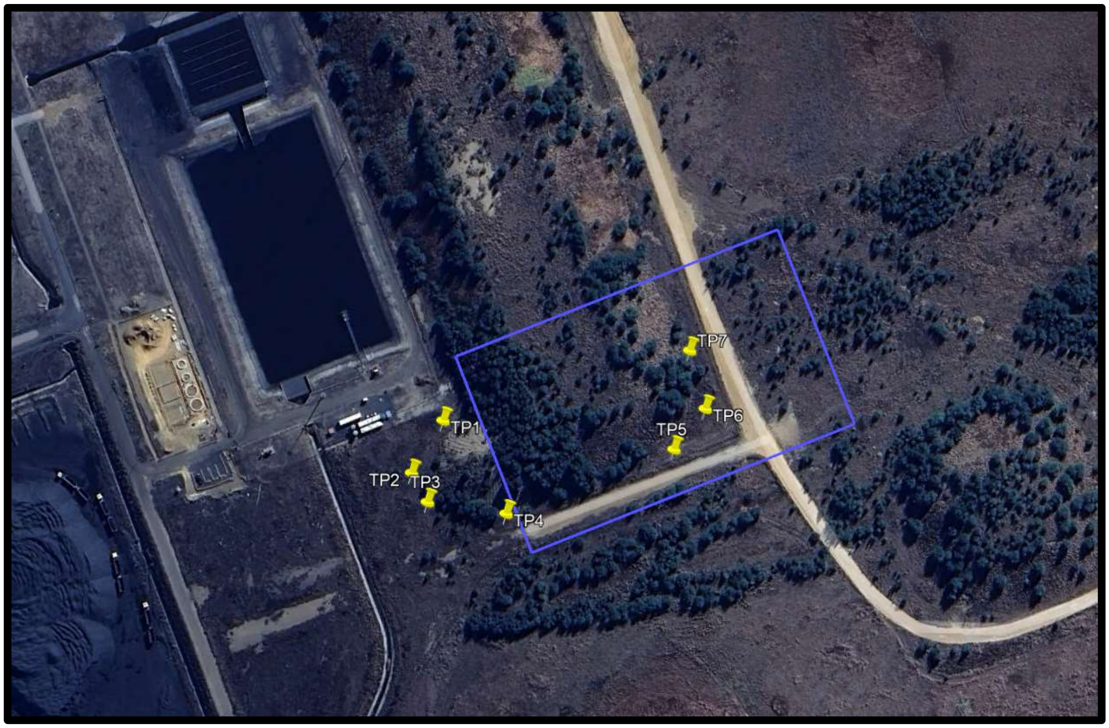


Figure 21: Brine Pond outline (blue) and Mukona test pits

8.3.3 Liner Strain Assessment

The design drawings were used to create a finite element (FE) software model of the brine pond, focusing on a cross-section at the deepest part of the pond. The analysis aimed to evaluate two key aspects: (i) the strains that may develop in the liner when the pond is filled to its design capacity and (ii) the stability of the slope under these conditions. A screenshot of the FE model is presented in Figure 2.

The Class B liner system was adopted as outlined in Section 8.2.

Adopting a conservative and simplified approach (due to software limitations), the liner system was modelled as consisting solely of a 1.5mm thick Hi-driline HDPE liner with smooth interfaces on both sides. No project specific interface friction angles (derived through testing) were provided for the liner system; therefore, published liner property tables (Howell & Kirsten, 2016) were utilised to determine the relevant interface friction angles and tensile properties. According to the published data, the 1.5mm thick HDPE liner has a yield strength of 22 kN/m and a yield elongation of 12%, resulting in a tensile modulus (stiffness) of approximately 183 kPa/m. The internal friction angle between a smooth HDPE liner and a geosynthetic material is reported as 9.6°, with a residual friction angle of 9.4°.

The analysis indicates that the maximum axial force developed in the liner system under maximum pond loading conditions is 0.3 kN/m, corresponding to a strain of 0.17%. Figure 3 below provides a graphical representation of the strain distribution, highlighting areas where strain concentrations or peaks occur. The maximum strain in the liner is calculated at the crest of the slope, where the liner enters the anchor trench.

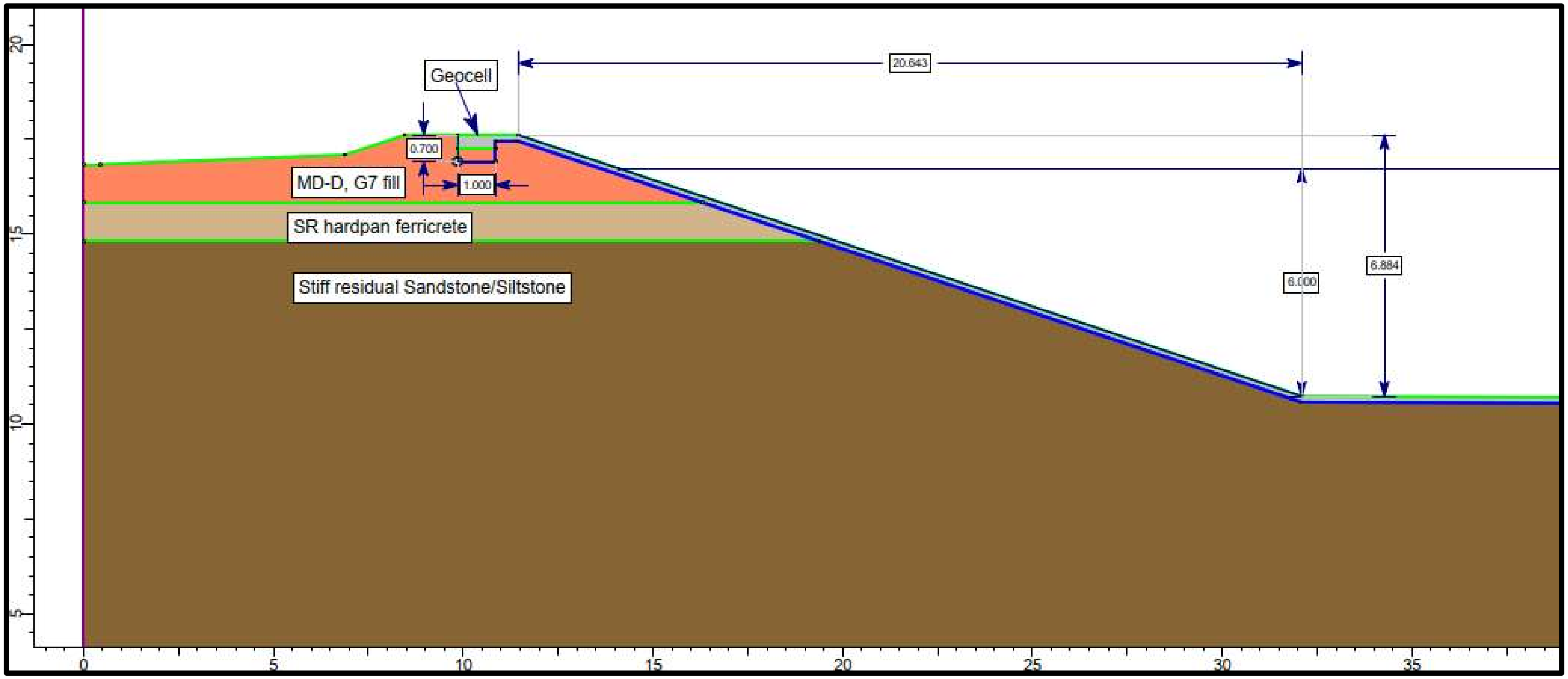


Figure 22: Brine Pond FE model geometry

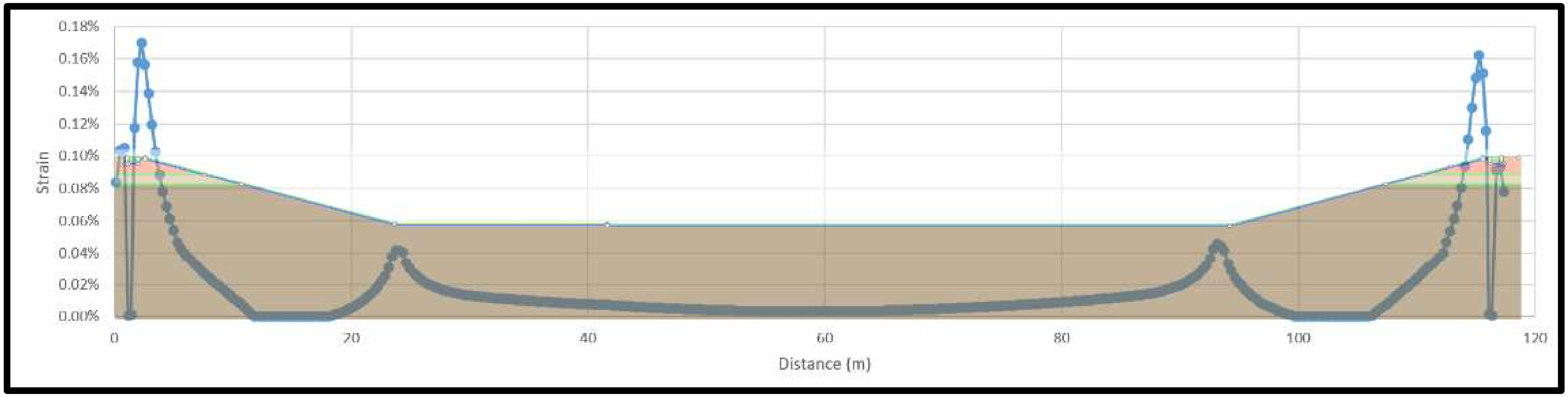


Figure 23: Liner strain assessment result

8.3.4 Slope Stability Assessment

The previously referenced test pits (Table 9) were not excavated within the pond's footprint due to the presence of an existing stockpile on site. However, a portion of this stockpile will be removed to facilitate the construction of the brine pond, which may result in in-situ conditions differing from the test pit profiles. To address this uncertainty, the entire slope, including the material beneath the basin, was conservatively modeled using G7 fill material, which is likely to have lower strength than the original in-situ material. Additionally, the geocell material and liner system were excluded and replaced with G7 material to evaluate the global stability of the pond slope in an unconstrained state.

Despite these conservative measures, the analysis (illustrated in Figure 24) demonstrates a Factor of Safety of 2.17 for the overall stability of the pond slope.

