



WALTER SISULU LOCAL MUNICIPALITY

ESTABLISHMENT OF A NEW LANDFILL SITE IN MALETSWAI LOCAL MUNICIPALITY

77/2024/2025

DESIGN REPORT

AUGUST 2026

PREPARED FOR:	PREPARED BY:
Walter Sisulu Local Municipality PO Box 13 BURGERSDORP 9744	Imbawula Civil Projects (Pty) Ltd P O Box 719 Butterworth 4960
Tell: (051) 653 1777 Cell: (063) 2383 648 Fax: (051) 653 0056	Tel: (012) 372 3708 Cel: (061) 446 4485 Fax: (086) 750 4292
<i>Enquiries:</i> Zolile Maqhinyana Manager: PMU Email: Zolile.maqhinyana@wslm.gov.za	<i>Enquiries:</i> Mr. B. Citwa Project Engineer Email: bulelani@imbawulacp.co.za

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Authors	Sifiso Maseko
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Name	Designation	Signature	Date
Sifiso Maseko	Profession Engineer (Pr)		28/08/2026
Bulelani Citwa	Project Engineer		28/08/2026

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1. INTRODUCTION

1.1 Preamble

Imbawula Imbawula Civil Projects (Pty) Ltd was appointed by the Walter Sisulu Local Municipality (WSLM) on 03 April 2025 as the Professional Service Provider and Employer's Agent for the planning, design, environmental authorisation, and implementation support associated with the development of a new municipal landfill facility and the rehabilitation and closure of the existing Maletswai landfill site under Contract No. 77/2024/2025.

The appointment forms part of the Municipality's strategy to improve waste management services within its area of jurisdiction and to ensure compliance with current environmental legislation governing waste disposal facilities. The scope of services includes project initiation, feasibility assessment, environmental authorisation processes, engineering investigations, detailed design development, preparation of tender documentation, construction monitoring, commissioning support and project close-out.

This report presents the detailed engineering design for the proposed Maletswai Landfill Facility and associated infrastructure. The report consolidates the findings of the site selection process, environmental investigations, geotechnical assessments, geohydrological studies, waste classification assessments and engineering analyses undertaken during the planning and design phases. The design has been developed to provide a construction-ready solution that satisfies regulatory requirements while ensuring the long-term protection of human health and the environment

1.2 Project Background and Mandate

The existing Maletswai landfill has reached the end of its operational lifespan and has been approved for closure in accordance with applicable waste management legislation. To ensure continued waste disposal services, the Walter Sisulu Local Municipality initiated the development of a new engineered landfill facility and associated Material Recovery Facility (MRF).

Imbawula Civil Projects (Pty) Ltd was appointed to undertake the Environmental Impact Assessment (EIA), prepare the Environmental Management Programme (EMPr), support the Waste Management Licence application, conduct the necessary engineering investigations, and develop the detailed engineering design and operational framework for the proposed facility.

A site selection study conducted in 2025 assessed three potential sites Table 1 and Figure 1 based on environmental, geotechnical, hydrological and engineering criteria. Following the assessment, a preferred site was selected due to its favorable environmental conditions, availability of services and potential for future expansion.

Table 1: Site Selection

Parameters	Preferred Site	Alternative Site No. 1	Alternative Site No. 2
Visual Impacts	Low	High	High
Proximity to Watercourses	More than 200m	Less than 100m	More than 100 m
Geotechnical	Ideal	Ideal	Difficult to excavate
Availability of Services	Road and Electricity	Road	None
Future Expansions	Possible	Not possible	Not Possible

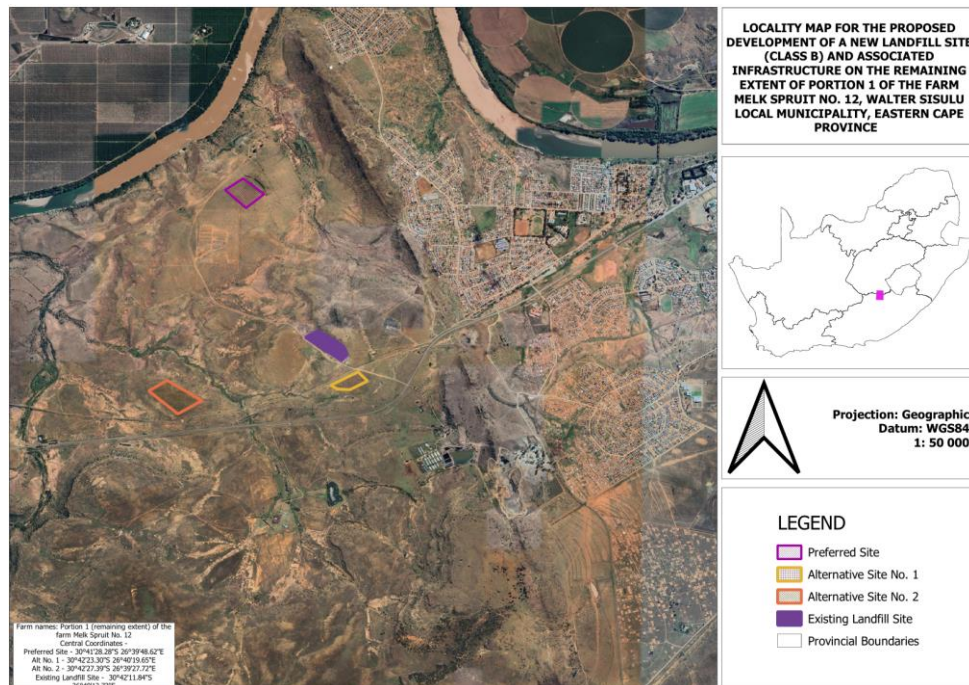


Figure 1 - Locality Map for the proposed site

As part of this appointment, Imbawula Civil Project is responsible for the complete civil design and operating plan for the proposed new Maletswai landfill. This report, therefore, presents the engineering design and operational framework for the planned extension and full development of the Maletswai Landfill Site, providing WSLM with a construction-ready solution that aligns with the National Environmental Management: Waste Act, 2008 (Act No. 59 of 2008) (NEMWA) and the National Waste Management Strategy (2020).

1.3 Study Objectives

The primary objective of the project is to develop a fully compliant engineered landfill facility capable of safely managing municipal solid waste generated within the Walter Sisulu Local Municipality over the projected design life of approximately twenty years.

To achieve this objective, the detailed design process sought to fulfil the following specific goals:

- Confirm the suitability of the selected site for long-term waste disposal through detailed environmental, geotechnical and geohydrological investigations.
- Determine the appropriate landfill classification in accordance with the National Norms and Standards for Disposal of Waste to Landfill and anticipated waste acceptance requirements.
- Establish the required landfill airspace based on projected waste generation rates and population growth forecasts.
- Develop a phased landfill layout capable of accommodating future expansion while maintaining operational efficiency.
- Design an engineered containment system incorporating a composite GCL-HDPE liner system, leachate collection infrastructure and environmental monitoring facilities.
- Develop a comprehensive stormwater management system capable of separating clean and contaminated runoff and preventing pollution of surrounding water resources.
- Design a Pollution Control Dam and integrated leachate management system to safely contain contaminated water generated during landfill operations.
- Provide supporting infrastructure including access roads, internal roads, weighbridge facilities, administration buildings, security infrastructure, utilities and waste handling areas.
- Develop an operational management framework that promotes efficient landfill operation, environmental compliance and occupational health and safety.
- Produce detailed engineering drawings, technical specifications and cost estimates suitable for procurement and construction purposes.

The detailed design presented in this report provides the technical basis for construction, operation and future management of the proposed landfill facility.

1.4 Extent of Work

The scope of this assignment encompasses the detailed engineering design of a new municipal landfill facility and associated infrastructure intended to serve the waste management requirements of the Maletswai area for a minimum design life of twenty years.

The design has been undertaken in accordance with the requirements of the National Environmental Management: Waste Act, 2008 (Act No. 59 of 2008), the National Waste Management Strategy (2020), the National Norms and Standards for Disposal of Waste to Landfill (as amended), and other applicable South African legislation, standards and engineering guidelines.

The scope of work included detailed topographical analysis, geotechnical investigations, geohydrological assessments, waste generation projections, landfill airspace calculations, landfill classification assessments and infrastructure planning. Based on these investigations, detailed designs were prepared for all major components of the facility.

The engineering design includes the development of landfill cells, phased filling plans, composite liner systems, leachate collection infrastructure, landfill gas management provisions, stormwater management systems, pollution control facilities, internal road networks, access roads, weighbridge facilities, administration buildings, workshop areas, security fencing, gate control infrastructure, utility services and a Material Recovery Facility.

Particular emphasis was placed on environmental protection measures including groundwater protection, stormwater control, leachate containment, erosion prevention and long-term closure planning. The design incorporates an engineered composite liner system consisting of a Geosynthetic Clay Liner and a 2.0 mm textured HDPE geomembrane, adopted following geotechnical investigations which confirmed that suitable clay materials are not available on site for construction of a conventional compacted clay liner.

In addition to the engineering design, the project scope includes preparation of an operational management plan, environmental monitoring framework, construction quality assurance requirements, technical specifications, detailed drawings and cost estimates. These documents collectively provide the basis for regulatory approvals, tender procurement, construction implementation and long-term operation of the facility.

Upon completion, the Walter Sisulu Local Municipality will possess a fully engineered, environmentally compliant and construction-ready landfill development capable of meeting current and future waste management requirements while protecting surrounding environmental resources and public health.

2. Site Description

The project site is located Figure 2 at 30°41'28.34"S, 26°39'52.36"E, near Aliwal North in the Eastern Cape Province, close to the boundary with the Free State. Primary access to the site is via the N6 national road, which links East London to Bloemfontein and passes directly through Aliwal North. From Aliwal North, access is obtained by turning onto provincial and local gravel roads leading south-east towards the project area. These access roads are currently used for agricultural activities and are suitable for light to medium construction vehicles.

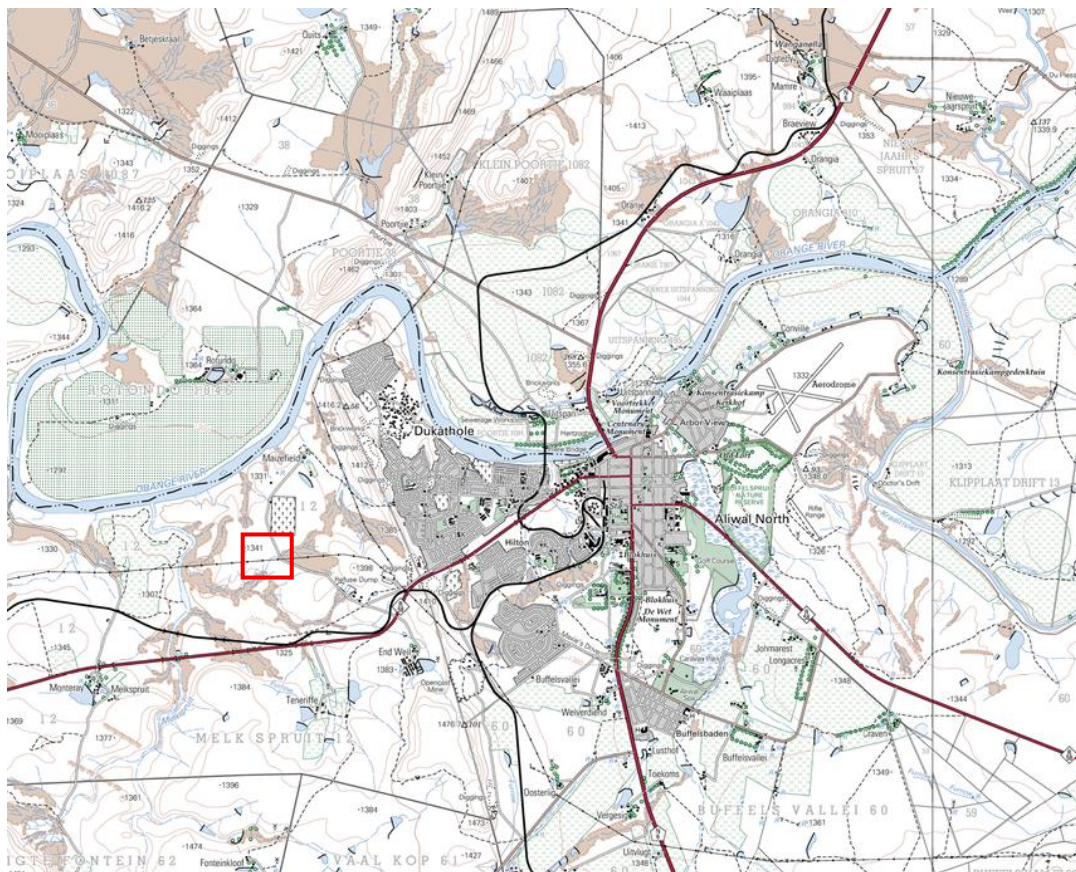


Figure 2: Locality

The final approach to the site is via existing farm access roads, which may require minor upgrading or maintenance to accommodate construction traffic, particularly during wet weather conditions. No new permanent access roads are anticipated, as existing infrastructure provides adequate access to the site

3. ENGINEERING DESIGN CRITERIA AND SPECIFICATIONS

3.1 Introduction

This section presents the engineering design criteria, assumptions and technical specifications adopted for the proposed Maletswai Landfill Facility and associated infrastructure. The purpose of the chapter is to establish the engineering basis upon which the detailed design has been developed and to demonstrate compliance with applicable legislative requirements, national standards and accepted engineering practice.

The proposed landfill facility has been designed to provide a sustainable, environmentally responsible and cost-effective waste disposal solution for the Walter Sisulu Local Municipality over a minimum operational period of twenty years. The design incorporates engineered containment systems, stormwater management infrastructure, leachate collection and storage facilities, access roads, environmental monitoring systems and supporting operational infrastructure required for the safe disposal of municipal solid waste.