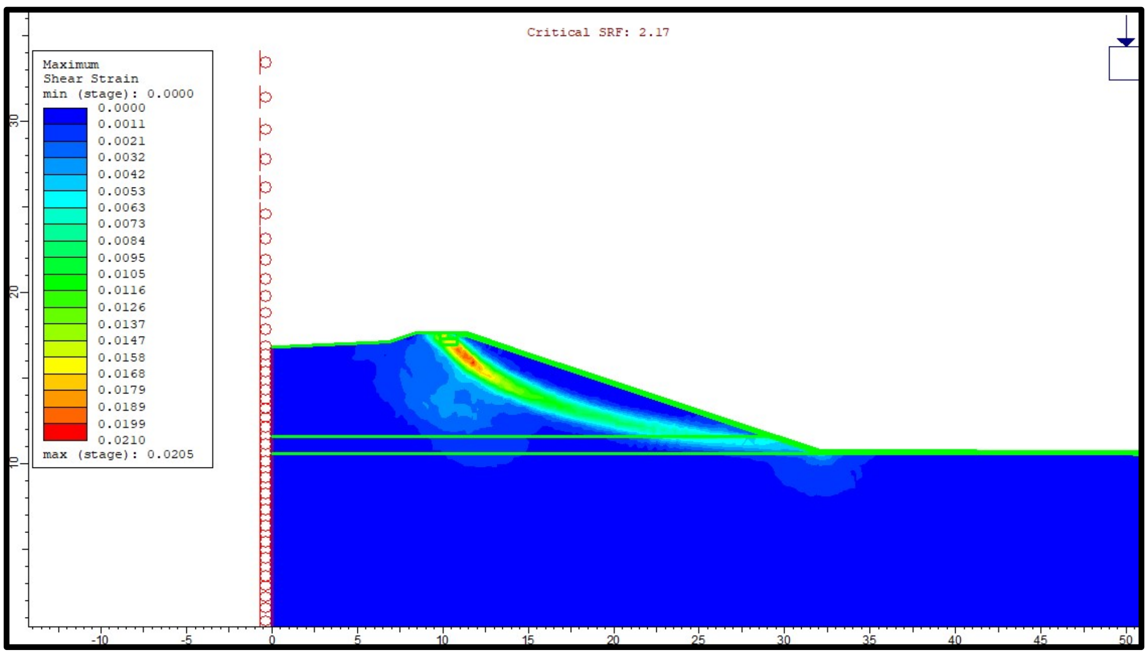


Figure 24: Global slope stability FoS result

8.3.5 General conclusions

The analysis indicates that strain levels within the liner remain well below the allowable limit of 3%, and the global stability of the side slopes exceeds the required safety margins. However, the following considerations should be noted:

- Published literature indicates that the interface strength between a smooth HDPE liner and Bidim (or similar geosynthetics) is slightly above 9. This has been further validated by previous interface shear testing (refer to **Annexure B**). As a result, when placed on a slope with an inclination angle greater than 9°, the interface may experience yielding. Therefore, the material placed over this interface—in this case, the geocell filled with cement-stabilized soil—must possess adequate strength and stiffness to stay in place without depending on the frictional strength of the underlying interface.

- The FE model confirms that this requirement is met, with both the interfaces above and below the liner yielding.
- The anchor trench construction, including the filling process, should be completed before filling the geocells. This will ensure that the load placed on the liner during construction is transferred up the slope to the anchor trench, preventing slippage along the interfaces above and below the HDPE liners.
- The geocell system should be constructed in stages from the toe of the slope upwards, allowing time between stages for the cement-stabilised material to gain sufficient strength to support the subsequent stage.
- It is assumed that the installed underdrainage system will perform adequately and that no positive pore water pressures will develop beneath the liner system.

8.3.6 Protective Cover System Stability Assessment

In order to ensure that the barrier protective cover system remains stable in the long-term under all conditions, it is necessary to confirm the veneer stability of the cover system. The barrier cover system comprises 150mm deep HDPE geocells filled with 6% cement stabilised soil on a 1000 g/m2 nonwoven needle-punched polyester geotextile.

The cover veneer stability and anchor trench design are based on the design method by (Qian, Koerner, & Gray, 2002). The tensile strength of the protection geotextile at less than 10% strain is used to restrain the cover system.

The slope veneer analysis is based on calculating the Factor of Safety (FoS) against the cover system sliding down the slope. The FoS is therefore defined as:

$$FOS = \frac{Stabilising\ forces\ of\ the\ cover\ system}{Mobilising\ forces\ acting\ on\ the\ cover\ system}$$

For long-term static conditions, the FoS against sliding > 1.5.

Table 10 gives the parameters used in the cover veneer stability analysis for the side slopes.

Table 10: Veneer slope stability assessment results

Parameter	Value	Reference / Comments
Unit weight of soilcrete	20 kN/m3	Literature
Friction angle of cover soil	40º	G7 sandy soil with cement
Cohesion of cover soil	0 kpa	Mabu Report 2021
Cover soil thickness	0.15 m	Design thickness. Analyse for 0.2 m to allow for overfilling and worst case scenario.
Soil slope angle	18,43°	Design slope is 1:3 (V:H)
Length of slope	20,7m	Longest slope at deepest end
Interface friction angle between GTX and smooth HDPE GM	10°	Based on literature and previous interface shear testing, residual - conservative
Tensile strength of geosynthetic	23 kN/m	Tensile strength of Bidim A10 at 10% strain (**Refer to Annexure B**)

Using the parameters described above, the factor of safety (FoS) against sliding of the geocells cover system down the slope is **5,5.**

The factor of safety against pull-out for the anchor trench is **1,99.**

The stability of the geocell cover system on the side slopes has therefore been confirmed. This has also taken account of construction equipment working on the slope (4T Dozer).

Refer to **Annexure A** for detailed calculation sheet of the Veneer slope stability as well as the stability assessment of the cover material and Anchor Trench.

8.4 LEAKAGE THROUGH COMPOSITE LINERS

8.4.1 General

The primary objective of the liner design is to eliminate the potential risk of contamination of the subgrade and water resources through the ingress of contaminants to the groundwater.

Zero leakage through a composite liner, due to the absence of holes in the liner, is a desirable target. However, various studies have shown it’s unrealistic to ensure no leakage because of damaged to the liners during installation. Therefore, in order to evaluate the risk of groundwater contamination, the possible leakage of the liner system should be acknowledged during the implementation of the system. The magnitude of the leakage is mitigated through sound design, high quality manufacturing and installation, and strong construction quality assurance on site

In the case of multiple layer geomembrane liner systems, the leakage is managed by draining it through the leakage drainage layer (cuspated HDPE sheet) and removing it to minimise the hydraulic head on the underlying geomembrane. The removed leakage will then be pumped back into the brine pond.

8.4.2 ACTION LEAKAGE RATE: PRIMARY LEAKAGE :

The brine pond is equipped with a primary leakage drainage and management system located between the primary and secondary geomembrane liners.

The Action Leakage Rate (ALR) for the primary leakage detection system of the brine pond must be estimated. The ALR is defined as the threshold leakage rate at which corrective actions are required to identify the cause of the leakage and implement remedial measures to reduce it below the established ALR. In the absence of specific South African legislation on the quantification of ALRs, it is recommended that the ALR be developed using the liner performance method.

The ALR is determined based on the expected leakage through the geomembrane liner, assuming a certain number of holes per hectare, each with a 2 mm diameter. Additionally, the ALR should not exceed the flow capacity of the leakage management system, which includes the leakage drainage layer and outlet pipe(s), with a safety factor of at least 1.5 applied.

The liner performance method, developed by Giroud and Bonaparte (1989), relies on the assumption of good installation and operational maintenance. Their research, along with later work by the U.S. EPA (1992a), found that a geomembrane-lined facility with proper construction quality assurance typically exhibits around two 2 mm diameter holes per hectare. This defect rate is considered acceptable, as facilities with high-quality construction have demonstrated virtually no leakage (EPA, 1992a; Bonaparte and Gross, 1990).

The above quality based action leakage (QBAL) rate method was supported by Jordan and Ruhl (2017). For excellent installation quality, they recommended a defect density of up to 2.5 defects per hectare. The ARL is calculated using the Giroud and Bonaparte formula:

$$ARL = N\ x\ C_B\ x\ a\ x\ \sqrt{2gxh_w}$$

Where:

N = No of defects per hectare;

C_B = dimensionless coefficient of 0.6;

a = area of geomembrane defect;

g = acceleration due to gravity (9.81 m/s2);

hw = head of water on the defect.

Using the above methodology, the ALRs for the primary leakage monitoring systems for the Brine Pond have been calculated for various water depths and is summarised in Table 11. The calculations are included in **Appendix C**.

Table 11 : Action Leakage Rates for the Brine Pond (Primary Leakage)

Water Depth (m)	Wetted Area (ha)	ALR Primary (l/ha/day)	ALR Primary (l/day)
1.47	1.21	2187	2647
2.47	1.36	2835	3856
3.57	1.52	3409	5181
4.57	1.68	3857	6479
5.87	1.90	4371	8305

8.4.3 PRIMARY LEAKAGE MANAGEMENT SYSTEM CAPACITY ASSESSMENT

Geocomposite Drainage System:

A geocomposite drainage core with a cuspated top (minimum 6 mm deep) will be installed between the primary and secondary liners. This drainage composite is specifically designed to function as a leak detection layer. In the event of leakage through the primary liner, the drainage geocomposite layer will intercept the leak and direct it to a single leachate collection drain, which then channels the water towards a collection sump. From the sump, the water can be pumped back into the brine pond. The primary purpose of this drainage layer is to reduce the hydrostatic pressure on the secondary liner, thereby minimising the risk of contaminants entering the groundwater.

The in plane flow capacity of the geocomposite drainage layer is typically 0.00038 m³/s/m for a Hydraulic Gradient of 0.1.

The Brine Pond has longitudinal slope of 1:200 (0.5%) will have a H.G.L of 0.005.

The in-plane flow capacity of the geocomposite drainage layer is typically 0.00038 m³/s/m for a hydraulic gradient (H.G.L) of 0.1. Given that the Brine Pond has a longitudinal slope of 1:200 (0.5%), the corresponding H.G.L is 0.005.

Calculation of the drainage system capacity at the base of the Brine Pond:

Q_{new} = 0.00038 x (0.005) = 1.9 x 10^{-5} m³/s/m.

Calculation of the flow capacity for the full width of the Brine Pond (76 m):

Q_{new} = 1.9 x 10^{-5} m³/s/m x 76 m = 0.0014 m³/s = 1.4 l/s = **120 960 l/day** > ALR at F.S.L with **F.O.S = 1.5**

Slotted HDPE pipe:

The geocomposite drainage layer will direct leakage to a liner drain trench with a trapezoidal profile, filled with 19 mm washed stone and equipped with a slotted HDPE pipe (refer to Figure 20). This pipe will convey the intercepted leakage to a collection sump designed for leakage monitoring, as discussed in the following section.

The capacity of the HDPE pipe is evaluated against the maximum flow capacity of the geocomposite drainage layer. Based on the Colebrook-White equation, the pipe's capacity at a 0.5% slope is approximately **6.5 l/s** (refer to Figure 25), which exceeds both the maximum flow rate of the geocomposite drainage layer and the established Action Leakage Rate (ALR).

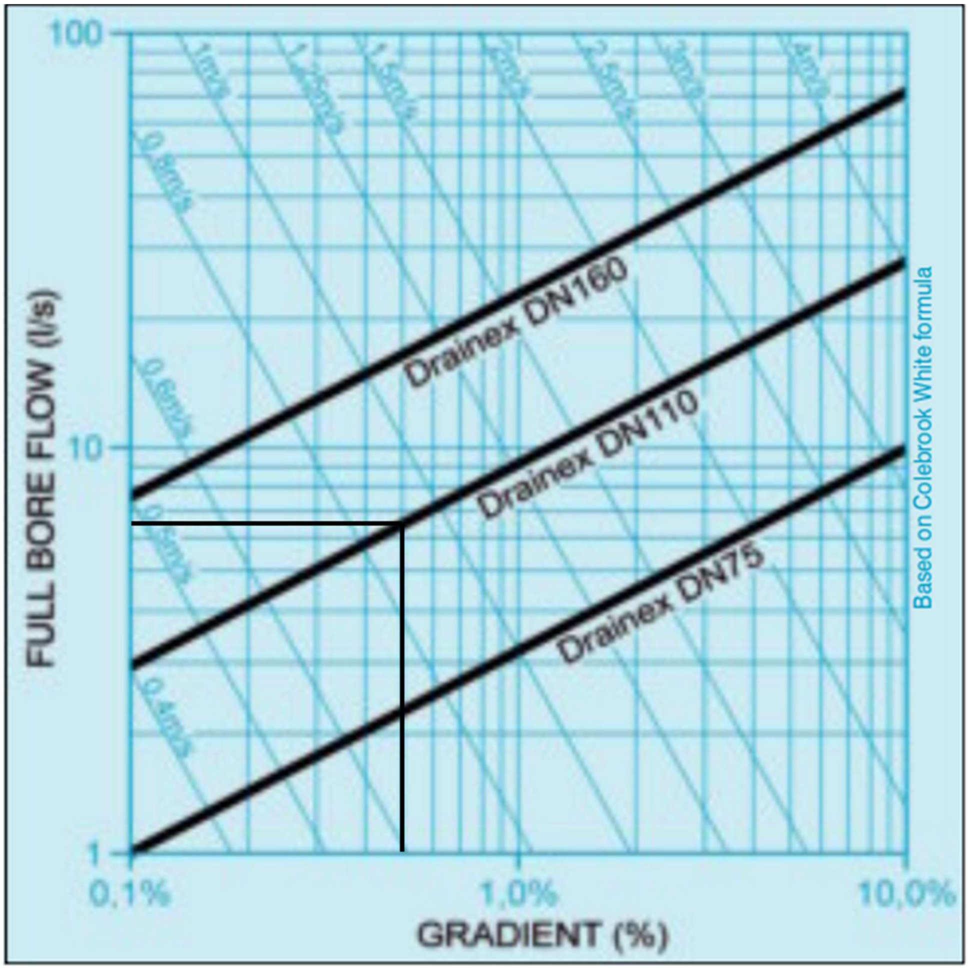


Figure 25 : Capacity of Leakage detection outlet pipe (HPDE pipe)

8.4.4 Leakage Monitoring

Any leakage in the primary geomembrane liner will result in the accumulation of water in the respective leakage sumps, which will subsequently cause the water level to rise in the monitoring sumps. To determine the leakage rate, the following procedure should be followed:

- Insert a submersible pump with a known delivery rate into the sump.
- Use a borehole dip meter probe to measure the initial water level in the monitoring pipe.
- Start the pump and allow it to run until it is completely dry. Measure the total pumping time and calculate the volume of water pumped out.
- Once the pump is stopped, measure the time it takes for the leakage water to rise to the same level in the monitoring pipe as was measured before pumping.
- Calculate the leakage rate using the formula:

- Leakage rate = Volume/Time
- Compare the calculated leakage rate with the allowable leakage rate (ALR) to assess compliance.

8.4.5 Secondary Liner Leakage Calculation

The secondary liner, positioned beneath the liner drain, is a composite system comprising a 1.5 mm geomembrane (GM) and a Geosynthetic Clay Liner (GCL).

Rowe has shown that assuming direct contact between the GM and underlying GCL liner significantly underestimate the actual leakage in a typical composite liner configuration. Rowe (2005) postulated that the reason for the discrepancy was that the GM's and GCL are generally not in direct contact, but rather there are wrinkles that, if coinciding with a hole (puncture), would substantially increase the leakage rate. As a result, Rowe (2012) developed an overview of the factors to be taken into account when determining the leakage rate through composite liners as set out in the following list:

- Hydraulic head on the liner (h_d);
- The length of a connected wrinkle (L) of the HDPE GM layer due to thermal expansion;
- The half-width of and wrinkle (b) of the HDPE GM layer due to thermal expansion ;
- The hydraulic conductivity of the GCL liner (k);
- The thickness of the GCL liner (D);
- The transmissivity of the interface between the GM and GCL liners (θ);

The relationship between the leakage rate and the above parameters is calculated with the following equation (Rowe 2012):

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

The leakage rate through the secondary liner system of the Brine Pond were calculated with the material properties and most likely values of the wrinkle length (L) and width (2b) as described in the technical paper published by Rowe titled:

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

.

Leakage is calculated through the secondary liner that consists of a 1.5mm HDPE geomembrane and a 6.2 mm thick GCL Liner will be calculated assuming poor QA/QC control during installation, the leakage through the liner is calculated as follows:

Table 12: Brine Pond Secondary Liner Calculations

Parameter	Value	Units	Comment
Footprint Area	2.14	ha	*Liner Footprint Area (Total Area of Pond including Embankments)*
L	188	m	*Assume Worst Case (Longitudinal Length of Brine Pond)*
k	2.6 x 10^{-11}	m/s	*Permeability of GCL liner*
b	0.15	m	Wrinkle Half-Width
D	0.006	m	*Thickness of GCL*
θ	2 x 10^{-12}	m²/s	*Transmissivity of GM-GCL interface (Rowe 2021)*
h_d	0.005	m	*Typical Headwater above secondary liner (Depth of Drainage layer)*
Q	1.39 x 10-9	m³/s	
Leakage Rate	8.61	Per Wrinkle per Day	
Wrinkles/ha	5	No	
Total Leakage	68.93	l/ha/day	

The U.S. Environmental Protection Agency (EPA) recommends an Action Leakage Rate (ALR) range of 50–200 liters per hectare per day (applicable for the secondary liner system for this analysis). Even under the conservative assumption of poor QA/QC practices during the construction of the secondary liner (as detailed in Table 9), the anticipated total leakage remains below the EPA guideline. This is primarily attributed to the drainage layer, which effectively reduces the hydrostatic head on the secondary liner, thereby mitigating the risk of contaminants reaching the groundwater.

Nonetheless, strict QA/QC measures must be enforced during the installation of the secondary liner to minimize the occurrence of defects such as holes and wrinkles, ensuring the liner's integrity and optimal performance.

8.5 SUB-SOIL SYSTEM AND GROUND WATER DRAINAGE

A subsoil drainage system has been provided below the liner systems of the brine pond. As per Regulation GN R636 of 2013, the sizing of sub-soil drainage pipes should guarantee atmospheric pressure within the pipes. The regulation states, all drainage layers must contain drainage pipes of adequate size, spacing, and strength to ensure atmospheric pressure within the drainage application for the service life of the landfill.

The design of the subsoil drainage system of the brine pond is based on the herringbone sub-surface drainage system as described in SANRAL's Road Drainage Manual (5th edition). A typical herringbone sub-surface drainage system is illustrated in Figure 26. The drainage pipes are designed to flow 70% of full capacity. The equation used for the design of the herringbone sub-surface drainage system is listed below:

$$A = \frac{(26.92 \times 10^6)\, d^{\frac{8}{3}}\, S_o^{\frac{1}{2}}}{n\, q}\ (0.7)$$

Where:

- S_o = spacing (m);
- A = surface area (m2) = S (L+0.5 S);
- d = diameter of pipe (m);
- L = length of the pipe (m);
- q = drainage rate (mm/day);
- n = Manning's n ($s/m^{1/3}$); and
- S_o = slope of the pipe m/m.

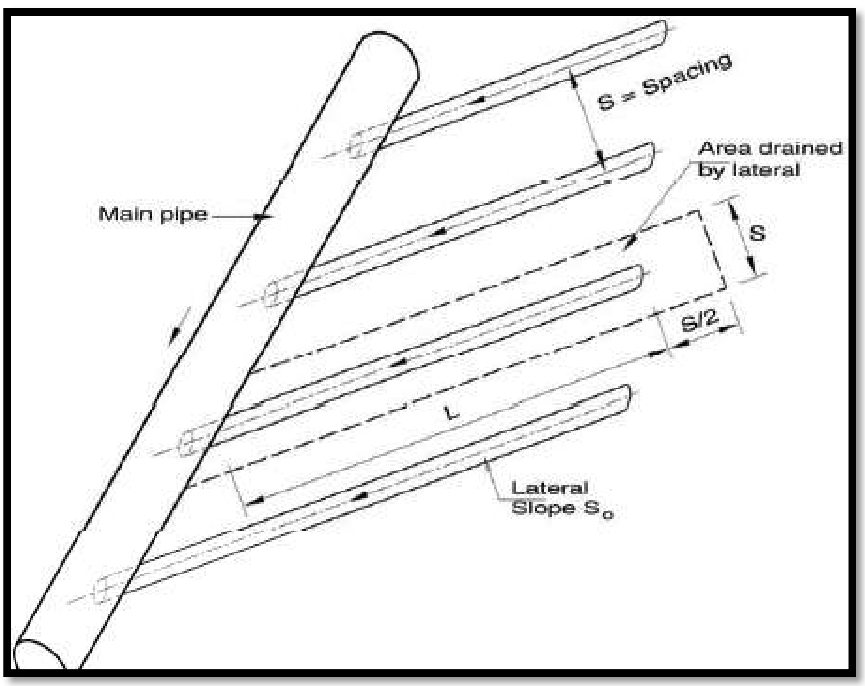


Figure 26: Typical Herringbone Sub-Surface Drainage System (SANRAL, 2007:9-18)

The test pits excavated for this project did not indicate the presence of groundwater; however, the brine pond will extend deeper than the test pit depths. To address this, data from various geotechnical investigations and groundwater assessments conducted by multiple consultants during the development of the Belfast mine were utilised. These findings informed the sizing of the sub-soil drainage system to ensure it maintains atmospheric pressure and effectively manages potential groundwater inflow.

Groundwater data used in the design of the sub-soil drainage system are summarised in Chapter 04 (Geo-Sciences) of the Prefeasibility Study report. According to the report, Karoo-type aquifers typically yield between 0.01 and 2 liters per second. Accordingly, the sub-soil drainage system has been designed to accommodate a maximum flow rate of 2 liters per second.