The engineering design philosophy is based on the principle of pollution prevention through the implementation of multiple containment barriers, effective separation of clean and contaminated water, continuous environmental monitoring and progressive rehabilitation. This approach ensures that the landfill can operate safely while protecting surrounding soil, groundwater and surface water resources.

The design has been informed by topographical surveys, geotechnical investigations, hydrogeological assessments, climatic data analyses, waste generation forecasts and environmental studies undertaken as part of the Environmental Impact Assessment process.

3.2 Legislative and Regulatory Framework

The engineering design has been undertaken in accordance with the requirements of the South African environmental and waste management legislative framework. Particular consideration has been given to the requirements of the National Environmental Management Act (Act 107 of 1998), the National Environmental Management: Waste Act (Act 59 of 2008), the National Water Act (Act 36 of 1998) and the National Norms and Standards for Disposal of Waste to Landfill.

The National Norms and Standards establish the minimum technical requirements for landfill classification, liner systems, leachate management, groundwater protection, environmental monitoring and landfill operation. The design has therefore been developed to satisfy these requirements while incorporating modern landfill engineering practices and international best-practice principles.

The facility has also been designed with due consideration for municipal planning requirements, occupational health and safety legislation, construction standards and relevant South African National Standards applicable to earthworks, geosynthetics, drainage infrastructure and civil engineering works.

3.3 Design Objectives

The primary objective of the engineering design is to provide a landfill facility capable of safely accommodating municipal waste generated within the Walter Sisulu Local Municipality while preventing unacceptable environmental impacts.

The detailed design seeks to achieve the following objectives:

- Provide sufficient disposal capacity for at least twenty years.
- Prevent contamination of groundwater resources.
- Prevent pollution of nearby surface water systems.
- Minimise leachate generation and facilitate effective leachate management.
- Separate clean and contaminated stormwater runoff.

- Ensure stable landfill slopes during operation and after closure.
- Facilitate efficient waste placement and compaction.
- Provide safe access for waste collection vehicles and operational equipment.
- Support future expansion of the facility.
- Comply with all regulatory and environmental requirements.

These objectives have informed the development of all engineering components included in the detailed design.

3.4 Waste Quantities and Design Life

Waste disposal records, population statistics and projected growth trends were used to estimate future waste quantities requiring disposal at the facility. Based on current municipal waste generation rates and anticipated growth within the municipal area, the proposed landfill has been designed to accommodate waste disposal for a minimum period of twenty years. The landfill layout has been developed to allow phased expansion while maintaining operational efficiency throughout the facility lifespan.

The final landfill footprint, excavation depth and airspace requirements have been determined based on the estimated annual waste quantities and assumed in-place waste densities achieved through compaction. Sufficient airspace has been provided to accommodate daily cover requirements, intermediate cover requirements and final capping materials.

The landfill development will be implemented in phases to reduce initial construction costs while ensuring that future expansion can occur without disrupting ongoing operations.

3.5 Climatic Design Parameters

The design of the landfill, stormwater management infrastructure and leachate management system has been based on regional climatic conditions.

Hydrological data obtained from the WR2012 database and South African Weather Service indicate that the study area receives an average annual rainfall of approximately 493 mm.

Average annual S-pan evaporation is approximately 1 675 mm, resulting in a strongly negative water balance. After application of an appropriate correction factor, actual evaporation is estimated to exceed annual rainfall by a significant margin.

These climatic conditions are favorable for landfill development as they reduce the potential for long-term leachate generation. Nevertheless, short-duration high-intensity storm events remain capable of generating significant runoff and contaminated water volumes requiring management.

Hydrological analyses for the sizing of stormwater management infrastructure were undertaken using deterministic design methods. Peak runoff from catchment areas associated with the landfill facility was estimated using the Rational Method, which is widely accepted for the design of drainage systems serving relatively small catchments. The Rational Method is expressed as:

$$Q = 0.278CIA$$

Where:

- Q = peak runoff (m^3/s)
- C = runoff coefficient
- I = rainfall intensity (mm/h) corresponding to the selected return period and time of concentration
- A = catchment area (km^2)

Appropriate runoff coefficients were selected based on the anticipated surface conditions within the landfill footprint, access roads, disturbed areas and surrounding natural catchments. Rainfall intensities were derived from regional intensity-duration-frequency (IDF) data and applied in accordance with the relevant design return periods.

The following design storm criteria have therefore been adopted:

- 1:10-year storm event for operational drainage systems.
- 1:20-year storm event for internal stormwater conveyance systems.
- 1:50-year storm event for Pollution Control Dam sizing.
- 1:100-year storm event for emergency spillway assessment and flood risk evaluation.

The Rational Method calculations were used to determine peak flow rates for diversion channels, stormwater drains, culverts and associated hydraulic structures, while storage facilities such as the Pollution Control Dam were assessed using deterministic runoff volume calculations based on applicable design storm events.

These design criteria ensure that the facility remains operational and environmentally compliant during extreme weather events.

3.6 Geotechnical Design Criteria

A geotechnical investigation was undertaken to assess the suitability of the proposed Maletswai Landfill site for development. The site is underlain by residual silty sands, sandy silts and weathered Beaufort Group mudstones, with competent bedrock encountered at shallow depth, providing favourable founding conditions.

Groundwater was not encountered during the investigation, and regional hydrogeological data indicate groundwater depths greater than 5–6 m below ground level. The proposed landfill excavation depth is approximately 3 m, ensuring separation between the landfill base and groundwater.

Laboratory testing yielded hydraulic conductivity values ranging from 9.12×10^{-10} m/s to 1.75×10^{-9} m/s, while direct shear testing produced friction angles of 39.2° – 44.3° and cohesion values of 1.9–32.1 kPa. Conservative design parameters adopted for stability analyses include $c' = 9$ kPa, $\phi' = 39^\circ$, $\gamma = 18$ kN/m³, and $\gamma_{\text{sat}} = 20$ kN/m³.

The principal geotechnical design criteria are:

- Allowable bearing capacity: 150 kPa

- Maximum excavation depth: 3 m
- Groundwater depth: >5–6 m
- Internal side slopes: 1V:3H
- External embankment slopes: 1V:3H
- Design cohesion (c'): 9 kPa
- Design friction angle (ϕ'): 39°
- Bulk unit weight (γ): 18 kN/m³
- Saturated unit weight (γ_{sat}): 20 kN/m³
- Hydraulic conductivity: 9.12×10^{-10} to 1.75×10^{-9} m/s
- Minimum long-term FoS: 1.5
- Minimum construction FoS: 1.3
- Minimum pseudo-static FoS: 1.1

Slope stability modelling yielded a minimum Factor of Safety of **1.7**, confirming that the proposed landfill geometry is stable under anticipated operating conditions. Although low-permeability soils occur locally, a **composite GCL-HDPE liner system** was selected to ensure consistent regulatory compliance, enhanced containment performance and long-term environmental protection.

3.7 Landfill Containment Design Criteria

Protection of groundwater resources represents one of the primary objectives of the landfill design. To achieve this objective, a composite liner system has been adopted as the principal containment barrier.

The selected liner system consists of:

- Prepared and compacted subgrade.
- Reinforced Geosynthetic Clay Liner (GCL).
- 2.0 mm textured HDPE geomembrane.
- Geotextile protection layer.

- Leachate drainage blanket.
- Perforated HDPE leachate collection pipes.

This configuration provides both primary and secondary containment barriers and significantly reduces the potential for contaminant migration into underlying soils and groundwater. The selected liner system was chosen based on site-specific geotechnical conditions, long-term performance requirements and current landfill engineering best practice.

3.8 Leachate Management Design Criteria

Leachate generated within the landfill must be effectively collected, conveyed and stored to prevent environmental contamination. The leachate management philosophy adopted for the facility is aligned with Rowe's contaminant barrier system framework, which emphasises the integration of multiple engineered barriers to minimise contaminant migration and protect groundwater resources over the long term.

According to Rowe's framework, landfill environmental protection is achieved through the combined performance of the waste mass, cover systems, leachate collection and removal systems, liner systems, natural geological barriers and environmental monitoring programmes. The proposed landfill design therefore incorporates a composite containment approach consisting of a low-permeability liner system, efficient leachate collection infrastructure, stormwater controls and ongoing environmental monitoring.

The leachate management system has been designed to:

- Minimise leachate generation through effective stormwater diversion and operational controls.
- Maintain leachate heads below 300 mm on the liner surface.
- Rapidly collect and remove leachate from landfill cells.
- Reduce hydraulic loading on the liner system.
- Prevent uncontrolled discharge to the environment.
- Provide sufficient storage capacity during storm events.

- Facilitate monitoring, maintenance and operational management.
- Support the long-term performance objectives of the composite liner system.

The proposed system comprises a granular drainage blanket, perforated HDPE collector pipes, collection sumps, transfer pipelines and a lined Pollution Control Dam. The drainage layer and pipe network have been designed to promote rapid leachate removal and minimise the potential for excessive leachate accumulation above the liner system, consistent with the principles advocated by Rowe for reducing contaminant flux through landfill barriers.

The Pollution Control Dam will be designed to accommodate both operational leachate generation and contaminated stormwater runoff generated during design storm events. The facility will also incorporate routine inspection and maintenance procedures to ensure continued performance of the leachate collection system throughout the operational and post-closure phases of the landfill. Through the integration of these engineered components, the design seeks to achieve a robust multi-barrier containment system that provides long-term environmental protection and regulatory compliance.

3.9 Stormwater Management Design Criteria

The stormwater management strategy is based on the principle of separating clean water from contaminated water.

Clean stormwater generated outside operational landfill areas will be diverted around the facility through a network of cut-off drains, diversion berms and lined channels. This will prevent unnecessary inflows into active waste disposal areas and reduce leachate generation.

Stormwater generated within operational areas will be classified as contaminated and directed towards the leachate management system and Pollution Control Dam.

The stormwater management system has been designed to:

- Preventing erosion.
- Minimize sediment transport.

- Reduce leachate generation.
- Protect downstream water resources.
- Maintain compliance with the National Water Act.

All channels, culverts, energy dissipation structures and detention facilities have been sized using accepted hydrological and hydraulic design methodologies.

3.10 Chapter Summary

The engineering design criteria and specifications presented in this chapter establish the technical framework for the detailed design of the proposed Maletswai Landfill Facility. The adopted criteria have been selected to ensure environmental protection, regulatory compliance, operational efficiency and long-term sustainability. The subsequent chapters present the detailed landfill cell design, containment system design, stormwater management plan and leachate management infrastructure developed in accordance with these criteria.

4. SITE CLASSIFICATION AND WASTE DISPOSAL NEED

4.1 Current volumes

The current volumes for the Maletswai waste disposal was source from the Integrated Waste Management Plan 2024-2026 which presented the total disposal volumes for the whole district municipality of Walter Sisulu. Table 2 below is a summary of district volume per local municipality per year.

Table 2: Showing estimated waste volumes in each landfill site- (Source: WSLM, IWMP2024-2029 pg 65)

Landfill Site	Estimated Volumes of Waste
Maletswai	12,225,67 tons
Burgersdorp	5,606,83 tons,
Steynsburg	1,828,9tons,
James Calata	784 tons
Venterstad	735 tons.

Using the data presented in Table 2, it is estimated that approximately 12,256 tonnes of waste per annum were disposed of at the existing landfill during the 2024 base year. Based on an assumed 260 operational days per year, this corresponds to an average disposal rate of approximately 47.1 tonnes per day in 2024.

Applying a conservative annual growth rate of 1.5%, and projecting waste quantities forward to 2027, which is assumed to be the commencement year of disposal at the proposed landfill, the estimated annual waste quantity increases to approximately 12,780.6 tonnes per annum. This translates to an Initial Rate of Deposition (IRD) of approximately 49.1 tonnes per day.

4.2 Sizing of the Landfill

The estimated size of the generated waste stream as discussed was based on the IWMP for Walter Sisulu District Municipality as 49.1 ton of waste per day which is inclusive of public and private disposal in the area.

Table 3 Landfill Size Classes

Landfill Size Class	Maximum Rate of Deposition (MRD) (Tonnes per day) 1998
Communal	<25
Small	>25 <150
Medium	>150 <500
Large	>500

The size of the Landfill is calculated as follows

- $MRD = (IRD)(1+d)^t$
- IRD- (Initial rate of deposition on site in T/day) = 49.1 (2027)
- d -(Expected annual development rate) = 1.5%
- t -(years since deposition started at IRD) = 20
- MRD -(Maximum rate of Deposition after t years) = 66.1 ton/day

The maximum rate of deposition (MRD) for Maletswai site is calculated as follows:

$$\text{MRD} = 49.1 \text{ tons /day} \times (1+0.015)^{20} = 66.1 \text{ ton/ day}$$

Base on Table 3 the Maletswai site will be classified as an S or Small site according to MRs

4.3 Design of Landfill Facility

The conceptual design process considered both the current waste volumes at Maletswai and expected growth over the design life of the facility. Key factors included daily and annual waste intake, compaction methods, cover material requirements, and site geometry, including side slopes, excavation depth, and final height above ground level. Operational assumptions, such as the use of landfill compactors and the anticipated ratio of cover to waste, were incorporated to provide a realistic estimate of airspace requirements and overall site capacity.

The proposed cell layout and design parameters were developed to optimize the use of available airspace while ensuring safe, environmentally compliant operations. Consideration was also given to potential operational efficiencies, such as waste diversion and recycling initiatives, which could extend the lifespan of the landfill beyond the initially projected period. The preliminary design serves as a basis for further detailed engineering, allowing for refinement of cell geometry, drainage, and cover strategies once more site-specific investigations have been completed.

4.4 Disposals need / Airspace required

The airspace utilisation calculations for the proposed Maletswai landfill are based on projected waste generation volumes for the Maletswai service area. The base year waste generation was 12,225.67 t in 2024, which, assuming a waste density of 0.8 t/m³, corresponds to approximately 15,282 m³. An annual growth rate of 1.5% was applied to project future waste volumes.