The calculated capacity of the sub-soil drainage system is presented in Table 13 and compared to the design flow rate.

Table 13: Brine Pond Sub-Soil Drainage calculation

Variable	Description	Unit	110mm Slotted Pipes
S	Spacing	m	15
A	Surface Area	m^2	1703
d	Diameter of pipe	m	0,11
L	Length of Pipe	m	106
q	Drainage rate	mm/day	10.06
q	Drainage rate	mm/h	242
Number	Number of Laterals	No.	9
Q	Calculated flow	m^3/s	3701
Q	Calculated flow	l/s	42.8
n	Manning n	$s/m^{1/3}$	0.009
So	Slope of pipe	m/m	0.005

The design flow rate of 2 liters per second is significantly below the system's capacity, as detailed in Table 13, ensuring the drainage system is adequately sized for anticipated conditions.

8.6 SERVICE LIFE OF THE COMPOSITE LINER

8.6.1 HDPE GEOMEMBRANE

A liner system has a finite service life and is dependent on a variety of factors specific to its application. It's required to compare the service life of the composite liner system should be to the anticipated polluting period of the mine.

A literature review has been done to demonstrate the longevity of the proposed liner systems for the applications of the Brine Pond.

The following white paper, published by the Geosynthetic Research Institute (GRI), is used as the main technical reference for this literature review:

GRI White Paper #6, 8 February 2011 - Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions.

Degradation of an HDPE geomembrane is typically caused by the following mechanisms:

- Oxidation
- UV radiation – applicable to exposed geomembranes

- Aggressive chemical solvents
- High-temperature exposure

Degradation decreases the physical properties of the geomembrane i.e. the Stress Crack Resistance (SCR) and limiting the exposure of the geomembrane to sunlight (heat) is the most effective way to maximize the service life. (Rowe, 2012)

The service life of a geomembrane is defined by three stages (Koerner et al, 2005):

- Stage A - Antioxidation Depletion Time;
- Stage B – Induction time to onset of degradation;
- Stage C – Time to reach 50% degradation (half-life) - ;

The industry standard of when the service life of the geomembrane has been reached is the point when the physical properties (i.e. SCR) has decreased to 50 % of its original value (Rowe, 2012). Table 2 of GRI White paper 6 will be used for the service life prediction of the geomembrane liner (both primary and secondary) for the Brin Pond.

Figure 27 presents GSI's best estimate of the service life for covered 1.5 mm thick HDPE geomembrane installations (manufactured according to the current GRI-GM13 specification). The results assume that the geomembrane will not be subjected to any significant tensile stresses and that it is covered by an adequate protection layer, which is comparable to our project.

Project Site Conditions:

- The liner applications for the brine pond will contain brine discharge from the WTP are not exposed to site leachate or other water that can contribute to chemical degradation of the geomembrane over its service life. This implies that the conditions to which the liners were exposed during the tests are more severe than the actual project conditions;
- The service life of facility and polluting period is 20 years since it is assumed that all infrastructure (including liners) will be removed and rehabilitated;
- Oxidation is the primary degradation mechanism, which is promoted by UV radiation (pegs, 2003). The liner applications for the Brine Pond will be covered by a geocells layer filled with cement stabilizes cement and will be directly overlayed by a protection geotextile, creating optimal conditions to maximise the service life of the geomembrane;
- The average monthly maximum temperature (February) is 23.4 °C. It is assumed that the temperature that the lining system will be exposed to will be considerably less due to the cover materials as well it will be constantly filled with a brine-water solution.

  The minimum service life of the brine pond lining systems is expected thereof to be in the order of 265 years which is well above the required service life of the brine pond.

Table 2 - Lifetime prediction of HDPE (nonexposed) at various field temperatures

In Service Temperature (°C)	Stage "A" (years)			Stage "B" (years)	Stage "C" (years)	Total Prediction* (years)
	Standard OIT	High Press. OIT	Average OIT			
20	200	215	208	30	208	446
25	135	144	140	25	100	265
30	95	98	97	20	49	166
35	65	67	66	15	25	106
40	45	47	46	10	13	69

*Total = Stage A (average) + Stage B + Stage C

Figure 27: Lifetime Prediction of HDPE (non-exposed) GM (GRI, 2011)

8.6.2 GCL AND GEOTEXTILE – SERVICE LIFE DETERMINATION

For the reasons explained in the previous section, the HDPE geocells is expected to have the same service life and durability as the HDPE geomembrane.

Kaytech has recently conducted extensive durability testing on its GCL component geotextiles as well as Bidim geotextiles. As long as the geotextiles are covered and not subject to ultraviolet radiation, the service life of the various geotextiles exceeds 100 years at 25°C as shown in Figure 28.

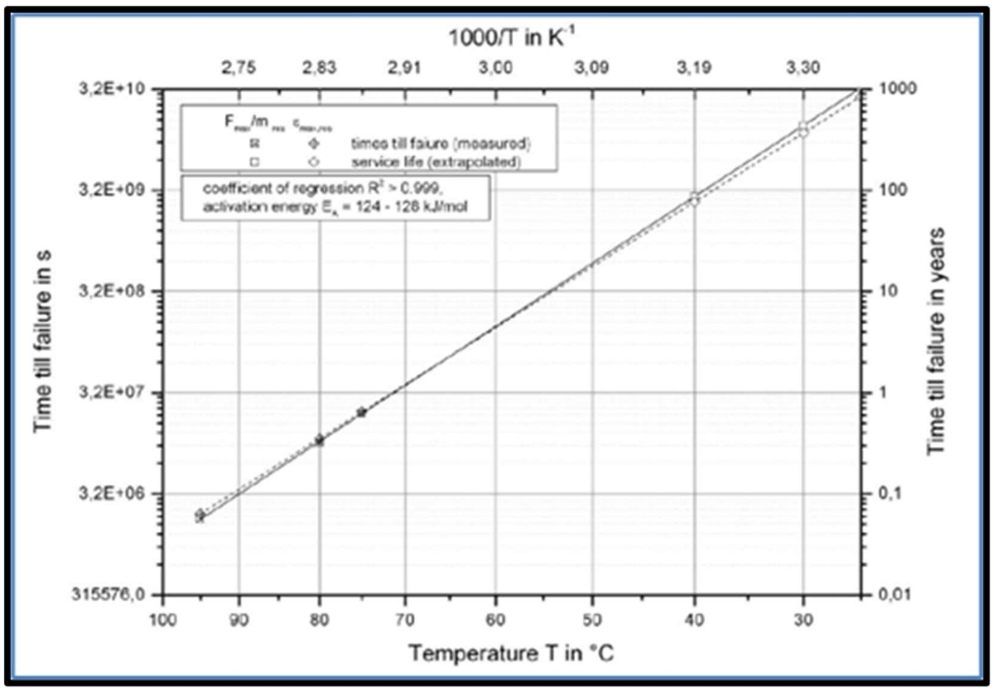


Figure 28: Arrhenius extrapolation to service temperature, based on residual maximal tensile force in relation to specimen's mass of 50 % and residual elongation at max tensile force of 50 % (Kaytech Technical Note, 2016)

8.7 GCL CHEMICAL COMPATIBILITY

As mentioned in section 8.2.1.4 of the report, a GCL is proposed to form part of the composite Class C liner system instead of the 300 mm Compacted Clay Liner (CCL). The economic considerations and the limited available competent clay material on site are the main motivating factors for the implementation of a GCL.

The possibility of a defect/hole in the secondary geomembrane causing leachate to percolate down to and through the GCL could cause the cations contained inside the brine to exchange with the sodium cation in the GCL’s bentonite. This could result in reduced swelling capacity and increased hydraulic conductivity of the GCL.

BVi appointed GMJ consulting to perform a chemical compatibility assessment of a representative brine sample from an existing water treatment facility from the Mafube Mine. The sample was sent to Waterlab (Pty) Ltd for a full chemical analysis, The water chemistry of the solute from the Mafube facility are expected to be similar to the leachate expected for this project and will therefore be referenced for motivating the compatibility of the GCL for the included in this liner system.

The following list sets out the main findings of their report and the full chemical compatibility report is attached to **Annexure D:**

- The GCL's sodium bentonite component is at risk of cation exchange when exposed to moderate Relative Montmorillonite Deficiencies (RMDs) and moderate ionic strengths in the Byan sample leachates. However, the RMD values are above the critical threshold of 0.07, mitigating the immediate risk.
- The high ionic strength (IS) of the leachates, correlated with elevated Total Dissolved Solids (TDS) and Electrical Conductivity (EC), poses a potential challenge to the GCL's performance.
- Hydraulic conductivity of the GCL is moderately low but higher by one to two orders of magnitude compared to laboratory values obtained with deionized water.
- The unavailability of suitable low-permeability clay ($<10^{-9}$ cm/s) for use as a secondary barrier emphasizes the importance of alternative measures like pre-hydration of the GCL.
- Swell Index testing is necessary to confirm compliance with the GRI-GCL3 standard for bentonite properties in the presence of the Byan leachate.

The following recommendations are made in the report:

- Pre-Hydration: Pre-hydrate the GCL before installation to mitigate the effects of cation exchange and improve its resistance to high ionic strength leachates.
- Surface Preparation: Ensure the GCL is placed on a smooth, inert sub-grade to maintain its integrity and prevent damage.
- Hydraulic Conductivity Testing: Conduct flexible wall permeameter tests to validate the hydraulic performance of the GCL under site-specific conditions and assess the practicality of pre-hydration methods.

- Swell Index Testing: Perform Swell Index testing with the Byan leachate to verify that the minimum requirement of 24 ml/2g is met, as per GRI-GCL3 standards.
- Precautionary Placement: Implement proper precautions during placement and ensure boundary conditions (sub-grade quality, moisture levels, and potential contamination) are managed effectively to optimise the GCL's performance.

8.8 EQUIVALENCY ASSESSMENT OF GCL'S TO CCL'S

The paper presented by R.M Koerner and D.E. Daniel at the 7th GRI seminar in 1993 titled "Technical Equivalency Assessment of GCL's and CCL's" contains a comprehensive comparison of GCL's and CCL's. The following performance parameters were mainly discussed:

- **Chemical Adsorption Capacity:**

  Adsorption is controlled by cation exchange reactions (Koerner et al., 1993) which were discussed in section 8.7 of the report.

- **Physical/Mechanical Performance:**

  Koerner et al. (1993) assessed the structural performance of a GCL and CCL against several mechanisms that could lead to poor hydraulic performance. They've shown that a GCL performed equivalent to a CCL against the temperature effects during freeze-thaw events and wet-dry cycles.

  The GCL has shown superior performance against the effects of differential settlement which is an important component for the MRF liner application.