Although the base year is 2024, the landfill is anticipated to become operational in year 2027, meaning the first actual disposal year is 2027. Using the growth rate, the waste volume in 2027 is projected to be approximately 15,960 m³:

$$\text{Waste}_{2027} = 15,282.1 \times (1.015)^3 = 15,960 \text{ m}^3$$

In addition to the waste itself, allowance was made for daily and final cover material. A cover-to-waste ratio of 1:4 was applied, resulting in a total airspace requirement equal to 1.25 times the waste volume.

The projected waste generation volumes, cumulative waste volumes, and total airspace utilisation over the 20-year design life are summarised in **Table 4**. Based on these projections, the Maletswai landfill will require a total airspace of approximately 460,880 m³, including cover. These calculations provide the basis for preliminary site layout and design, ensuring sufficient capacity for the projected growth in waste disposal.

Table 4 Waste Generation Volumes and Airspace Required

No.	Year	Waste Volume Generated (m ³)	Cumulative Waste (m ³)	Total Airspace Used (m ³)
1	2027	15,960	15,960	19,950
2	2028	16,199	32,159	40,199
3	2029	16,441	48,600	60,750
4	2030	16,686	65,286	81,607
5	2031	16,936	82,222	102,778
6	2032	17,190	99,412	124,265
7	2033	17,447	116,859	146,073
8	2034	17,708	134,567	168,209
9	2035	17,973	152,540	190,675
10	2036	18,241	170,781	213,476
11	2037	18,513	189,294	236,618
12	2038	18,788	208,082	260,103
13	2039	19,067	227,149	283,936
14	2040	19,349	246,498	308,123
15	2041	19,635	266,133	332,667
16	2042	19,923	286,056	357,570
17	2043	20,215	306,271	382,839
18	2044	20,511	326,782	408,478
19	2045	20,810	347,592	434,490
20	2046	21,113	368,705	460,881

From Table 4 it can be deduced that, for a landfill site life of 20 years, a total airspace of approximately 460 881 m³ is required. These values are approximate and depend on several

operational variables, including compaction efficiency, waste composition, and settlement of the waste mass over time

4.5 Available airspace

A conceptual landfill cell layout was developed to determine the available airspace for the Maletswai site. The disposal cell footprint area measures approximately **46,985 m²**, and the design parameters are:

- Excavation to approximately 3 m below ground level.
- A final landfill height of approximately 10 m above ground level.
- Side slopes of 1:3 (vertical: horizontal);
- Sufficient working area on the top platform for vehicle manoeuvring.

The total available airspace was calculated using:

Gross volume = Footprint area × (Final height + excavation depth) × Slope factor

$$= 46,985 \text{ m}^2 \times 13 \text{ m} \times 0.85 = 519\,184 \text{ m}^3$$

his gross airspace includes the full landfill volume and accounts for the slope factor to reflect the side slopes of the cell. The calculation demonstrates that the proposed Maletswai landfill has sufficient capacity to accommodate projected waste disposal volumes, including allowance for cover material, over its 20-year operational life.

4.6 Landfill airspace demand

The total gross landfill airspace requirement for the Maletswai Landfill over the 20-year design horizon has been calculated at approximately 460 881 m³, which includes a 1:4 waste to cover ration allowance for daily and intermediate cover, final capping, and operational inefficiencies. This figure represents the volumetric capacity that must be provided through a combination of landfill footprint and waste height. Selecting the appropriate height is therefore essential to balance land-use efficiency, engineering feasibility, environmental risk, and cost.

To assess the effect of varying landfill heights, the gross airspace requirement with 0.85 shape factor was compared across four scenarios: 5 m, 10 m, 15 m, and 20 m above-ground fill heights, each combined with a 3 m excavation depth. The effective containment heights were therefore 8 m, 13 m, 18 m, and 23 m, respectively. As shown in the table below, increasing the landfill height significantly reduces the required footprint area. For instance, at 5 m above ground, the landfill would require more than 5.7 ha of net footprint, while at 10 m the net footprint reduces to approximately 3.5 ha.

Table 5: Containment heights considerations

Above-ground Fill (m)	Effective Height (m)	Net Footprint (m ²)	Net (ha)
5	8	57 610	5.7
10	13	35 452	3.5
15	18	25 604	2.5
20	23	20 038	2.0

The 10 m above-ground height (effective 13 m with excavation) is considered the optimal balance for the proposed site. It provides a compact footprint of 3.5 ha, which increases to approximately 6 ha gross once ancillary infrastructure is included. This size fits comfortably within a practical 5-8 ha development needs, allowing space for leachate ponds, access roads, buffer zones, and support facilities. Compared with shallower fills, the 10 m configuration significantly reduces visual impact, stability problems and wind blow effect.

4.7 Sensitivity analysis

A sensitivity analysis is used to test how changes in the assumed waste growth rate affect the required landfill airspace and design life. Since your base case uses 1.5% annual growth, you would normally test lower and higher growth scenarios.

For landfill design, a common approach is:

Low Growth 1.0%

Base Case 1.5%

High Growth 2.0%

Extreme Growth 3.0%

A sensitivity analysis was undertaken to assess the influence of waste growth assumptions on landfill airspace requirements. Growth rates of 1.0%, 1.5%, 2.0% and 3.0% per annum were evaluated over the 20-year design period. The results indicate that total airspace requirements vary from approximately 440,000 m³ under a low-growth scenario to approximately 536,000 m³ under a high-growth scenario. The adopted design value of 460,881 m³, based on a 1.5% annual growth rate, lies within this range and is considered reasonable for planning purposes. Even under the higher growth scenarios, the variation in required airspace remains within approximately 16% of the base case, indicating that the proposed landfill layout provides sufficient flexibility to accommodate uncertainty in future waste generation trends. Annexure E give calculation.

4.7 Leachate Generation Potential

4.7.1. Water balance under local climate

The potential for leachate generation is one of the most critical factors in landfill design, as uncontrolled leachate can contaminate groundwater, surface water, and soils. The likelihood and volume of leachate depend primarily on the climatic water balance of the site and are further influenced by site-specific conditions such as the moisture content of incoming waste, groundwater ingress, and the effectiveness of surface water management. Assessing these variables allows the landfill to be appropriately classified and determines whether a leachate management system is required under the National Environmental Management: Waste Act (Act 59 of 2008) and the 2024 Norms and Standards for Disposal of Waste to Landfill.

4.7.2 Regional Hydrology

The proposed Maletswai Landfill site is located within the Upper Orange Water Management Area (WMA 13) Figure 3, specifically in Quaternary Catchment D14A. This drainage system ultimately discharges north-westward into the Orange River, forming part of the larger Orange–Senqu basin. The regional hydrology is therefore defined by a semi-arid inland drainage network comprising predominantly non-perennial streams and ephemeral drainage lines that flow only during significant rainfall events.

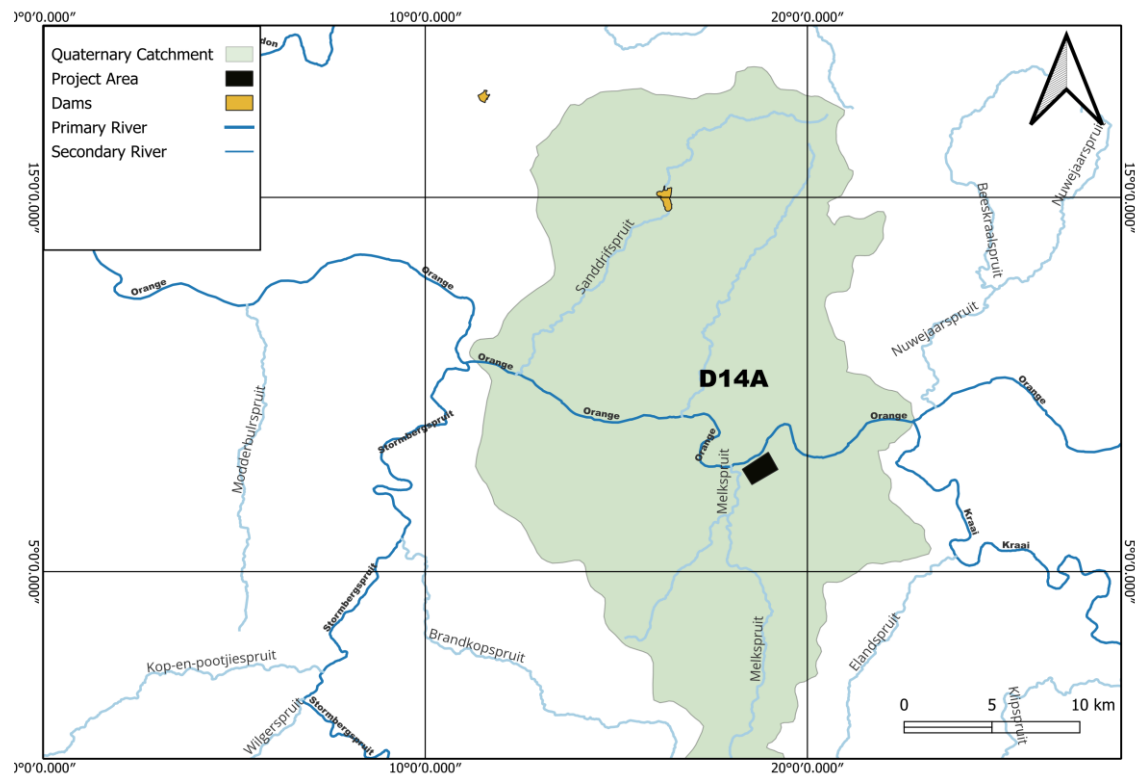


Figure 3 Reginal Hydrological Setting

The topography of the Maletswai area is gently undulating, with surface gradients ranging from 1 % to 5 %, directing overland flow toward minor tributary channels situated north-west of the proposed landfill footprint. The regional mean annual precipitation is approximately 493 mm/year, while mean annual potential evaporation is around 1,675 mm/year, resulting in a strongly negative climatic water balance. Under such conditions, runoff volumes are low, infiltration is limited, and streamflow occurs mainly as short-duration flash events during the summer rainfall period (November – April).

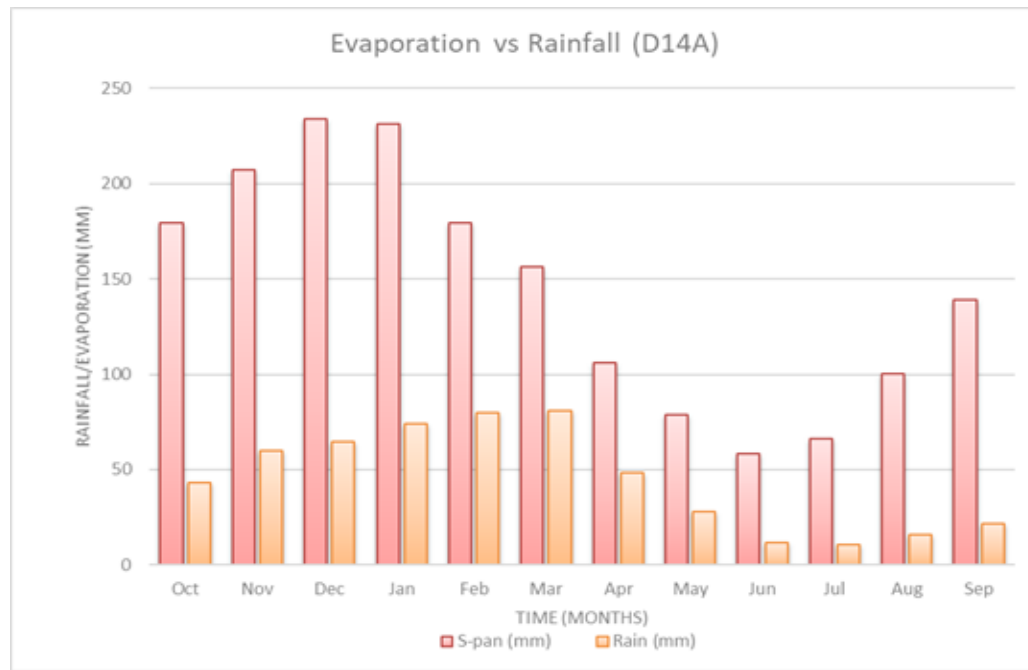


Figure 4 Catchment Rainfall and Evaporation

4.7.3 Climatic Water Balance

The potential for significant leachate generation at a landfill site is governed primarily by the site water balance. This balance is controlled by regional climatic conditions, together with site-specific factors such as the moisture content of incoming waste and the potential ingress of surface water runoff or groundwater into the waste body. In accordance with the Minimum Requirements for Waste Disposal by Landfill, the climatic water balance provides the basis for assessing whether a landfill is likely to generate significant quantities of leachate and for assigning the appropriate B- or B+ classification.

The climatic water balance (B) for the proposed Maletswai Landfill site has been calculated using the standard equation:

$$B = R - E$$

where:

- B is the climatic water balance (mm),

- R is the mean annual rainfall (mm), and
- E is evaporation from a soil surface (mm), taken as either $0.7 \times A$ -pan evaporation or $0.88 \times S$ -pan evaporation, in accordance with the Minimum Requirements.

Climatic data for the Maletswai area indicate a mean annual rainfall of approximately 493 mm, while mean annual S-pan evaporation is approximately 1,675 mm, resulting in an estimated soil evaporation of approximately 1 340 mm ($1675 \text{ mm} \times 0.8$). On this basis, the calculated long-term climatic water balance for the site is strongly negative:

$$B = 493 \text{ mm} - 1340 \text{ mm} = -847 \text{ mm}$$

The Minimum Requirements require that the climatic water balance be evaluated for the wet season of the wettest year on record to determine whether the balance is positive for more than one year in five. Based on available regional climatic records, the water balance for the Maletswai area is negative in most years, indicating that evaporation significantly exceeds rainfall under normal conditions.