  GCL are not equivalent to CCL's with regard to bearing capacity and should therefore be adequately protected against traffic loading during construction and operation of the facility.

- **Quality Assurance and Constructability:**

  The constructability of a GCL liner is generally considered to be better than a CCL given the demanding quality assurance tests associated with the construction of a low-permeability CCL. The time and cost of constructing a CCL liner, especially in the absence of competent clay material available on site, is substantially more than a GCL.

  GCL's are however more susceptible to puncture during construction than CCL's and should be addressed with careful QC/QA procedures.

- **Hydraulic performance:**

  Koerner et al. (1993) have shown that the estimated hydraulic conductivity for equivalency could be assessed if it is assumed that the water flux through a GCL and CCL is similar. If we consider only the GCL and CCL individually (not part of a composite liner) Darcy's law can be used as follows:

$$(K_{GCL})_{Required}\left(\frac{H+T_{GCL}}{T_{GCL}}\right)=K_{CCL}\left(\frac{H+T_{CCL}}{T_{CCL}}\right)$$

Where:

$(K_{GCL})_{Required}$ = Required hydraulic conductivity of GCL Liner for equivalence (m/s);

(K_{CCL}) = Hydraulic conductivity of CCL (m/s)

H = Head of water assumed (m)

T = GCL thickness (mm)

T_{CCL} = CCL thickness (mm)

The hydraulic conductivity and thickness of the CCL are known i.e. k = $1x10^{-9}$ m/s and T_{CCL} = 600mm. The hydrated thickness of the GCL is typically 6.2 mm. If we assume a headwater depth of 6 mm (maximum depth of geocomposite drainage layer), the required hydraulic conductivity of the GCL is calculated as 5.13×10^{-10} m/s. Commercially produced GCL's have a hydraulic conductivity typically ≤ 1 – 5 $x10^{-11}$ m/s and therefore has at least an equivalent ability to a CCL to contain liquids.

However, Koener et al. (1993) and Rowe (2012) have shown that the transmissivity (θ) of the interface between the Geomembrane (GM) and clay liner (GCL or CCL) controls the leakage through a composite liner rather than the hydraulic conductivity of the GCL or CCL. Their experimental data show that GM/GCL composite liners have a lower interface transmissivity than GM/CCL composite liners.

9. GUIDELINE FOR THE DECOMMISSIONING OF BOREHOLE BBH05 ON A MINE SITE

9.1 INTRODUCTION

BBH05 is located within the footprint of a new brine pond (Refer to Figure 29), making its closure necessary to facilitate the construction of the Brine Pond. This section outlines the approach for the compliant decommissioning of Borehole BBH05 at the Belfast Mine. The closure process aligns with SANS 10299-9:2003 to ensure compliance with groundwater protection standards and mining regulations.

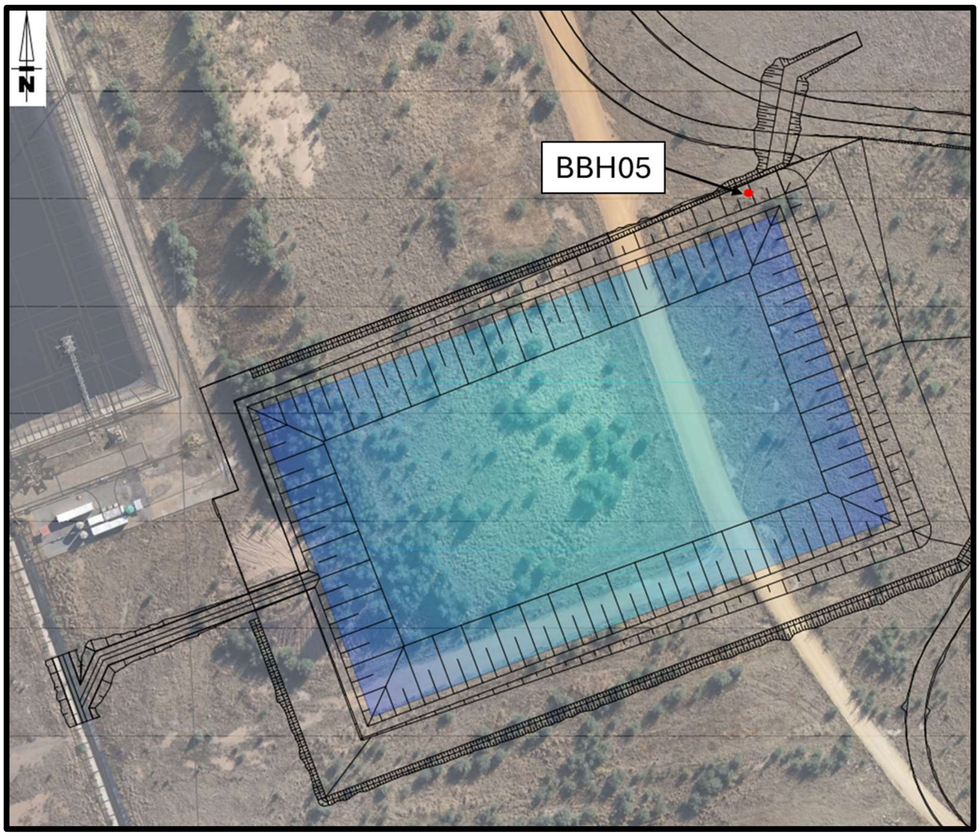


Figure 29: Borehole BBH05 Location

9.2 JUSTIFICATION FOR DECOMMISSIONING BBH05

BBH05 must be decommissioned as it is situated within the footprint of the planned brine pond. The decommissioning is required to:

- Prevent the borehole from becoming a pathway for groundwater contamination.
- Ensure it does not act as a conduit between different aquifers.
- Comply with site rehabilitation requirements as part of mine closure planning.

- Mitigate any safety risks associated with open or abandoned boreholes.

9.3 DECOMMISSIONING REQUIREMENTS

Decommissioning shall be conducted under the supervision of a competent person or a professional geohydrologist to ensure:

- No contamination risk: The borehole must be sealed to prevent surface pollutants from reaching the aquifer;
- No inter-aquifer flow: Since BBH05 penetrates multiple geological layers, proper sealing is required;
- Artesian control: If BBH05 has any pressure head, sealing must prevent uncontrolled groundwater discharge;
- Surface rehabilitation: The site must be restored to its original condition after closure.

9.4 DECOMMISSIONING METHODS

The following steps will be followed for the permanent closure of Borehole BBH05:

9.4.1 Backfilling

- Backfill Placement and Compaction Method:
- **Layered Placement:** Backfill material shall be placed in controlled layers, typically 1 to 3 metres at a time, to prevent bridging and void formation.
- **Gravity Settling**: Bentonite – based materials of fine gravel will be allowed to naturally settle under gravity to ensure uniform distribution within the borehole.
- **Hydration for Compaction**: If necessary, slight hydration of backfill material may be introduced to aid in compaction and prevent excessive settlement over time.
- **Avoiding Air Pockets**: Material shall be poured in a steady and controlled manner to minimise trapped air pockets, which can lead to instability.

9.4.2 Grouting

- To prevent contamination, cement-bentonite grout shall be used in sections where inter-aquifer flow risks exist.
- Grout must be placed using a tremie pipe to ensure even distribution.

9.4.3 Capping and Surface Sealing

- The top 1 metre of BBH05 shall be sealed with a concrete plug.

- The upper casing must be removed or cut at least 1 metre below ground level.
- A final 0.5-metre-thick concrete cap shall be placed to complete the closure.

9.5 SURFACE REHABILITATION

- The area around BBH05 shall be backfilled with native soil and compacted.
- If necessary, vegetation will be re-established to match surrounding conditions.
- Any surface structures related to BBH05 must be dismantled and removed.

9.6 DOCUMENTATION & COMPLIANCE REPORTING

A decommissioning report must be prepared and submitted, including:

- Borehole specifications: Depth, casing type, water level.
- Closure method: Materials used for sealing and capping.
- Photo documentation: Before, during, and after decommissioning.
- Regulatory compliance: Submission of the report to the mine's environmental team and regulatory authorities.

9.7 RESPONSIBILITIES

- Mine Environmental Team: Ensures closure aligns with SANS 10299-9 and site rehabilitation plans.
- Hydrogeologist / Competent Person: Oversees and certifies the decommissioning process.
- Mining Contractor: Carries out physical closure work under supervision.

9.8 CONCLUSION

The closure of BBH05 is a necessary step in ensuring groundwater protection, regulatory compliance, and site safety. Following this guideline will ensure proper decommissioning and rehabilitation of the borehole location in accordance with best mining practices.

10. COMPLIANCE WITH THE CRITERIA

The design for this scope of work has been carried out in compliance with the design criteria (BCX-000017-12968-ENG-DCR-0001) and by the requirements of the National Water Act (NWA), 1998 (Act No. 36 of 1998), along with the associated regulations, including GN 704 of 4 June 1999 and the National Norms and Standards for the Disposal of Waste to Landfill (GN R.636 in terms of Section 7(1)(c) of the NEMWA).

ANNEXURE A: VENEER SLOPE STABILITY

BELFAST WTP BRINE POND - GEOCELLS VENEER STABILITY

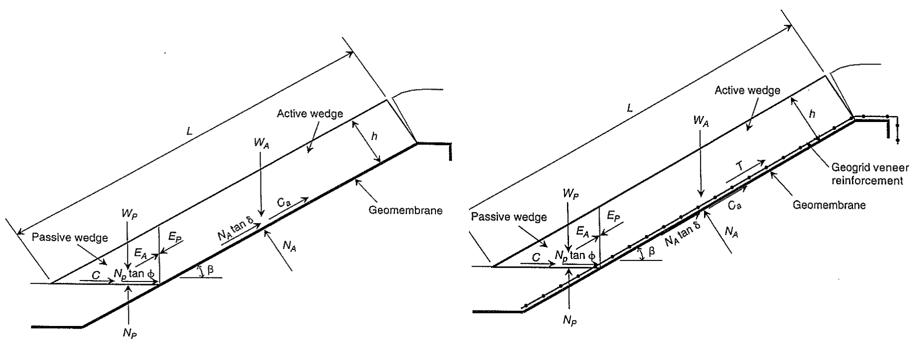
PROJECT IDENTIFICATION	
Title	Belfast WTP Brine Pond
Project number	35278
Client	Exxaro
Designed	FLR
Checked	
Approved	

DESCRIPTION
Veneer Reinforcement Calculations for Soilcrete (6%) filled 150mm geocells on 1:3 smooth liner

COMPANY'S INFORMATION	
Name	Exxaro Brine Pond
Address	
Telephone number	
Fax number	
Email	

Ref. Designing with Geosynthetics - 5th edition - Robert M Koerner 3.2.7 pag 380-383

Whenever a slope is covered with soil, a stability calculation should be made to assess the potential for sliding failure of the soil on the barrier layer.Four situations come to mind: landfill liners with leachate collection sand or gravel above them until such time that the solid waste acts as a passive resistance restraint; surface impoundment liners where the cover soil is placed over the geomembrane to shield it from ultraviolet light, heat degradation, and equipment damage; landfill covers that have topsoil and protection soil placed over the geomembrane; and general slopes and embankments containing geotextiles or erosion control materials being covered with a layer of soil. In all cases the soil layer is relatively thin (0.3 to 1.0 m), hence the sliding stability of such a veneer of cover soil is the issue.