Table 6 Climatic Water Balance for the 10 Wettest Years

Year (Wettest Years)	Rainfall(mm) (R)	Evaporation (mm) (E)	Climatic Balance (B)
1966/67	578.3	863.28	-284.98
1973/74	742.8	907.76	-164.96
1975/76	688.3	996.88	-308.58
1987/88	636.9	1156.72	-519.82
2001/02	580.3	1069.04	-488.74
2005/06	578.1	1057.68	-479.58
2009/10	483	1074.4	-591.4
2010/11	591	1024.4	-433.4
2021/22	639.1	1221.36	-582.26
2022/23	492.3	1197.6	-705.3

Where the climatic water balance is negative for more than four out of five years, the landfill is classified as B-, and significant leachate generation is not expected under properly designed and operated conditions. In such cases, the Minimum Requirements indicate that a leachate management system may not be strictly necessary from a climatic perspective alone.

Notwithstanding the B- climatic classification, other site-specific factors must also be considered. These include the moisture content of incoming waste, the potential for stormwater ingress, and the need to protect groundwater resources in accordance with the National Environmental Management: Waste Act. For the Maletswai Landfill, no significant volumes of high-moisture waste are anticipated, and the design includes perimeter diversion drains and surface water management measures to prevent the ingress of upslope runoff onto the waste body. To protect the receiving environment a leachate pollution dam will be constructed to collect all the leachate and also capture the runoff from rainfall.

4.7.4 Other Site-Specific Factors

While the climatic water balance suggests minimal leachate potential, several site-specific considerations must be considered:

- **Moisture content of waste:** The projected waste stream is dominated by household refuse and putrescibles with moderate moisture content. While this may contribute to localized leachate, the overall low rainfall and high evaporation balance will limit accumulation.
- **Ingress of groundwater:** Geotechnical investigations confirm that groundwater levels at the site are deeper than 5–6 m below ground surface, well below the planned excavation depth of 3 m, reducing the likelihood of groundwater inflow into the waste body.
- **Surface water management:** The site is located on gently sloping terrain. If properly engineered, upslope drainage channels, diversion berms, and stormwater cut-offs are installed, stormwater ingress into the landfill cells will be minimal.
- **Operational practices:** Daily and intermediate cover will further reduce infiltration and percolation. Capping closed cells with a low-permeability cover system will also limit rainfall ingress.

Taken together, these factors reinforce the conclusion that the Maletswai landfill has low inherent potential for leachate generation. However, in line with the precautionary principle and the fact that the old Aliwal North landfill has historically impacted groundwater, it is

recommended that the new landfill be constructed with a composite liner system and a leachate detection and collection system. This ensures compliance with the 2024 Norms and Standards, safeguards groundwater resources, and provides assurance to regulators and the community.

5. GEOLOGY, SOILS, GROUNDWATER AND GEOTECHNICAL CONDITIONS

5.1 Introduction

A site-specific geotechnical investigation was undertaken by Earthinv Lab (Report attached) for the proposed Aliwal North Landfill Facility to assess the geological, geotechnical and hydrogeological conditions relevant to the proposed landfill cell, Pollution Control Dam (PCD), access roads, drainage infrastructure and associated site infrastructure.

The investigation comprised field investigations, test-pit profiling, Dynamic Cone Penetrometer (DCP) testing and laboratory testing of representative soil samples. The investigation included test pits distributed across the proposed development area, including the landfill and PCD areas. The test pits were excavated to depths ranging from approximately 1.9 m to 3.0 m below existing ground level. No groundwater seepage or refusal was encountered in the test pits.

The geotechnical investigation provides the principal site-specific basis for the assessment of founding conditions, earthworks, excavation, road subgrade conditions, drainage and landfill liner requirements.

5.2 Regional Geology

The geological mapping for the area indicates that the site is underlain predominantly by Quaternary-age alluvial deposits. The site-specific geotechnical investigation confirms that the materials encountered during the field investigation are consistent with an alluvial depositional environment. The presence of alluvial soils is significant to the design because such materials may contain relatively loose granular horizons and may be susceptible to settlement or collapse when wetted or subjected to changes in loading conditions.

The geotechnical investigation specifically identifies the potential for a collapsible grain structure associated with the alluvial sands. The site is consequently classified as C1 in accordance with NHBRC classification and appropriate precautions are required during development to minimise the potential for structural damage. The geotechnical conditions should therefore not be considered uniformly competent across the site. Foundation preparation, earthworks, drainage and moisture control shall be incorporated into the detailed design and construction methodology.

5.3 Site Investigation

Seven (7) test pits were excavated and profiled by an engineering geologist. The investigation included test pits at representative locations across the proposed development area, including the landfill and PCD areas. The recorded test-pit depths and groundwater observations are summarised below.

Table 7: Summary of Trial Pit and Site Investigation Depths and Groundwater Conditions

Investigation Location	Depth Investigated (m)	Groundwater / Seepage	Refusal
TP1	Approximately 2.7	None encountered	None
TP2	Approximately 2.9	None encountered	None
TP3	Approximately 2.85	None encountered	None
TP4	Approximately 2.9	None encountered	None
Centre	Approximately 2.8	None encountered	None
PCD	3.0	None encountered	None
Dump Waste	Approximately 1.9	None encountered	None

The investigation therefore indicates that no groundwater seepage was encountered within the depth investigated range. The test-pit positions and soil profiles are included in the geotechnical investigation report appendices.

5.4 Soil Profile

The investigation identified transported and pedogenic soil horizons across the site.

5.4.1 Colluvium / Topsoil

Colluvium and topsoil materials were encountered at all test-pit locations to a maximum depth of approximately 0.8 m. The material was generally described as silty sand and clayey sand,

with a medium-dense consistency and an intact structure. These materials represent naturally transported material associated with hill wash and/or gravity-driven deposition. Topsoil and unsuitable organic or waste-containing material shall not be incorporated into structural fills or used as founding material. Topsoil shall be stripped from areas to be developed and stockpiled separately for later rehabilitation and landscaping purposes.

5.4.2 Pedogenic / Calcrete Material

Pedogenic material, including calcrete, was encountered in parts of the investigated area between the transported soils and underlying residual material. The material was generally described as silty sand with a medium-dense to dense consistency and an intact structure. The occurrence and thickness of this material vary across the site. The variability of the pedogenic horizon shall be considered during earthworks and foundation preparation.

5.4.3 General Soil Characteristics

Laboratory testing indicates that the predominant materials comprise silty sands and clayey/silty granular materials. The foundation-indicator testing produced the following representative classifications:

Table 8: Site Material Composition

Location / Sample	Material Description	AASHTO	Unified Classification	TRH14
TP1	Light brown silty sand	A-4	ML	G7
TP2	Dark brown silty sand	A-4	ML	—
TP3	Orange-brown silty sand	A-1-b	CL	G9
Centre	Orange-brown silty sand	A-4	ML	G9
PCD	Orange-brown silty sand	A-4	CL	G10

The laboratory results indicate that the materials range from approximately G7 to G10 and generally have fair to poor subgrade characteristics. The soils have relatively low activity values, indicating a generally low tendency for volume change associated with changes in moisture content and applied loading.

5.5 Soil Plasticity and Activity

The laboratory testing included Atterberg limits and hydrometer analysis.

The measured plasticity indices are generally low, with representative values including:

- TP1: PI = 3%;
- TP3: PI = 6%;
- Centre: PI = 3%; and
- PCD: PI = 6%.

The hydrometer investigation produced low activity values for the tested materials. The PCD sample had an activity of approximately 5.99, while the remaining investigated samples also indicated low activity characteristics as reported in the geotechnical investigation. The low activity of the soils indicates that significant expansive behavior associated with highly active clay minerals is not expected to be the principal geotechnical concern. The more significant consideration for the site is the collapsible nature of the alluvial soil structure, together with variable subgrade strength and moisture sensitivity.

5.6 Collapsible Soil Conditions

The geotechnical investigation identifies the alluvial sands as having a potentially collapsible grain structure. The site is classified as C1 in accordance with NHBRC classification. This condition requires appropriate construction precautions to prevent differential settlement and structural damage. The following measures shall therefore be incorporated into detailed design and construction:

- Removal of unsuitable, loose, organic or waste-containing material;
- Stripping of topsoil from structural areas;
- Controlled moisture conditioning of founding materials;
- Adequate compaction of engineered fill;
- Prevention of uncontrolled water ingress into founding soils;
- Positive surface drainage away from structures;
- Control of leakage from water and wastewater services;
- Proper preparation of building platforms;
- Inspection and approval of founding surfaces before construction; and
- Localised improvement or replacement of unsuitable material where encountered.

No structural foundations shall be placed directly on untreated natural soil unless the founding material has been assessed and approved as suitable for the proposed loading.

5.7 Dynamic Cone Penetrometer Investigation

DCP testing was undertaken adjacent to the test pits to assess in-situ subgrade conditions and provide an indication of the relative strength and bearing capacity of the investigated materials. The DCP results indicate variable but generally stable founding conditions, with an average estimated CBR of approximately 12%. The average calculated bearing capacity from the DCP assessment is approximately 234 kPa.

The individual estimated bearing capacities were:

Table 9: Bearing Capacity estimated on site

Location	DCP CBR (%)	Estimated Bearing Capacity (kPa)
TP1	8.6	153
TP2	13.4	224
TP3	15.6	254
TP4	11.1	193
Centre	8.0	149
PCD	14.0	218
Dump Waste	31.4	449

The relatively high value recorded at the Dump Waste location should be treated with caution because this area contains existing waste-related material and should not automatically be regarded as representative of natural founding conditions.

For structural and minor infrastructure design, a conservative allowable bearing pressure of 150 kPa is adopted, subject to confirmation by the structural engineer and inspection of founding conditions during construction.

5.8 Laboratory Compaction and CBR Results

Laboratory testing was undertaken to establish the compaction characteristics and bearing performance of representative soil materials. The reported maximum dry density values range from approximately 1 743 to 2 023 kg/m³, with optimum moisture contents ranging from approximately 11.9% to 18.1%.

The laboratory CBR results show significant variability:

Compaction Level CBR Range (%)	
93% MDD	2–18
95% MDD	3–22
98% MDD	4–27

The investigation concludes that the tested materials generally have fair to poor subgrade properties and range from G7 to less than G10 according to TRH14 classification.

These results shall be taken into account in the design of:

- Internal access roads;
- Landfill operational roads;
- PCD access;
- Working platforms;
- Parking and turning areas;
- Drainage channels;
- Pipeline trenches; and
- Other trafficked areas.

Where the natural subgrade does not achieve the required design CBR, the material shall be improved, stabilised or replaced with suitable imported material.

5.9 Earthworks and Site Preparation

All earthworks shall be undertaken in accordance with the recommendations of the site-specific geotechnical investigation and the applicable earthworks specifications. The geotechnical investigation recommends that earthworks be undertaken in accordance with **SABS 1200DM**, or the applicable current equivalent.

The following general earthworks requirements shall apply:

1. All vegetation shall be cleared from areas to be developed.
2. Topsoil shall be stripped and stockpiled separately for later rehabilitation.
3. Garden refuse, general waste and other unsuitable fill shall be removed from areas where engineered fills or structures are proposed.

4. Natural founding surfaces shall be inspected and approved before placement of structural fill.
5. Fill shall be placed in controlled layers and compacted to the specified density.
6. Moisture conditions shall be controlled during compaction.
7. Areas affected by uncontrolled water ingress shall be reworked or replaced as necessary.
8. Engineered fill shall comprise approved material suitable for the intended purpose.
9. Earthworks shall be protected from erosion and uncontrolled stormwater.
10. Construction platforms shall be maintained in a dry and stable condition.

The geotechnical investigation specifically recommends removal of unsuitable fill and vegetation and stripping and stockpiling of upper topsoil prior to construction.

5.10 Excavation Conditions

The investigated soil horizons have been assessed as generally soft to intermediate excavation in terms of the applicable South African earthworks excavation classification. The proposed landfill cell and PCD excavation can therefore generally be undertaken using conventional earthmoving equipment, subject to confirmation of actual conditions encountered during construction. Where harder calcrete or cemented horizons are encountered, excavation methods shall be adjusted to suit the actual material.

No assumption of rock excavation is made in the current design because refusal was not encountered in the investigated test pits.

5.11 Trench Stability

The test-pit excavations were undertaken with vertical sidewalls, and no significant sidewall collapse was observed during the investigation. The geotechnical report indicates that trenches excavated within the soil conditions investigated are expected to remain stable under normal conditions. However, instability may occur where:

- Water ingredients occur;

- Excavations remain open for extended periods;
- Heavy equipment is operated too close to excavation edges; or
- Excavations encounter variable or disturbed material.

Where unstable conditions are encountered, trenches shall be battered to stable angles or adequately shored. Excavation safety shall comply with the applicable occupational health and safety requirements.

5.12 Groundwater and Hydrogeological Conditions

No groundwater seepage was encountered during excavation of the seven test pits, including the PCD investigation area. The investigated test pits extended to depths of approximately 1.9–3.0 m below ground level. No perched groundwater was identified during the investigation.

The investigation therefore provides no evidence of shallow groundwater within the investigated depth range. Notwithstanding this finding, groundwater conditions can vary seasonally and spatially. The landfill design shall therefore continue to provide appropriate groundwater protection through:

- Liner containment;
- Leachate collection;
- Controlled stormwater drainage;
- PCD containment;
- Groundwater monitoring; and
- Prevention of uncontrolled discharge.

The absence of groundwater during the investigation shall not be interpreted as proof that groundwater cannot occur beneath the site at greater depth or under different seasonal conditions.

5.13 Surface Water and Drainage Considerations

The geotechnical investigation identifies control and removal of surface water as an important requirement for maintaining site stability. The drainage system shall therefore prevent uncontrolled ponding and infiltration into potentially collapsible alluvial soils.

The landfill development shall incorporate:

- Clean-water diversion drains;
- Controlled dirty-water drainage;
- Adequate falls on platforms;
- Lined or protected drainage channels where required;
- Erosion protection at discharge points;
- Positive drainage away from buildings and foundations; and
- Controlled discharge to approved drainage areas.

The geotechnical report specifically recommends that surface drainage be properly designed and constructed to prevent excessive erosion. This requirement is consistent with the landfill water-management philosophy whereby clean stormwater is separated from contaminated operational runoff.

5.14 Soil Chemical Characteristics

Laboratory testing included pH and electrical conductivity. The tested soil samples generally exhibited pH values in the neutral to mildly alkaline range, with reported values of approximately 8.2–8.7. Electrical conductivity values were reported in the range of approximately 0.004–0.009 S/m. The investigation provides an assessment of soil corrosiveness based on these measurements. These results shall be considered when selecting buried metallic infrastructure and other materials potentially exposed to the soil environment. Where necessary, corrosion protection shall be provided to buried services in accordance with the relevant material and infrastructure specifications.

5.15 Landfill Liner Material Suitability

A critical finding of the site-specific geotechnical investigation is that no clay material suitable for use as a landfill liner was encountered on site. Consequently, the natural site soils shall not be relied upon as the primary low-permeability mineral liner for the landfill cell. The investigation recommends that suitable liner material be imported or that a Geosynthetic Clay Liner (GCL) be used. The proposed landfill containment system shall therefore utilise engineered imported liner material and/or a GCL in combination with the proposed HDPE geomembrane system, subject to the final landfill liner design and applicable waste-disposal requirements.

The liner subgrade shall be carefully prepared to provide:

- A smooth and stable founding surface;
- Removal of protruding stones and unsuitable material;
- Adequate compaction;
- Appropriate grading;
- Protection against differential settlement;
- Suitable support to the GCL and geomembrane; and
- Positive drainage towards the leachate collection system.

The liner installation shall be subject to quality assurance and quality control testing during construction.

5.16 PCD Geotechnical Conditions

The PCD investigation area was specifically assessed through a test pit extending to approximately 3.0 m. The PCD test pit encountered orange-brown silty sand classified as A-4/CL with a TRH14 classification of approximately G10. No groundwater seepage or refusal was encountered. The DCP investigation at the PCD location produced an estimated CBR of approximately 14% and an estimated bearing capacity of approximately 218 kPa.

The PCD foundation shall therefore be prepared by:

- Stripping topsoil and unsuitable material;
- Removing loose or disturbed material;
- Proof-rolling and inspecting the prepared surface;
- Treating weak areas where encountered;
- Providing the specified compacted foundation layer; and
- Constructing the PCD liner system on a properly prepared and approved subgrade.

The PCD shall be designed to prevent seepage and shall incorporate the specified containment liner and protection layers.

5.17 Geotechnical Design Criteria

The following parameters are adopted for preliminary civil and structural design based on the site-specific geotechnical investigation.

Table 10: Geotechnical Design Adoption

Design Parameter	Adopted Value / Condition
Regional geological setting	Quaternary alluvial deposits
Dominant soil	Silty sand / clayey sand
Topsoil/colluvium	Up to approximately 0.8 m
Soil consistency	Medium dense to dense in investigated horizons
Groundwater encountered	None within investigated depths
Investigated depth	Approximately 1.9–3.0 m
Average DCP CBR	Approximately 12%
Average DCP bearing capacity	Approximately 234 kPa
Adopted conservative allowable bearing pressure	150 kPa
Laboratory CBR	Approximately 2–27%, depending on material/compaction
TRH14 material range	Approximately G7 to G10
Soil activity	Low
Soil pH	Approximately 8.2–8.7
Excavation classification	Soft to intermediate
Collapsible soil classification	C1
Natural clay liner suitability	Not suitable / not encountered
Landfill mineral liner	Imported suitable material and/or GCL
Surface drainage	Required to control infiltration and erosion
Foundation requirement	No untreated unsuitable natural soils
Earthworks	Controlled filling and compaction

Trench support	Battering/shoring where required
----------------	----------------------------------

5.18 Geotechnical Implications for the Landfill Design

The site-specific investigation indicates that the site is geotechnically suitable for the proposed landfill development provided that the identified soil conditions are addressed through appropriate engineering controls.

The principal geotechnical considerations are:

1. Alluvial soils: The site is underlain by alluvial deposits rather than competent shallow bedrock.
2. Collapsible conditions: The alluvial sands may exhibit collapse behaviour when wetted, and the site is classified C1.
3. Variable subgrade: The natural materials exhibit variable CBR and bearing capacity characteristics.
4. Groundwater: No groundwater was encountered within the investigated depths.
5. Drainage: Effective surface-water control is required to limit infiltration, erosion and wetting of potentially collapsible soils.
6. Liner materials: Suitable natural clay liner material was not encountered and shall therefore not be relied upon.
7. Imported/GCL liner: The landfill containment system shall incorporate imported suitable liner material and/or GCL together with the proposed geomembrane system.
8. Earthworks: Topsoil and unsuitable material shall be removed and engineered fill shall be placed and compacted under controlled conditions.
9. Roads: The variable and locally poor CBR conditions require appropriate subgrade preparation and imported selected layers where necessary.
10. Foundations: Structural foundations shall be founded on approved competent material or engineered platforms, with a conservative allowable bearing pressure of 150 kPa adopted for preliminary design.

11. Construction control: Geotechnical inspection shall be undertaken during construction to verify actual founding conditions and identify any localised weak, wet, loose or unsuitable materials.

6. LEACHATE GENERATION AND LINER PERFORMANCE ASSESSMENT

6.1 Design Basis for Leachate Management

The preferred containment system for the proposed Maletswai Landfill comprises a composite liner consisting of a Geosynthetic Clay Liner (GCL) overlain by a 2.0 mm textured High-Density Polyethylene (HDPE) geomembrane. The selection of this liner configuration is based on the outcomes of the geotechnical investigation, laboratory permeability testing, environmental risk assessment and the requirements of the National Norms and Standards for Disposal of Waste to Landfill (2024).

Although laboratory testing demonstrated that selected site-derived materials were capable of achieving hydraulic conductivity values ranging between 9.12×10^{-10} m/s and 1.75×10^{-9} m/s under controlled conditions, variability in soil composition across the site raises concerns regarding the ability to consistently achieve the required permeability standard over the entire landfill footprint. Consequently, the GCL-HDPE composite liner was selected as the preferred containment system as it provides a more reliable and robust barrier by combining the self-sealing properties of bentonite with the excellent chemical resistance and durability of HDPE geomembranes.

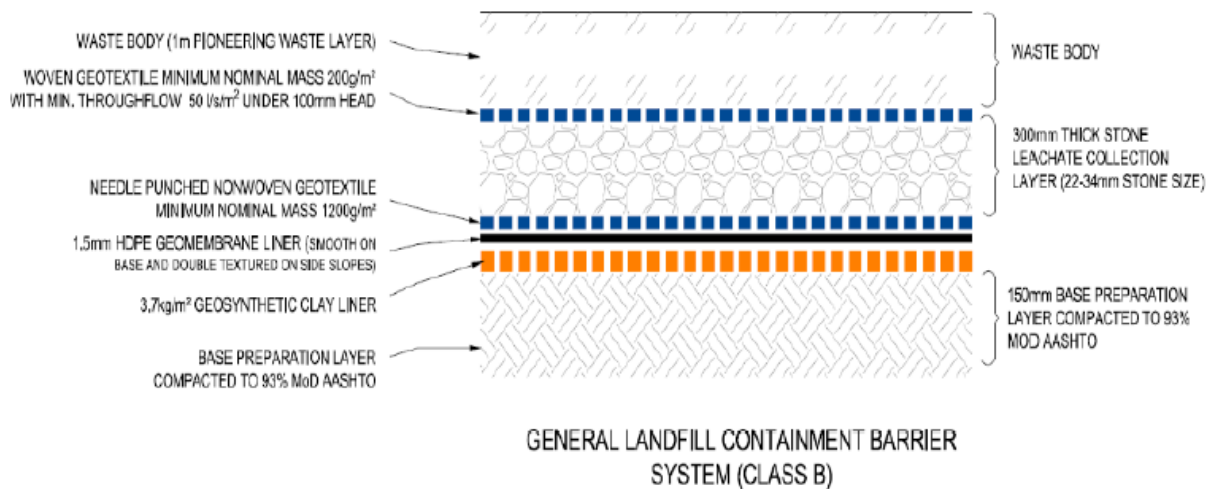


Figure 5: Selected Class B Barrier

For the detailed design purposes, the active landfill cell footprint has been taken as approximately 35 500 m² (3.55 ha). Above the composite liner, a granular drainage blanket and leachate collection system will be installed to ensure that the leachate head on the liner does not exceed 300 mm, thereby reducing hydraulic loading and limiting potential seepage through the containment system.

The leachate management system has been designed to achieve the following objectives:

- Minimise leachate generation through effective stormwater control measures.
- Rapidly collect and remove leachate from the landfill base.
- Maintain leachate heads below 300 mm across the liner surface.
- Prevent contamination of underlying groundwater resources.
- Provide adequate storage, monitoring and operational flexibility.
- Ensure compliance with the National Water Act and National Environmental Management: Waste Act.

6.2. Leakage Assessment of the GCL-HDPE Composite Liner

The strongly negative water balance discussed in section 5.3 indicates that evaporation significantly exceeds rainfall over the long term. As a result, net infiltration into the landfill body

is expected to be relatively low and long-term leachate generation will be limited. Nevertheless, short-duration storm events and operational activities can still result in measurable leachate production and must therefore be accounted for in the design.

For this engineering assessment, the following conservative assumptions were adopted below Table 7:

Table 11: Summary of Design Parameters Adopted for Leachate Generation and Composite Liner Leakage Assessment

Parameter	Value
Cell footprint	35 500 m ²
Mean annual rainfall	493 mm
Maximum leachate head	0.30 m
Defect density	1 defect/ha
GCL hydraulic conductivity	5×10^{-11} m/s
GCL thickness	6 mm
HDPE geomembrane thickness	2.0 mm
Interface transmissivity	1×10^{-6} m ² /s
Wrinkle half-width	0.05 m
Wrinkle half-length	10 m

The principal pathway for leakage through a modern composite liner system is through isolated defects or imperfections within the geomembrane. The magnitude of leakage is influenced by the size and number of defects, the hydraulic head acting on the liner, the transmissivity of the interface between the geomembrane and underlying layer, and the hydraulic conductivity of the secondary barrier.

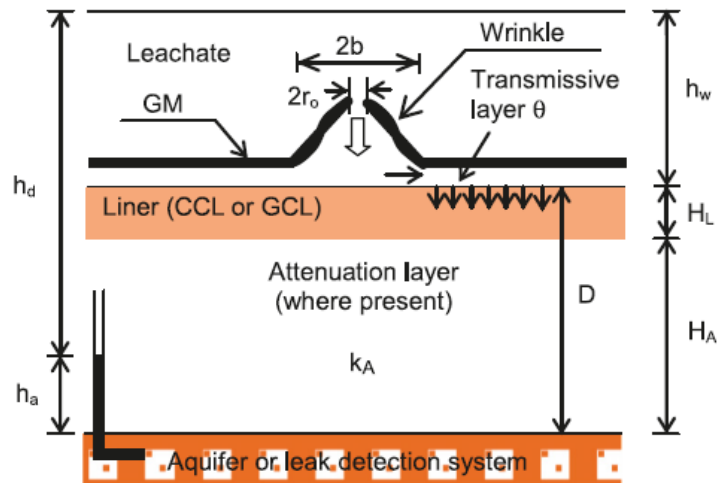


Figure 6: Conceptual representation of leakage through a defect and wrinkle in a composite landfill liner system (Rowe, 2012).

To evaluate potential leakage, the composite liner leakage methodology developed by Rowe (2012) was adopted. This method is widely accepted internationally and provides a realistic assessment of leakage associated with wrinkle-hole interactions within geomembrane systems.

For the selected liner system:

$$D = 0.006 \text{ m}$$

$$\theta = 1 \times 10^{-6} \text{ m}^2/\text{s}$$

The transmissive leakage component is determined as:

$$\sqrt{kD\theta}$$

Substituting the design values:

$$\begin{aligned} & \sqrt{(5 \times 10^{-11})(0.006)(1 \times 10^{-6})} \\ & = 5.48 \times 10^{-10} \end{aligned}$$

The calculated value is substantially lower than that associated with a conventional CCL-HDPE composite liner, confirming the superior hydraulic performance of the selected GCL-HDPE system.

Applying the Rowe model yields an estimated leakage rate of:

$$Q \approx 5.5 \times 10^{-9} \text{ m}^3/\text{s}$$

for a single wrinkle-hole interaction.

Converting this value to a daily leakage rate:

$$\begin{aligned} Q_{day} &= Q \times 86400 \\ Q_{day} &= 0.000475 \text{ m}^3/\text{day} \end{aligned}$$

which is equivalent to approximately:

$$0.48 \text{ L/day}$$

per defect.

Assuming a conservative defect density of one defect per hectare and a landfill footprint of 3.55 ha:

$$\begin{aligned} &0.48 \times 3.55 \\ &= 1.7 \text{ L/day} \end{aligned}$$

Accordingly, the estimated total leakage through the composite liner system is approximately 1.7 litres per day across the entire landfill cell.

6.3 Interpretation of Leakage Results

The leakage assessment demonstrates the excellent containment performance of the selected GCL-HDPE composite liner system. Even under conservative design assumptions, the predicted leakage rate is less than 2 litres per day across the full landfill footprint.

This level of leakage is considered negligible when compared with the anticipated leachate collection volumes generated during routine landfill operations and is well below internationally accepted performance thresholds for engineered landfill facilities.

The assessment confirms that the selected composite liner system provides a highly effective barrier against contaminant migration and will offer long-term protection of underlying soil and groundwater resources throughout both the operational and post-closure phases of the facility.

The results also emphasise the importance of rigorous Construction Quality Assurance (CQA) procedures during installation. Particular attention must be given to:

- Geomembrane welding and seam testing.
- Defect identification and repair.
- Minimisation of geomembrane wrinkles.
- Correct installation of protection geotextiles.
- Proper placement and overlap of GCL panels.
- Preparation and approval of the underlying subgrade.

Provided that these measures are implemented effectively, the predicted leakage rates are considered environmentally insignificant and fully acceptable from a regulatory perspective.