(a) Without reinforcement (b) With the use of geogrid veneer reinforcement

WA = total weight of the active wedge,
WP = total weight of the passive wedge,
NA = effective force normal to the failure plane of the active wedge,
NP = effective force normal to the failure plane of the passive wedge,
γ = unit weight of the cover soil,
h = thickness of the cover soil,
L = length of slope measured along the geomembrane,
β = soil slope angle beneath the geomembrane,
φ = friction angle of the cover soil,
δ = interface friction angle between cover soil and geomembrane,
Ca = adhesive force between cover soil of the active wedge and the geomembrane,
ca = adhesion between cover soil of the active wedge and the geomembrane,
C = cohesive force along the failure plane of the passive wedge,
c = cohesion of the cover soil,
EA = interwedge force acting on the active wedge from the passive wedge,
EP = interwedge force acting on the passive wedge from the active wedge, and
FS = factor of safety against cover soil sliding on the geomembrane.

DESIGN INFORMATION			
SYMBOL	VALUE	UNIT	DESCRIPTION
γ	20	kN/m^3	unit weight of cover soil (6% soilcrete)
ϕ	40.00	deg	friction angle of the cover soil
c	0	kpa	cohesion of the cover soil
h	0.2	m	thickness of the cover soil (add 0,05m to allow for installation loading)
β	18.43	°	soil slope angle
L	20.68	m	length of the slope
c_a	0		adhesion between cover soil of the active wedge and geomembrane
δ	10	°	interface friction angle between GTX and smooth geomembrane

WITHOUT REINFORCEMENT			
SYMBOL	VALUE	UNIT	DESCRIPTION
Wa	80.3	kN/m^3	total weight of the active wedge
Na	76.2	kN	effective force normal to the failure plane of the active wedge
Ca	0.0	kN	adhesive force between cover soil and gsy
Wp	1.3	kN/m^3	total weight of the passive wedge
C	0.0	kN	cohesive force along the failure plane of the passive wedge
a	7.6		n/a
b	-6.5		n/a
c	1.1		n/a

13.43639338 kN

$FoS_{unreinf}$	**0.61**

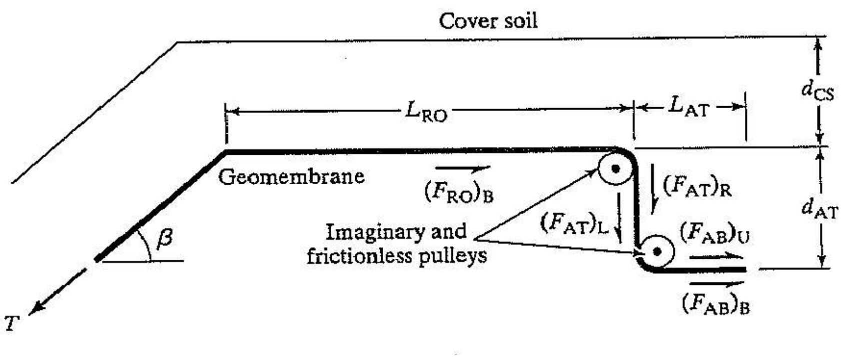
The slope is unsafe from sliding block failure, wtihout reinforcement. The Protection Geotextile doubles as a geosynthetic reinforcement for the slope, due to its high tensile strength.

Calculations below show the FOS against sliding failure, as well as anchor trench requirements accounting for the protection geotextiles anchorage needs to mobolise the necessary tension.

WITH REINFORCEMENT			
SYMBOL	VALUE	UNIT	DESCRIPTION
Tuts	50	kN/m	Ultimate tensile strengh of the geosynthetic (Bidim A10)
RF	2.2		Reduction factor (RF=RFcrxRFidxRFenvxFS)
Tall	22.73	kN/m	Allowable tensile strength
δ_{Reinf}	10.00	°	interface friction angle between cover reinforcement and underlying gsy
a	0.80		n/a
b	-4.61		n/a
c	1.13		n/a

FoS_{reinf}	**5.49**

BELFAST WTP BRINE POND - GEOCELLS VENEER STABILITY



Ref. Geotechnical Aspects of landfill design and construction - X. Qian, R.M. Koerner, D. H. Gray - par. 4.7.2

WORST CASE: 1:3 SLOPE, 21m LENGTH, Bidim A10 geotextile

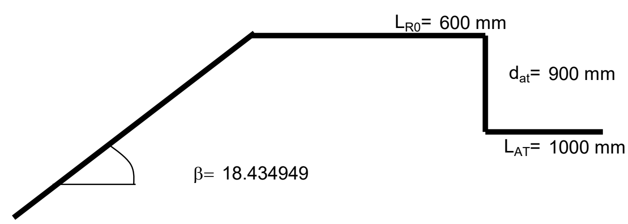
DESIGN INFORMATION

SYMBOL	VALUE	UNIT	DESCRIPTION
β	18.43	°	side slope angle
d_{cs}	0.15	m	thickness covering soil
γ_{soil}	20	kn/m^3	weight of soil
d_{at}	0.9	m	trench depth
L_{R0}	0.60	m	length of geosynthetic runout
L_{AT}	1.00	m	length of geosynthetic inside the trench
ϕ_{soil}	40	°	trench soil friction angle
δ_C	10	°	friction angle between the geosynthetics and the underlying soil
δ_F	20	°	friction angle between the geosynthetic and the backfill soil
Tuts	50	kN/m	Ultimate tensile strengh of the geosynthetic
RF	2.2		Reduction factor ($RF=RF_{cr}xRF_{id}xRF_{env}xFS$)
Tall	22.73	kN/m	Allowable tensile strength

RESULTS

SYMBOL	VALUE	UNIT	DESCRIPTION
$(F_{R0})_B$	3.27	KN/m	friction force beneath runout geosynthetics
$(F_{AT})_R$	1.40	KN/m	friction force between the right side of the geosynthetic and the side wall of anchor trench
$(F_{AT})_L$	0.68	KN/m	friction force between the left side of the geosynthetic and the side wall of anchor trench
$(F_{AB})_B$	3.33	KN/m	friction force between the right side of the geosynthetics and the underlying soil at the botton of anchor trench
$(F_{AB})_U$	6.88	KN/m	friction force between the right side of the geosynthetics and the overlying soil at the botton of anchor trench
T_{MAX}	9.34	KN/m	geosynthetic tensile force developed by the anchor trench
T_D	4.70	KN/m	geosynthetic design tensile force

FoS (T_{MAX}/T_D)	**1.99**



ANNEXURE B: INTERFACE SHEAR TESTING

GEOSYNTHETIC LABORATORY

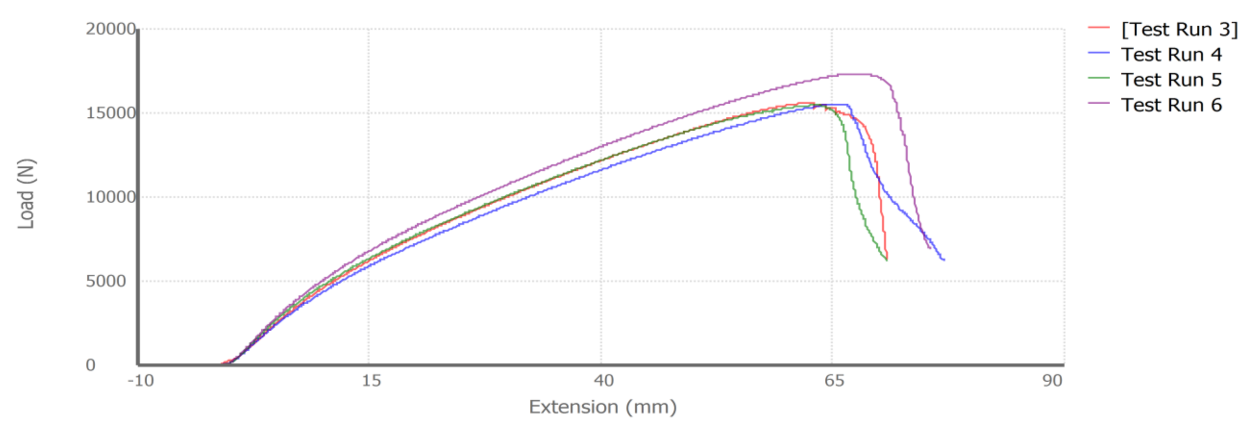
11 Livingstone Road
Pinetown, KZN
Tel: 031 717 2300
info@geolaboratory.com

TEST RESULTS

TEST STANDARD	WIDE WIDTH TENSILE	EN ISO 10319-15

Test Reference:	*Kaytech A10 Tensile P2017015*	*Issue No*	1
Client Details:	*Kaytech*		
Project:	*Conformance Testing*		

Sample Reference:	*Bidim A10*	Roll Number:	*n/a*
Sample Description:	*NW-N-CF-PET*	Dimensions:	*200x200*
Sampling Standard:	*ISO 9862*	Temp:	*23.000 °C*
Sampled By:	*NM*	Humidity:	*60.000 %*
Condition when Sampled:	*Good*	Pre-load	*20 N*
Grip Separation:	*100 mm*	Tested By:	*NM*
Rejected Specimens:	*2 due to slippage*	Date Received:	*NA*
Test Direction:	*MD- Parent sample*	Test Date:	*2017/08/01 12:24:56 PM*
Test Condition:	*Dry*	Test Speed:	*20 mm/min*



Specimen	Peak Load (kN)	Elongation at P	Peak (kN/m) (uni	F@2% (kN	F@5% (kN/m	F@10% (kN/m)
Test Run 3	15.538	62.317	77.688	4.710	12.183	22.524
Test Run 4	15.497	65.768	77.486	4.638	11.841	21.419
Test Run 5	15.427	63.349	77.135	5.186	13.098	23.418
Test Run 6	17.279	67.744	86.397	5.358	13.855	24.938
Mean	**15.935**	**64.794**	**79.677**	**4.973**	**12.744**	**23.075**
Minimum	15.427	62.317	77.135	4.638	11.841	21.419
Maximum	17.279	67.744	86.397	5.358	13.855	24.938
Standard Dev	0.897	2.441	4.486	0.354	0.911	1.487
% Coefficient	5.631	3.767	5.631	7.110	7.149	6.445