6.4. Gross Leachate Generation from Climatic and Operational Sources

Although the regional climate exhibits a strongly negative water balance, leachate generation cannot be assumed to be negligible. Rainfall falling directly onto active disposal areas, working faces, temporary stockpiles and intermediate cover surfaces will generate contaminated runoff and limited infiltration into the waste mass.

For design purposes, a conservative assumption was adopted whereby 10% of the annual rainfall incident on the active landfill footprint infiltrates the waste body and becomes collectable leachate.

The annual rainfall volume falling on the selected cell is calculated as:

$$V = P \times A$$

Where:

- $P = 0.493 \text{ m/year}$
- $A = 35\,500 \text{ m}^2$

Therefore:

$$\begin{aligned} V &= 0.493 \times 35\,500 \\ V &= 17\,501.5 \text{ m}^3/\text{year} \end{aligned}$$

If 10% of this volume infiltrates and becomes leachate:

$$\begin{aligned} L &= 0.10 \times 17\,501.5 \\ L &= 1\,750 \text{ m}^3/\text{year} \end{aligned}$$

The corresponding average daily leachate generation rate is:

$$\begin{aligned} &\frac{1750}{365} \\ &= 4.8 \text{ m}^3/\text{day} \end{aligned}$$

The leachate collection and storage system should therefore be designed to accommodate a baseline leachate production rate of approximately 5 m³/day under average operating conditions.

In addition to routine leachate generation, adequate contingency storage must be provided to accommodate contaminated stormwater runoff generated during extreme rainfall events. Consequently, the Pollution Control Dam, leachate sumps and associated pumping infrastructure should be designed to safely contain and manage both operational leachate and runoff generated during the selected 1:50-year design storm event.

6.5 Diffusive Transport Through the Composite Liner

Even where no measurable leakage occurs, dissolved contaminants may migrate through liner systems via molecular diffusion. Diffusion occurs when contaminants move from areas of high concentration within the landfill leachate to areas of lower concentration beneath the liner system.

Unlike hydraulic leakage, diffusion is independent of hydraulic head and may occur even when the geomembrane remains intact. The process is governed by Fick's Law, whereby contaminant movement is proportional to the concentration gradient across the barrier.

The HDPE geomembrane provides the primary resistance to diffusion. Research undertaken by Rowe, Sangam, Muller and other international landfill researchers has demonstrated that HDPE geomembranes exhibit extremely low diffusion coefficients for most inorganic contaminants, including heavy metals commonly associated with municipal waste leachate.

The underlying GCL further enhances containment performance by providing:

- Additional contaminant attenuation.
- Sorption of dissolved metals.
- Reduction in contaminant flux.
- Increased contaminant travel times.

The bentonite component within the GCL possesses a high cation exchange capacity, allowing many dissolved contaminants to be adsorbed before reaching underlying groundwater.

For the proposed Maletswai Landfill, the combined HDPE-GCL barrier is expected to reduce contaminant migration to negligible levels for several decades after closure.

6.6. Long-Term Performance of the GCL-HDPE Composite Liner System

a) Hydraulic Performance

The selected containment system consists of a reinforced sodium bentonite Geosynthetic Clay Liner (GCL) overlain by a 2.0 mm textured HDPE geomembrane. The combination of these two components forms a composite barrier system that provides substantially better containment performance than either component operating independently.

The HDPE geomembrane functions as the primary hydraulic barrier and effectively prevents advective flow through the liner system. The underlying GCL acts as a secondary barrier and provides additional protection in the event of defects, punctures or seam imperfections occurring within the geomembrane. The sodium bentonite component of the GCL hydrates upon wetting, producing extremely low hydraulic conductivity values typically ranging between 1×10^{-11} and 5×10^{-11} m/s. This hydraulic performance exceeds the minimum requirements generally specified for compacted clay liners and significantly reduces leakage through localised defects.

b) Diffusive Contaminant Transport

Although hydraulic leakage is often the primary consideration for design, contaminant migration may also occur through diffusion. Diffusion is the movement of dissolved contaminants from areas of high concentration within landfill leachate toward areas of lower concentration beneath the liner system.

The HDPE geomembrane provides the principal resistance to diffusive transport due to its dense polymer structure. In addition, the bentonite layer within the GCL provides adsorption and attenuation mechanisms that further reduce contaminant migration. The bentonite

possesses a high cation exchange capacity and can retain many dissolved contaminants through sorption processes.

Consequently, the composite GCL-HDPE liner substantially increases contaminant travel times and reduces contaminant flux to underlying groundwater. The combined effect of hydraulic containment and diffusive resistance provides long-term protection of groundwater resources beneath the landfill footprint.

c) Chemical Compatibility

The long-term effectiveness of a GCL is influenced by the chemical composition of the leachate to which it is exposed. Municipal solid waste leachate commonly contains elevated concentrations of dissolved salts, ammonia, organic compounds, chlorides, sulphates and trace metals.

The HDPE geomembrane exhibits excellent chemical resistance and is generally unaffected by the constituents typically present in municipal landfill leachate. The GCL component may be affected by cation exchange reactions where calcium and magnesium ions replace sodium ions within the bentonite structure. Excessive cation exchange can reduce bentonite swelling capacity and increase hydraulic conductivity.

However, because the GCL is protected beneath the HDPE geomembrane, direct exposure to aggressive leachate is significantly reduced. For municipal waste applications such as the proposed Maletswai Landfill, long-term chemical degradation of the composite liner system is not anticipated to be significant. Nevertheless, periodic groundwater and leachate monitoring will be undertaken throughout the operational life of the facility to verify containment performance.

d) Service Life of the HDPE Geomembrane

The durability of HDPE geomembranes has been extensively studied internationally. Long-term performance is governed primarily by antioxidant depletion, oxidative induction and eventual polymer degradation.

Modern 2.0 mm HDPE geomembranes manufactured for landfill applications contain stabilisers and antioxidants specifically designed to resist oxidation and chemical attack. When buried beneath waste and protected from ultraviolet radiation, the degradation process occurs very slowly.

Based on published research and international landfill experience, the anticipated service life of a 2.0 mm HDPE geomembrane under municipal waste conditions is typically in excess of 100 years and may exceed 150 years under favourable conditions. The relatively moderate climatic conditions at Maletswai are expected to contribute positively to geomembrane longevity.

e) Service Life of the Geosynthetic Clay Liner

The bentonite contained within the GCL is a naturally occurring mineral with excellent long-term stability. Provided that the GCL remains confined and hydrated beneath the geomembrane, its hydraulic performance is expected to remain largely unchanged over extended periods. International studies indicate that properly installed GCL systems can maintain hydraulic conductivities below 1×10^{-10} m/s for many decades. Because the bentonite layer possesses self-sealing properties, minor punctures or localised defects may be partially mitigated through bentonite swelling. The expected service life of the GCL is therefore considered to exceed the active operating life of the landfill and extend well into the post-closure period.

f) Containment Life of the Composite Liner System

The overall containment life of the landfill is governed by the combined performance of the HDPE geomembrane and GCL. The composite liner provides:

- A primary impermeable barrier through the HDPE geomembrane.

- A secondary low-permeability barrier through the GCL.
- Resistance to diffusive contaminant transport.
- Chemical attenuation and adsorption.
- Self-healing characteristics are associated with bentonite hydration.
- Redundancy against localized defects.

Based on current international experience and published performance assessments, the selected GCL-HDPE composite liner is expected to provide effective containment for a period exceeding 100 years and potentially approaching 150–200 years under municipal landfill conditions. This containment period substantially exceeds the 20-year operational life of the proposed Maletswai Landfill and provides a high degree of confidence in long-term environmental protection.

7. Geotechnical Stability Assessment

The long-term stability of the proposed Maletswai Landfill is a critical design consideration as instability may result in damage to the containment system, failure of waste retention structures, disruption of leachate and stormwater management infrastructure, and potential environmental impacts. A comprehensive geotechnical stability assessment was therefore undertaken to evaluate the suitability of the site for landfill development and to confirm the stability of the proposed landfill geometry under both operational and post-closure conditions.

The assessment incorporates the findings of the geotechnical investigation, laboratory testing, proposed landfill configuration, and anticipated loading conditions. Particular consideration has been given to foundation stability, waste mass stability, liner interface stability, embankment performance, and the long-term stability of stormwater management infrastructure.

The assessment has been undertaken in accordance with accepted geotechnical engineering practice and the requirements of the National Norms and Standards for Disposal of Waste to Landfill (2024).

7.1 Geotechnical Investigation Summary

The geotechnical investigation confirmed that the site is underlain by residual silty sands, sandy silts and weathered mudstones associated with the Beaufort Group. Competent founding conditions occur across the proposed landfill footprint, with no evidence of weak compressible soils, expansive clays or organic deposits that could adversely affect landfill performance.

Groundwater was not encountered within the proposed excavation depth and is expected to occur at depths greater than 5 m below ground level. As the proposed landfill excavation depth is approximately 3 m, the landfill base will remain above the regional groundwater table throughout construction and operation.

The geological and geotechnical conditions are therefore considered favourable for landfill development and provide a suitable foundation for the proposed containment system and associated infrastructure.

7.2 Laboratory Shear Strength Testing

Direct shear testing was undertaken on representative soil samples collected from various locations across the site. The testing was conducted to determine the effective shear strength parameters required for stability assessments.

The laboratory results are summarised in Table 8.

Table 12 Geotechnical Stability Properties Insitu Material

Sample Location	Cohesion, c' (kPa)	Friction Angle, ϕ' (°)
TP03	9	39.2
Centre	1.9	44.3
Pollution Control Dam Area	32.1	39.9

The measured friction angles are significantly higher than those typically assumed during preliminary landfill design, where values between 28° and 35° are commonly adopted. These

results indicate excellent shear resistance and confirm the competency of the residual soils and weathered bedrock underlying the proposed development.

For design purposes, the following conservative characteristic parameters were adopted:

Table 13 Design Values Adopted

Parameter	Design Value
Effective Cohesion (c')	9 kPa
Effective Friction Angle (ϕ')	39°
Bulk Unit Weight (γ)	18 kN/m ³
Saturated Unit Weight (γ_{sat})	20 kN/m ³
Allowable Bearing Capacity	150 kPa

These values table 9 represent lower-bound design parameters and provide a conservative basis for stability evaluation.

7.2 Foundation Stability Assessment

The proposed landfill will be founded on competent residual soils and weathered Beaufort Group sediments. The absence of weak or compressible strata, coupled with the relatively deep groundwater table, significantly reduces the potential for foundation-related instability.

The adopted bearing capacity of 150 kPa exceeds the anticipated stresses imposed by waste placement, perimeter berms, landfill infrastructure and Pollution Control Dam embankments. Furthermore, the high friction angles obtained from laboratory testing indicate excellent resistance to shear failure within the founding materials.

Based on the geotechnical investigation and laboratory results, the risk of foundation bearing failure, excessive settlement or basal instability is considered negligible.

7.3 Foundation Stability Assessment

The proposed landfill side slopes have been designed at a maximum gradient of 1 Vertical to 3 Horizontal, equivalent to an angle of approximately 18.4°.

The measured soil friction angles ranging from 39.2° to 44.3° provide a substantial margin of safety against rotational and translational failure.

A simplified assessment of slope stability may be expressed as:

$$FoS = \frac{\tan \phi'}{\tan \beta}$$

Where:

- ϕ' = Effective friction angle
- β = Slope angle

Using the adopted design friction angle of 39° and a slope angle of 18.4°:

$$FoS = \frac{\tan 39^\circ}{\tan 18.4^\circ}$$
$$FoS = 2.44$$

This value significantly exceeds the minimum long-term factor of safety of 1.5 generally required for landfill facilities and demonstrates that the proposed slope geometry possesses substantial stability reserves. The assessment confirms that the proposed landfill side slopes are suitable for both operational and post-closure conditions.

7.4 Waste Mass Stability

The stability of the proposed municipal solid waste (MSW) mass was assessed using limit equilibrium slope stability modelling. The analysis considered the proposed landfill geometry, waste loading conditions, foundation materials and anticipated operational filling configuration.

Municipal solid waste exhibits frictional behaviour similar to granular materials and develops shear strength through a combination of particle interlocking, reinforcement effects associated with fibrous waste components and compaction achieved during landfill operations. Based on published landfill engineering literature, municipal solid waste typically exhibits effective friction angles ranging from approximately 25° to 35°, depending on waste composition, density, moisture content and degree of degradation.

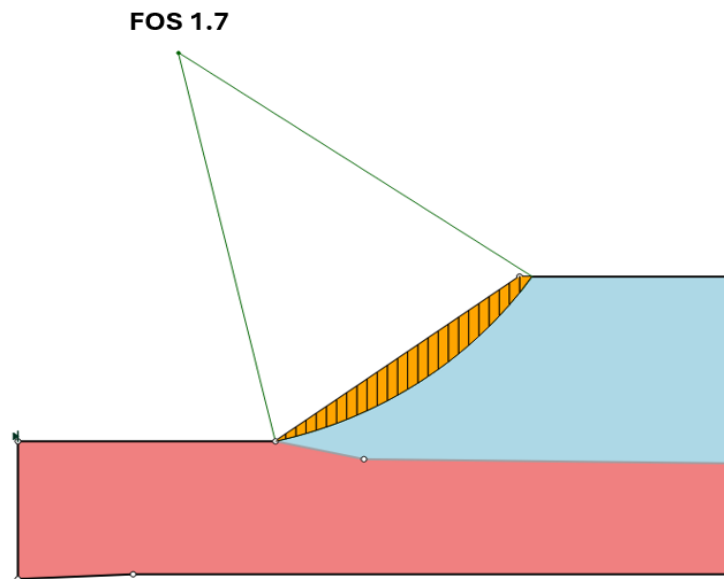


Figure 7 Waste Stability Results

For the stability assessment, conservative municipal solid waste strength parameters were adopted together with the site specific geotechnical properties obtained from the foundation investigation. The analysis incorporated the proposed waste slope geometry and evaluated potential rotational failure mechanisms extending through the waste mass and into the underlying foundation soils.

The critical failure surface identified by the stability model is shown in Figure 9. The calculated minimum Factor of Safety was:

$$FoS=1.7$$

This value significantly exceeds the minimum long-term static stability criterion of:

$$FoS \geq 1.5$$

commonly adopted for landfill facilities.

The high factor of safety reflects several favourable site conditions, including:

- Competent residual soils and weathered Beaufort Group sediments underlying the landfill footprint.
- Measured foundation soil friction angles ranging between 39.2° and 44.3°.
- Relatively flat landfill side slope geometry.
- Progressive placement and compaction of waste during operations.
- Absence of shallow groundwater beneath the landfill footprint.
- Limited leachate head maintained through the engineered leachate collection system.

The analysis demonstrates that the proposed landfill configuration possesses substantial resistance to rotational failure and that instability of the waste mass is unlikely under normal operating conditions.

Furthermore, the calculated factor of safety provides a considerable margin above regulatory and industry requirements, indicating that the proposed landfill geometry is suitable for both operational and long-term post-closure conditions.

Overall, the waste mass stability assessment confirms that the proposed landfill design is geotechnically stable and capable of safely accommodating the planned waste volumes throughout the design life of the facility.

7.5 Composite Liner Interface Stability

The stability of the composite liner system is critical to the long-term performance of the landfill. The proposed containment system comprises a prepared subgrade, reinforced Geosynthetic Clay Liner (GCL), 2.0 mm textured HDPE geomembrane, protection geotextile and leachate drainage layer.

The use of a textured HDPE geomembrane substantially increases interface friction compared to smooth geomembranes and improves resistance to translational sliding. The competent foundation soils further contribute to liner stability by providing a stable and uniform support platform.

Based on published interface shear strengths for textured HDPE-GCL systems and the measured site-specific soil properties, the potential for liner interface failure is considered low. The proposed liner configuration is therefore regarded as suitable for the planned landfill geometry.

7.6 Pollution Control Dam Stability

The Pollution Control Dam forms a critical component of the leachate and contaminated stormwater management system. Stability of the embankments is therefore essential for environmental protection.

The direct shear test results obtained from the PCD area indicate a cohesion value of 32.1 kPa and a friction angle of 39.9°, representing excellent embankment construction material.

The proposed embankments will be constructed using engineered fill compacted to the specified density requirements and will not exceed side slopes of 1 Vertical to 3 Horizontal. Under these conditions, the risk of embankment instability is considered very low.

7.7 Stormwater Infrastructure Stability

Stormwater diversion channels, berms and associated drainage structures will be constructed using compacted site materials. The measured friction angles indicate excellent resistance to erosion and side slope failure.

Where concentrated flows are anticipated, appropriate erosion protection measures including riprap, gabions, Reno mattresses and energy dissipators will be incorporated into the design.

The proposed stormwater infrastructure is therefore expected to remain stable throughout both the operational and post-closure phases of the landfill.

7.8 Long-Term Post-Closure Stability

Following closure, the landfill will be capped with a final cover system designed to minimize infiltration and promote runoff. The combination of favorable geotechnical conditions, a high-performance GCL-HDPE composite liner system, low regional rainfall and high evaporation rates provide favorable conditions for long-term stability.

Although gradual settlement of the waste mass will occur as decomposition progresses, such settlement is expected to occur uniformly and is unlikely to compromise the integrity of the containment system, stormwater controls or rehabilitation measures.

Long-term stability of the landfill is therefore considered achievable provided that construction and operational activities are undertaken in accordance with the approved design.

7.9 Engineering Conclusion

The geotechnical stability assessment confirms that the proposed Maletswai Landfill will be founded on competent residual soils and weathered Beaufort Group sediments possessing favourable engineering characteristics.

Laboratory testing indicates friction angles ranging from 39.2° to 44.3° and cohesion values ranging from 1.9 kPa to 32.1 kPa, demonstrating excellent shear strength properties and providing strong support for the proposed landfill development.

The assessment confirms:

- Adequate foundation stability.

- Stable landfill side slopes.
- Acceptable waste mass stability.
- Favourable liner interface stability.
- Stable Pollution Control Dam embankments.
- Stable stormwater management infrastructure.
- Compliance with accepted landfill engineering safety requirements.

No significant geotechnical constraints have been identified that would preclude development of the proposed landfill, and the site is considered suitable for safe operation throughout the 20-year design life and subsequent post-closure monitoring period.

8.1 Landfill Design Philosophy

8.1.1 General Approach

The design adopts a phased vertical development strategy within a single landfill footprint. This ensures that the facility is developed incrementally, with each 5-year vertical phase constructed, filled, and temporarily capped before the next lift is developed Figure 10 A.



Figure 8A Selected Site Layout and infrastructure

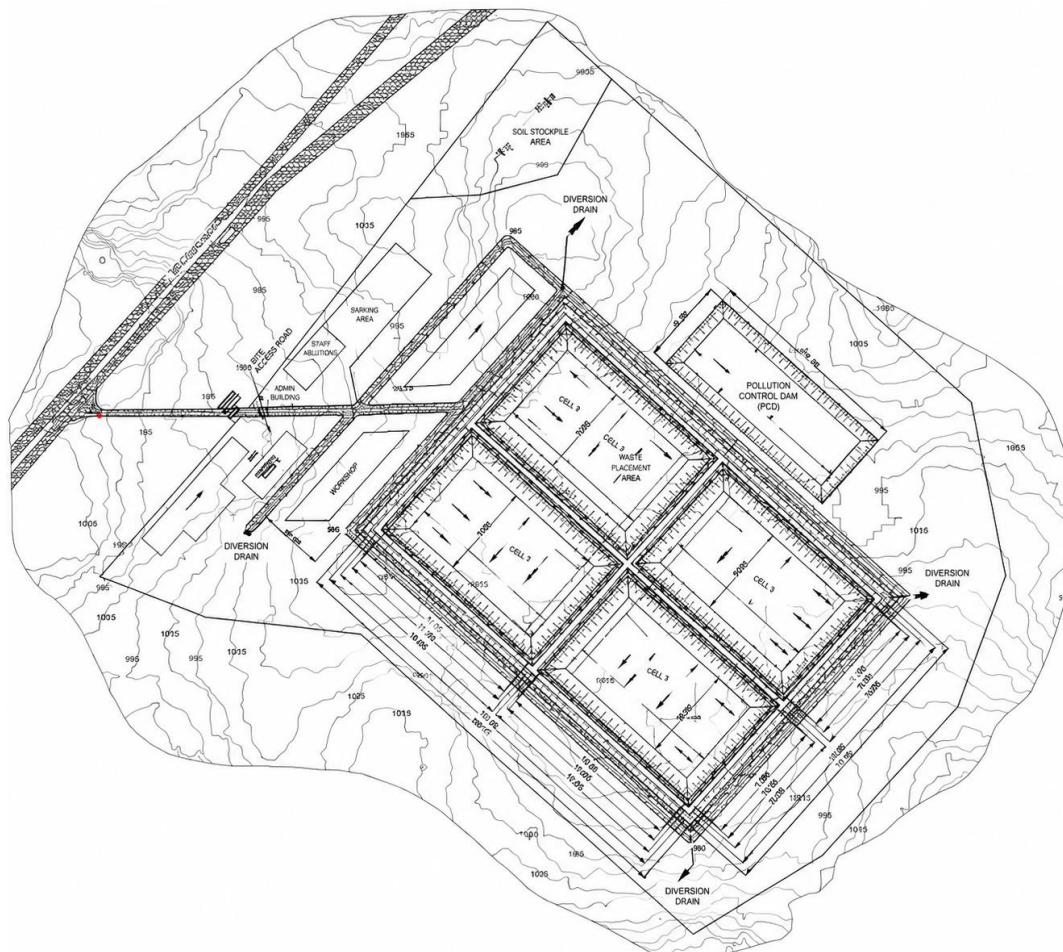


Figure 9 B Selected Site Layout and infrastructure

This approach minimizes the exposed tipping area, reduces rainfall infiltration, and allows for early environmental protection through progressive rehabilitation.

8.1.2 Capping of Existing Waste Cells

The existing Aliwal North landfill, once decommissioned, will be closed and capped using a final cover system consisting of GCL, a geomembrane, a drainage layer, and a vegetative soil cover. This intervention will reduce infiltration, improve environmental compliance, and allow for integration of the old site into the new facility's operational framework.

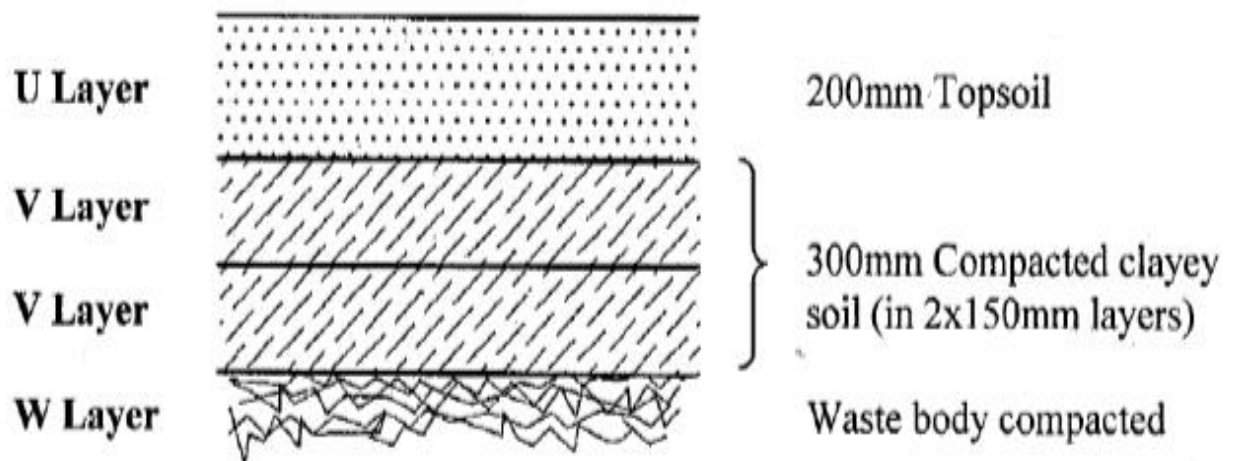


Figure 10: Proposed Capping Layer

8.1.3 Landfill Gas Generation and Management

Landfill gas (LFG) is generated through the anaerobic biodegradation of organic waste within the landfill body, producing primarily methane (CH_4) and carbon dioxide (CO_2), with trace quantities of volatile organic compounds and hydrogen sulphide (IPCC, 2006). The rate and quantity of gas generation vary throughout the landfill's lifespan and are influenced by several factors including waste type, waste depth, moisture content, degree of compaction, pH, temperature, and time since deposition (EPA, 2003). Due to these interacting variables, accurate prediction of gas quantities carries a high level of uncertainty and must be refined through ongoing field monitoring and data evaluation during landfill operation (DWAF, 1998).

Models such as the IPCC first-order decay model and USEPA LandGEM can be applied to estimate gas generation potential at the design stage. These estimates should later be validated through gas sampling, pumping trials, and subsurface investigations once waste placement commences (USEPA, 2005). A number of approaches exist for estimating LFG production, including rule-of-thumb methods, field pumping tests, and computer-based modelling.

According to the EPA Landfill Site Design Guide (Gregory et al., 1991; EPA, 2003), as a rule of thumb, every ton of degradable waste can produce approximately 6 m^3 of landfill gas per year

for up to ten years after deposition, under typical conditions. Under optimum conditions with high moisture content, one ton of degradable waste can theoretically generate 400–500 m³ of landfill gas, although practical yields are much lower often 200 m³ or less per wet ton of municipal waste.

A comprehensive survey of 262 landfill sites conducted by Gregory et al. (1991) reported an average cumulative landfill gas yield of 146 m³/t (wet waste), of which approximately 70% could be recovered for utilization, equating to an achievable yield of about 102 m³/t over the life of a landfill. However, in semi-arid regions such as Maletswai, the actual gas yield is typically less than 10% of these values, owing to limited waste moisture and a negative climatic water balance (DWAF, 1998; Jones & Wagener, 2010).

Based on these findings and applying the IPCC (2006) methodology for the Maletswai site, the theoretical methane generation potential is approximately 270,000 m³ CH₄ per year, but due to the semi-arid climate and small operational scale, the effective gas generation rate is expected to range between 27,000 m³ and 54,000 m³ per year. This equates to about 37–75 m³/day of methane, confirming that active extraction, flaring, or energy recovery systems are not technically or economically warranted.

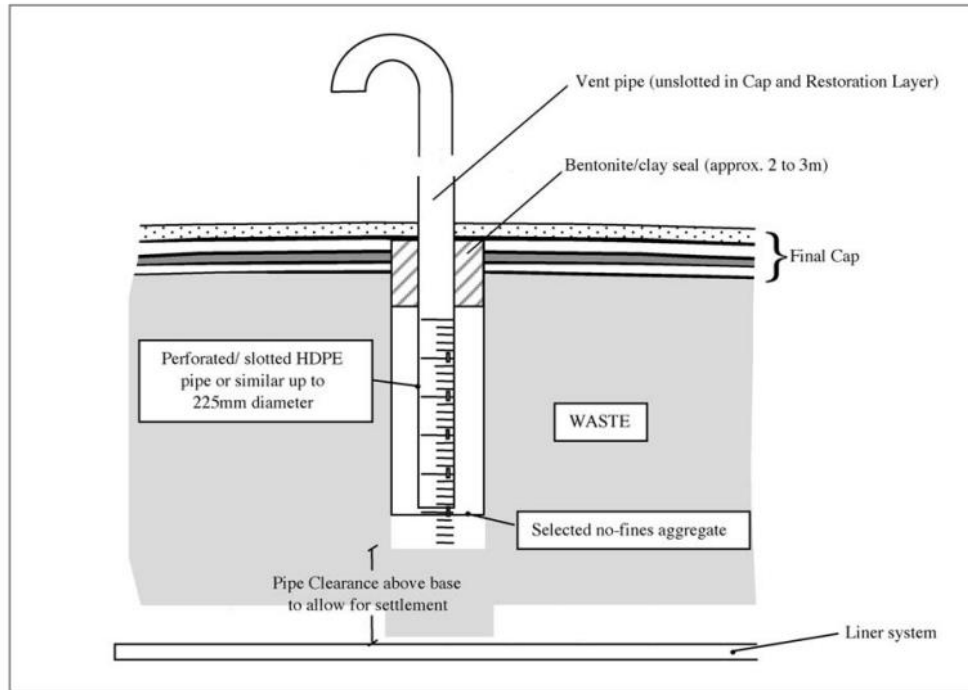


Figure 11: Passive gas control system to atmosphere (EPA, 2003)

The recommended management approach is passive gas control figure 7, allowing natural venting through the working face during early operations. As final cover systems are installed based on figure 6, gas venting infrastructure comprising vertical vent pipes connected to subsurface perforated collection trenches will be implemented to prevent gas build-up. To enhance passive ventilation, “whirly bird” ventilators may be installed on selected vents. These low-cost systems use wind-driven rotation to draw gases upward while carbon or biofilters remove odorous compounds before release (Koerner & Koerner, 2012).