Authorised by
Technical Signatory

Date 2017/08/01 01:23:19 PM

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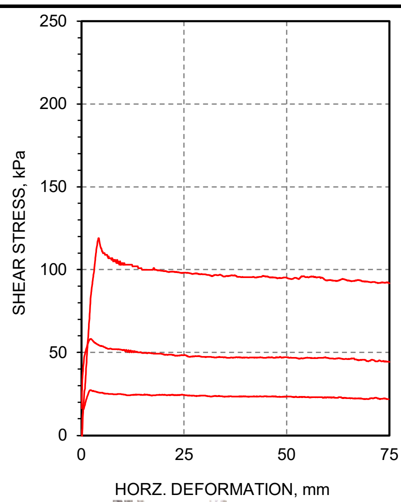
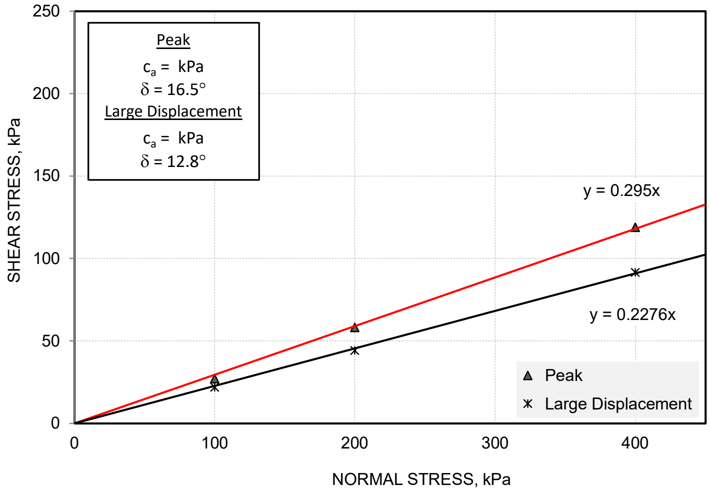
DIRECT SHEAR TEST
(Geosynthetic - Geosynthetic Interface)

ASTM D5321

Client: Burger and Wallace Construction
Geosynthetic 1: 1.5mm HDPE GMB (Smooth)
Project / Reference: Stellenbosch Landfill
Geosynthetic 2: Geotextile Bidim A15
Interface Tested: Geomembrane (Smooth) / Geotextile
Date: 30th Mar 2024





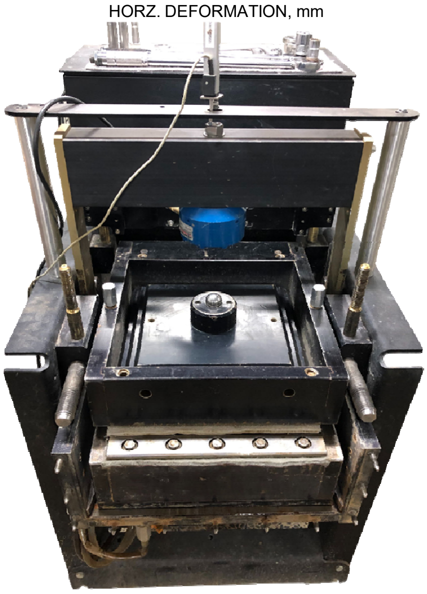


Figure showing direct interface shear apparatus used

Test No.		1	2	3		
Shape		Square	Square	Square		
Initial	Dimensions, mm	300	300	300		
	Area, mm^2	90000	90000	90000		
	Height, mm	80	80	80		
	Water Content, %					
	Dry Density, Mg/m^3					
Consolidation Height, mm		73.8	72.8	72.5		
Final	Moisture Content, %					
	Dry Density, Mg/m^3					
Normal Sress, kPa		100	200	400		
Maximum Shear Stress		27.3	58.3	119.0		
Ultimate Shear Stress, kPa		22.0	44.4	91.8		
Time to Failure, min		2.4	2.4	4.9		
Displacement Rate, mm/min		1.0	1.0	1.0		

Figures showing specimens after testing

Remarks:
- Geosynthetics tested in machine direction.
- Allowed 5 min for settling after reaching the applied load for test.
- Interfaces for each test were wetted but not saturated before each test.
- The reported interface friction angle (δ) and adhesion (c_a) were determined from a line of best-fit drawn through the test data and origin. Caution should be exercised in using these strength parameters for applications involving normal stresses outside the range of the stresses covered by the test series.
- The large displacement shear values were obtained using the shear stress at the displacement of 75 mm.

Tested by: NT **Checked by: DK** **Date: 12th April 2024**

NOTES:
(a) The reported results apply only to the materials and test conditions used in the laboratory testing program. The results do not necessarily apply to other materials or test conditions.
(b) The reported results are submitted for the exclusive use of the client to whom they are addressed.

ANNEXURE C: ACTION LEAKAGE RATE CALCULATIONS: PRIMARY LEAKAGE DETECTION SYSTEM

Belfast Brine Pond - Action Leakage Rates

Primary Liner Leakage Management System

		1,5m depth		**2,5m depth**		**3,5m depth**		**4,5m depth**		**5,5m depth**	
Surface area	SA=	1.21	ha	1.36	ha	1.52	ha	1.68	ha	1.90	ha
Max depth of water	hw=	1.47	m	2.47	m	3.57	m	4.57	m	5.87	m
Radius of hole	R=	0.001	m	0.001	m	0.001	m	0.001	m	0.001	m
Area of hole	A=	3.14286E-06	m^2	3.14E-06	m^2	3.14E-06	m^2	3.14E-06	m^2	3.14E-06	m^2
Number of holes per hectare	No:	2.5	no	2.5	no	2.5	no	2.5	no	2.5	no
Leakage rate per hole	Q=	$0.6A\ (2gh_w)^{0.5}$	m^3/s	$0.6A\ (2gh_w$	m^3/s	$0.6A\ (2gh_w$	m^3/s	$0.6A\ (2gh_w$	m^3/s	$0.6A\ (2gh_w$	m^3/s
	Q=	1.01271E-05	m^3/s	1.31E-05	m^3/s	1.58E-05	m^3/s	1.79E-05	m^3/s	2.02E-05	m^3/s
Action Leakage Rate	ALR=	3.06344E-05	m^3/s	4.46E-05	m^3/s	6E-05	m^3/s	7.5E-05	m^3/s	9.61E-05	m^3/s
	ALR=	**1.838**	lit/min	**2.678**	lit/min	**3.598**	lit/min	**4.500**	lit/min	**5.768**	lit/min
	ALR=	**2187**	**lit/ha/day**	**2835**	**lit/ha/day**	**3409**	**lit/ha/day**	**3857**	**lit/ha/day**	**4371**	**lit/ha/day**
		2647	**lit/day**	**3856**	**lit/day**	**5182**	**lit/day**	**6480**	**lit/day**	**8305**	**lit/day**
		80.5	**m3/month**	**117.3**	**m3/month**	**157.6**	**m3/month**	**197.1**	**m3/month**	**252.6**	**m3/month**

ANNEXURE D: CHEMICAL COMPATIBILITY ASSESSMENT

GMJ Consulting
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E Mail: garthmjames@gmail.com
Mobile: 083 2530592

BVI Consulting Engineers (Pty) Ltd
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Pretoria
South Africa 0001

Attention: Mr F le Roux

27 November 2024

Geosynthetic Clay Liner Compatibility Assessment on Two Leachates in a

Class B Liner for a new Brine Evaporation Pond at Exxaro Belfast Coal Mine, Mpumalanga

1.0 Introduction

On 7 November 2024 Frans le Roux of BVI Consulting Engineers requested a quotation for a compatibility assessment on a Geosynthetic Clay Liner (GCL) with the leachate recovered from the Brine Pond at Mafube Mine, Mpumalanga. The GCL layer will comprise the secondary layer of the Class B Composite Liner for a new Brine Evaporation Pond at the Exxaro Belfast Coal Mine. The specified GCL – *"Kaytech EnviroFix® X800 (or similar approved) needlepunched and thermally locked geosynthetic clay liner"* - comprises natural sodium bentonite and the risk of cationic exchange needs to be assessed.

G James of GMJ Consulting recommended that samples of the leachate from the Mafube Mine Brine Pond be sent to Waterlab (Pty) Ltd for a full chemical analysis. He sent a document listing the typical requirements for such an analysis. G James could then carry out a compatibility assessment of this leachate with the GCL and issue a report thereon.

On 19 November 2024 GMJ Consulting was notified by Frans le Roux that their quote for the required assessment was accepted.

In his communication of 7 November 2024 Frans le Roux supplied GMJ Consulting with the following information:

1. A Waterlab Certificate of Analyses, dated 21 October 2024, listing the General Water Quality Parameters of two leachate samples, namely:
   - Byan (BV1)
   - Byan (BV2)

In a later email from F le Roux dated 27 November 2024 GMJ Consulting were supplied with the Brine Evaporation Pond Embankment and Liner details.

The aim of the GCL compatibility assessment on these two samples is to calculate both the:

- Ratio of Monovalent ions to Divalent ions (RMD) which is defined as the ratio of the total molarity of monovalent cations to the square root of the total molarity of multivalent cations at a given ionic strength, and

- The Ionic Strength (IS) of the leachate

The GCL is specified as the secondary liner to a primary liner specified as a *“1.5 mm thick Hi-Driline (or similar approved) smooth HDPE geomembrane lining”* because the availability of suitable clay in the area is not guaranteed. The details of the composite liner design are detailed in Drawing Number: 35278-00-187-02-01.

The results of the GCL compatibility assessments are described below.

2.0 Assessments

Generally a full and detailed chemical analysis allows one to calculate or estimate the various parameters needed to evaluate bentonite compatibility and includes:

- Cation balance (particularly Ca, Mg & Na)
- Other elements as may be specifically important
- Anions (Cl, Br, Nitrate (NO_3), Nitrite (NO_2), Sulphate (SO_4) and Phosphate (PO_4))
- pH (for H and OH concentrations)
- Total alkalinity for Carbonate (CO_3) and Bicarbonate (HCO_3) concentrations
- Electrical conductivity (EC)
- Total dissolved salts (TDS)
- Total suspended solids (TSS)

The Waterlab Certificate of Analyses provided most of the required information.

2.1 RMD

The chemical analysis shows high Ca and Mg divalent ion concentrations:

- Byan BV1 and BV 2: **Ca** - **1385** and **1440** mg/L and **Mg** - **1416** and **1455** mg/L, respectively, as opposed to the **Na** and **K** monovalent ion concentrations of **1027** and **1055** mg/L and **212** and **212** mg/L, respectively.

This indicates a preponderance of divalent ion concentrations over the monovalent ions and implies a propensity for cation exchange however the ratio of monovalent to divalent ions (RMD) needs to be examined.

Based on all the monovalent and divalent ions in the solutions the RMDs were calculated.

The RMD thresholds used in this assessment are:

- **RMD < 0.07** » **problems with compatibility**
- **0.07 < RMD < 0.22** » **caution** should be exercised
- **RMD > 0.22** » **acceptable** to use GCL in most cases

RMD Results (based on all the monovalent and divalent ions)**:**

- Byan Sample BV1 = **0.164** » **0.07 < RMD < 0.22** » **caution** should be exercised.
- Byan Sample BV2 = **0.166** » **0.07 < RMD < 0.22** » **caution** should be exercised.

Both samples are very similar in chemical composition and identical in pH.

Based on just the **Ca** & **Mg** divalent ion and **Na** & **K** monovalent ion concentrations the RMD values are very similar as well at **0.147** and **0.148** for the BV1 and BV2 samples, respectively. This confirms the need to exercise caution in the application.

Meer and Benson (2009) showed that large increases in hydraulic conductivity and loss of swelling capacity occurred for solutions having RMD ⩽ 0.07. This is not the case for these samples. Modest or small changes in hydraulic conductivity and swell index were obtained when the RMD was ⩽ 0.14.

Based on the RMD results for samples BV1 and BV2 small changes in hydraulic conductivity and swell index are thus anticipated.

However ionic strength has to be considered in conjunction with all of these findings.

2.2 Ionic Strength (IS)

Ionic strength is a function of concentration and charge of all ions in a given solution. The chemical analysis of the Byan samples presents high concentrations of Sulphates and Bicarbonates and thus yield reasonably moderate ionic strengths based on the total molar concentrations.

The thresholds used in the assessment for **IS** are:

- **IS > 100mM** » Osmotic Issues.
- **1mM < IS < 100mM** » Exercise caution if RMD < 0.07.
- **IS < 1mM** » Acceptable conditions for compatibility.

IS Results:

Based on the total molar concentrations:

- Byan Sample BV1 = **28.89** mM » **1mM < IS < 100mM** » Exercise caution if RMD < 0.07.
- Byan Sample BV2 = **29.32** mM » **1mM < IS < 100mM** » Exercise caution if RMD < 0.07.

The RMD values are not less than 0.07 so compatibility appears to be acceptable.

However using the TDS and EC results as additional checks for IS the calculations indicate much higher ionic strength values:

Based on TDS:

- Byan Sample BV1 = **432.9** mM » **IS > 100mM** » Osmotic Issues.
- Byan Sample BV2 = **416.6** mM » **IS > 100mM** » Osmotic Issues.

Based on EC:

- Byan Sample BV1 = **162.4** mM » **IS > 100mM** » Osmotic Issues.
- Byan Sample BV2 = **162.4** mM » **IS > 100mM** » Osmotic Issues.

Thus osmotic issues could present themselves. The potential for cationic exchange can be expected.

High ionic strength leachate directly in contact with the bentonite will result in reduced swells of the bentonite.

2.3 Swell Index

As a further check it is recommended that Swell Index (SI) tests be carried out on the bentonite recovered from a sample of the specified natural sodium bentonite GCL in direct contact with either of the Byan samples – they are very similar.

Generally the higher the ionic strength the greater the negative effect on the swell. This needs to be assessed by measuring the effects of the ionic strength on the swell index of the candidate bentonite.

GRI-GCL3 Standard Specification for “Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCLs)” - Table 1(a) – Specification for Geosynthetic Clay Liners (GCLs) recommends a minimum Swell Index value of 24 ml/2g.

2.4 Geotextile Durabilty

The geotextile components of the specified needlepunched and thermally locked GCL, namely a nonwoven staple fibre cover geotextile and a woven tape carrier geotextile, are manufactured from polypropylene resin which has a known resistant to high alkaline conditions. The pH results of the two Byan samples are identical at 7.5 and thus will pose no threat to the durability of the geotextile components of the GCL.

3.0 Interpretation of GCL Compatibility Assessment

In general, increasing the ionic strength of a leachate increases the hydraulic conductivity of the GCL. Also for a given ionic strength, decreasing the RMD of the leachate results in a higher GCL hydraulic conductivity.

Using Kolstad's plots of RMD versus Ionic strength, **Figure 1** (Kolstad et al, 2004 & 2006), one can see that the hydraulic conductivity based on the calculated RMD (for all the monovalent and divalent ions) and IS (for all the molar concentrations) the two leachates can be estimated to be in the lower 10^{-9} cm/s (10^{-11} m/s) range which may be considered acceptable for the intended purpose as this is about one order of magnitude lower than the hydraulic conductivity of a natural sodium bentonite GCL as tested with deionised water under laboratory conditions. A potential for higher hydraulic conductivities in the GCL exist when one considers the high IS when determined by the TDS and EC of the Byan leachate samples.

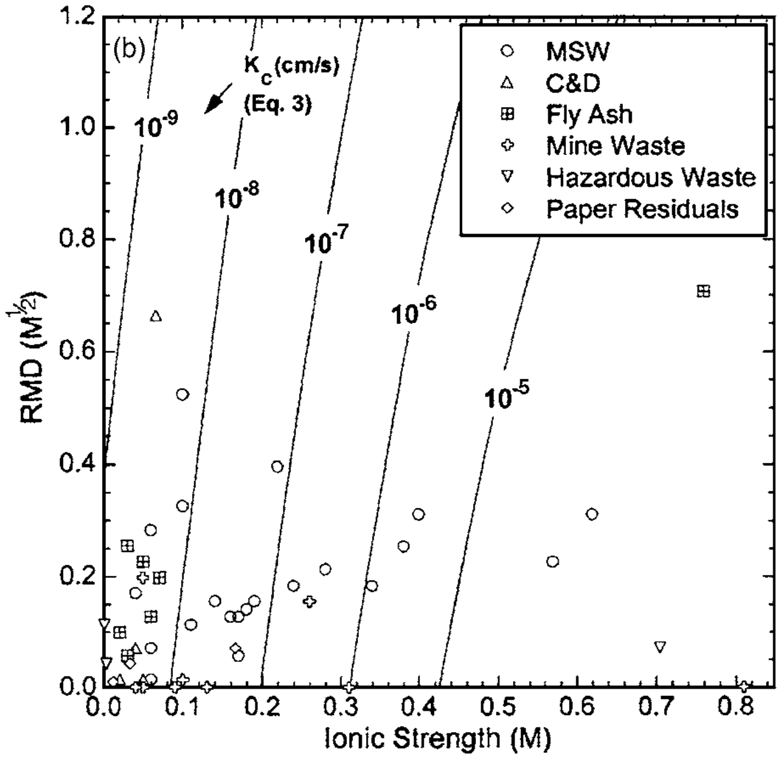

Figure 1: Isoperm chart with anticipated hydraulic conductivities for various CCR leachates (after Kolstad et al)

Note: 1M = 1000mM

It is important to note that the Kolstad method is based on laboratory testing where the GCL was directly hydrated with the different leachates and liquors listed (i.e., no freshwater pre-hydration) and testing was performed at a confining pressure of 20 kPa (representing less than ±1.0m of material load). These testing conditions may be overly conservative for certain sites but may have some relevance to the conditions pertaining to the new Brine Evapiration Pond.

According to Drawing Number 35278-00-187-02-01 the design detail of the liner system shows the sub-grade onto which it is being laid as a *"150 mm thick G7 quality levelling layer from free-issue or commercial sources, compacted to 95 % of Modified AASHTO Density"*. In situations where a GCL is deployed on an inert subgrade soil that is compacted wet of optimum, the GCL will tend to hydrate from the relatively clean moisture in the subgrade.

Pre-hydration of the GCL to prevent hydration of the bentonite from moisture within a leachate contaminated sub-grade prior to placement of the primary geomembrane liner is always a recommended approach to mitigate the effects of cationic exchange.

The new Brine Evaporation Pond is on a greenfield site and thus contaminated sub-grade conditions are unlikely to occur.

Lee and Shackelford (2005) showed that a GCL which is pre-hydrated with clean water (before being exposed to a harsh solution or in this case the Byan leachate) is expected to exhibit a lower hydraulic conductivity than one hydrated directly with the solution. This endorses the need to pre-hydrate the candidate GCL with clean water to increase the bentonite's resistance to cationic exchange which would otherwise occur when exposed to the Byan leachates or similar.

Petrov et al (1997) showed that higher confining pressures will decrease bentonite porosity, and tend to decrease GCL permeability. Testing has shown that higher confining pressures can improve hydraulic conductivity even in contact with aggressive solutions Obviously in the Brine Pond application the compressive loads will be imposed by the depth of brine which is unlikely to be constant.

Thus, maintaining hydration of the GCL is important so that the effects of leakages of Byan-type leachates through the geomembrane into the GCL or a leachate contaminated sub-grade layer beneath the GCL is minimised.

4.0 Conclusion

This compatibility assessment indicates that caution should be exercised if the GCL is in direct contact with the Byan sample leachates which exhibit moderate RMDs and moderate ionic strengths. There is a risk of cationic exchange of the natural sodium bentonite component of the specified GCL (*Kaytech EnviroFix® X800 (or similar approved) needlepunched and thermally locked geosynthetic clay liner*) if the RMD is below 0.07 which fortunately is not the case. However one must be cognisant of the high IS when considered in terms of the TDS and EC of the leachates.

The unavailability of suitable, low permeability clay (≤ 10^{-9} cm/s) is always an issue and thus pre-hydration of the GCL is recommended to reduce the negative effects of cationic exchange. It is critical that the GCL be placed on a smooth inert surface and pre-hydrated before installing the HDPE liner.

The Kolstad plot, **Figure 1**, indicates a moderately low hydraulic conductivity being maintained in the GCL for the RMDs and Ionic Strengths calculated but is one to two orders of magnitude greater than the hydraulic conductivity of the specified natural sodium bentonite GCL hydrated with deionised water under laboratory conditions.

Hydraulic conductivity testing using flexible wall permeameters may be considered to substantiate these assertions especially where the practicalities of preparing the GCL using different pre-hydration methods prove to be a real challenge.

Swell Index testing of the candidate GCL bentonite with the Byan leachate should be considered if necessary to establish whether the recommended minimum Swell Index value of 24 ml/2g according to *GRI-GCL3 Standard Specification for "Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCLs)" - Table 1(a) – Specification for Geosynthetic Clay Liners (GCLs)* is achieved.

However with proper precautions taken during placement and pre-hydration of the GCL and having a full understanding of the boundary conditions (sub-grade, moisture, contamination etc) optimal performance of the GCL can be expected.

5.0 References

Kolstad, D., Benson, C. and Edil, T., (2004) "Hydraulic Conductivity and Swell of Nonprehydrated GCLs Permeated with Multispecies Inorganic Solutions", Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 12, December 2004, pp.1236-1249. 19.

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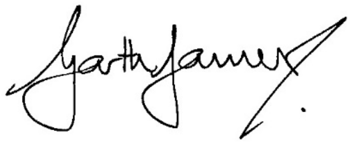

GM James