Routine gas monitoring will form part of the environmental management program. Quarterly sampling will be undertaken from perimeter wells and vent pipes to verify methane and carbon dioxide concentrations and to ensure compliance with the National Environmental Management: Air Quality Act (Act No. 39 of 2004) and the 2024 National Norms and Standards for Disposal of Waste to Landfill (DFFE, 2024).

In conclusion, the Maletswai Landfill is expected to generate low volumes of landfill gas due to its small size, dry climatic setting, and reduced organic fraction. Passive venting, supported by

simple mechanical ventilators and odour filtration, provides a safe, cost-effective, and environmentally compliant solution for landfill gas management (2024).

8.1.4 Phase 1 Development

Phase 1 figure 10 selected layout will comprise the first 5-year vertical lift within the designated footprint, reaching an effective filling height of approximately 3–4 m. This phase will be constructed immediately after licensing, with all supporting infrastructure (weighbridge, access road, leachate lagoon, stormwater system, and staff facilities) established concurrently. Phase 1 will provide 15 000 m³ of airspace, sufficient for the first five years of operation.

8.1.5 Long-term Development Plan

Subsequent phases will be developed upward in 5-year increments, progressively raising the landfill height to the final 13 m design limit by Year 20. Each completed phase will be capped with an interim cover to limit infiltration and nuisance, while leachate management systems will remain active across all phases. The final closure will include a composite capping system with geomembrane and vegetation, returning the site to a stable and rehabilitated landform. The design also provides flexibility, allowing for potential vertical extension or transfer of waste to regional facilities beyond the 20-year planning horizon.

8.2 Supporting Infrastructure and Services

8.2.1 Site Access and Roads

The landfill will be accessed via an upgraded gravel access road designed for 20–25 tons compactor and haul trucks, with appropriate drainage and erosion control. Internal roads will run along the perimeter of the footprint and provide all-weather access to the active tipping face, constructed from compacted gravel with a wearing course. Roads will be 6 m wide with adequate turning radii for compaction and haulage equipment. Stormwater side drains will be provided to prevent ponding and minimize maintenance requirements.

8.2.2. Weighbridge Facility

A weighbridge and control office will be located at the site entrance to record all incoming waste loads, supporting billing, municipal reporting, and compliance with the National Waste Information Regulations. The weighbridge system will include electronic record-keeping, ensuring accurate monitoring of waste inflows and available airspace utilization.

8.2.3. Laboratory and Testing Facilities

A small on-site laboratory will be established for quality control of construction materials such as clay liners and cover soils, including routine compaction, density, and moisture testing. Limited leachate quality tests may also be conducted. More complex chemical analyses of leachate and groundwater will be outsourced to accredited laboratories.

8.2.4 Perimeter Security (Fencing)

The site will be enclosed by a 2.4 m high steel palisade or chain-link fence topped with barbed wire, with a controlled access gate at the entrance. Security will include guard facilities and lighting to prevent unauthorised access, illegal dumping, or scavenging.

8.2.5 Water Supply and Wastewater Management

8.2.5.1 Design Basis

The water supply and wastewater management systems for the proposed Maletswai Landfill Facility have been designed to provide a reliable, hygienic and environmentally appropriate service for the anticipated site population and associated landfill operations. The water-services design is based on a maximum operational population of 50 persons, comprising landfill operators, administrative personnel, security personnel, maintenance personnel, contractors and visitors.

The design has been developed with reference to the Neighbourhood Planning and Design Guide (Red Book), applicable South African National Standards, the National Water Act, 1998

(Act 36 of 1998), the Water Services Act, 1997 (Act 108 of 1997), applicable municipal requirements and relevant environmental requirements.

The proposed facility will rely primarily on groundwater abstraction from the available production borehole infrastructure. The borehole investigation records reviewed for the site indicate that the boreholes are approximately 60–62 m deep and have demonstrated groundwater availability at substantially greater depths than the proposed landfill excavation. Borehole No. 1 recorded water strikes at approximately 12 m, 20 m and 24 m, with additional geological formations encountered at greater depth, while the borehole completion information indicates a total drilled depth of approximately 62 m. The recorded pump yield for Borehole No. 1 was approximately 3,500 L/h.

The borehole test records also indicate that Borehole No. 2 is approximately 60 m deep and produced a lower yield, with the test documentation recording an allowed pumping rate of approximately 500 L/h and a pump yield of approximately 750 L/h. The difference in borehole yields demonstrates the importance of confirming the sustainable abstraction rate for the selected production borehole before finalising the water-supply system.

The proposed water-services system will comprise groundwater abstraction, borehole pumping, on-site storage, potable-water treatment where required, potable and non-potable distribution, a dedicated fire-water reserve and a separate domestic wastewater-management system.

The principal design criteria are summarised in Table 10

Table 14: Water and Waste Design Criteria

Parameter	Adopted Design Criterion
Maximum design population	50 persons
Domestic water demand	70 L/person/day
Staff/operational allowance	20 L/person/day
Base water demand	90 L/person/day
Water-loss/design allowance	15%

Parameter	Adopted Design Criterion
Adopted average daily demand	5.18 m ³ /day
Normal storage period	Minimum 2 days
Normal storage requirement	10.35 m ³
Preliminary fire-water reserve	9.0 m ³
Total calculated storage requirement	19.35 m ³
Proposed storage capacity	20 m ³
Storage configuration	2 × 10 m ³ tanks
Primary water source	Groundwater production borehole
Potable water	Treated groundwater where required
Design wastewater flow	Approximately 4.7 m ³ /day
Preliminary septic-tank capacity	10 m ³
Septic-tank configuration	Two compartment
Wastewater disposal	Controlled subsurface infiltration
Domestic wastewater/leachate	Completely separate systems

The above criteria provide the preliminary design basis. Final infrastructure sizing shall be confirmed following final selection of the production borehole, confirmation of the sustainable borehole yield, water-quality assessment, fire-risk assessment, infiltration testing and final site layout.

8.2.5.2 Groundwater Supply

Groundwater will constitute the primary water source for the proposed landfill facility. The available borehole investigation records provide site-specific evidence that groundwater can be abstracted from the underlying aquifer.

The borehole completion information indicates that Borehole No. 1 was drilled to approximately 62 m and encountered water-bearing zones during drilling. Water strikes were recorded at approximately 12 m, 20 m and 24 m, with the drilling log also recording sandstone, mudstone

and doleritic formations at greater depths. The borehole was subsequently tested, with the documentation recording an airlift yield of more than 4,000 L/h and a pump yield of approximately 3,500 L/h. The pumping-test information provides further evidence of the borehole's production capability. The test records show pumping at varying discharge rates and corresponding changes in drawdown, followed by recovery monitoring. The records indicate good recovery following the test.

Borehole No. 2 is also approximately 60 m deep; however, the available test record indicates a considerably lower production capability. The record identifies an allowed pumping rate of approximately 500 L/h and a pump yield of approximately 750 L/h. The borehole results therefore indicate that the groundwater resource available at the site is variable between boreholes. The final water-supply design shall consequently be based on the borehole selected for production and its confirmed sustainable yield rather than relying solely on the instantaneous yield recorded during drilling or short-duration testing.

The required average site water demand is approximately 5.18 m³/day. This is equivalent to approximately 5,175 L/day. Even the lower recorded borehole pumping capacity of approximately 750 L/h would theoretically provide approximately 6,000 L over an eight-hour pumping period; however, this comparison is only indicative and does not establish the sustainable abstraction capacity of the aquifer. The final pumping regime shall therefore be determined from the hydrogeological assessment and pumping-test results.

For Borehole No. 1, the recorded pump yield of approximately 3,500 L/h provides substantial capacity relative to the calculated average domestic demand. At this rate, approximately 5.18 m³ of water could theoretically be abstracted in approximately 1.5 hours of pumping. The borehole does not, however, need to operate continuously at this rate because the proposed storage tanks will allow water abstraction to be scheduled according to demand and borehole recovery.

The final production pumping rate shall be selected conservatively and shall take account of drawdown, recovery, aquifer response, seasonal groundwater conditions, pump performance

and the requirements of the relevant water-use authorization production borehole shall be located outside the active landfill footprint and appropriately separated from the landfill cell, leachate collection system, Pollution Control Dam, septic tank and effluent disposal area. The borehole installation shall include suitable casing and screen arrangements, sanitary headworks, submersible pump, rising main, non-return valve, isolation valve, flow meter, water-level monitoring point, pump controls, electrical protection and appropriate security. The borehole shall be metered to enable groundwater abstraction to be recorded. Static and pumping water levels shall also be monitored periodically to identify any sustained decline in groundwater levels.

8.2.5.3 Water Quality

The suitability of the groundwater for potable use shall be established through laboratory water-quality testing before the water is supplied for human consumption. The borehole drilling information provides preliminary observations of groundwater quality. Borehole No. 1 is recorded as having an apparent water quality described as “Good”, while Borehole No. 2 is recorded as having poorer apparent quality. These observations shall not, however, be used as a substitute for laboratory analysis. The final water-quality assessment shall determine whether treatment is required for potable use. Depending on the analytical results, treatment may include filtration, disinfection or other appropriate treatment processes.

Potable water supplied to drinking points, kitchen facilities and ablution facilities shall comply with the applicable drinking-water quality requirements SANS 241. Where groundwater quality is unsuitable for potable use without treatment, the treatment system shall be designed according to the actual water-quality results. Routine water-quality monitoring shall be implemented during operation to identify changes in groundwater quality.

8.2.5.4 Water Demand

The water demand has been calculated for a maximum operational population of 50 persons.

A domestic water demand of 70 L/person/day is adopted for drinking, ablution, handwashing, kitchen use and other normal domestic requirements. For 50 persons, this corresponds to a domestic demand of 3,500 L/day, or 3.50 m³/day. An additional allowance of 20 L/person/day is provided for minor operational requirements associated with staff facilities, cleaning and related activities. For 50 persons, this corresponds to 1,000 L/day, or 1.00 m³/day. The resulting base demand is therefore 4.50 m³/day. A 15% allowance is then applied to account for minor system losses, leakage, operational variation and uncertainty. The resulting design demand is approximately 5.18 m³/day.

The adopted average daily water demand is therefore **5.2 m³/day**, equivalent to approximately **5.2 kL/day**.

Table 15: Water Demand for domestic

Water-Demand Component	Design Basis	Demand
Domestic demand	50 persons at 70 L/person/day	3.50 m ³ /day
Staff/operational allowance	50 persons at 20 L/person/day	1.00 m ³ /day
Base demand	Domestic + operational	4.50 m ³ /day
Design allowance	15%	0.675 m ³ /day
Total design demand	Base + allowance	5.175 m ³ /day
Adopted design demand	Rounded	5.2 m³/day

The calculated demand represents normal domestic and minor operational requirements. Significant bulk water requirements, particularly dust suppression and road wetting, are excluded from the above value. Dust suppression requirements shall be assessed separately based on road length, traffic intensity, climatic conditions and the required frequency of application. Where significant volumes are required, operational water should preferably be supplied through a separate non-potable storage system.

8.2.5.5 Water Storage

On-site water storage is required because the facility will depend on groundwater abstraction. Storage provides operational resilience during periods of borehole maintenance, electrical interruptions, pump failure, reduced groundwater yield and temporary restrictions on abstraction. A minimum normal storage period of two days is adopted. Based on the design demand of approximately 5.175 m³/day, the normal operational storage requirement is approximately 10.35 m³.

A preliminary fire-water reserve of 9.0 m³ is provided in addition to the normal operational storage. The resulting total calculated storage requirement is approximately 19.35 m³.

A total storage capacity of 20 m³ is therefore proposed, comprising two 10 m³ tanks.

Table 16: Storage Requirements

Storage Component	Volume
Normal operational storage	10.35 m ³
Fire-water reserve	9.00 m ³
Total calculated requirement	19.35 m ³
Proposed storage	20.00 m ³
Tank arrangement	2 × 10 m ³

The storage tanks shall be configured to ensure that the 9 m³ fire reserve remains protected from normal domestic consumption. With 20 m³ total storage and 9 m³ reserved for fire-fighting, approximately 11 m³ is available for normal use. At the adopted average daily demand of 5.175 m³/day, this represents approximately 2.1 days of normal operational storage.

Each tank shall be fitted with a lockable cover, screened vent, overflow, drain/scour connection, isolation valve, level indicator and appropriate level-control equipment. High- and low-level alarms shall be incorporated into the pump-control system. The overflow shall be directed to a

safe drainage location and shall not be discharged into the landfill cell, leachate system or Pollution Control Dam.

8.2.5.6 Potable and Non-Potable Water Separation

The water-services system shall distinguish between potable water intended for human consumption and non-potable water intended for operational purposes.

Potable water shall be supplied to drinking points, kitchen and canteen facilities, handwashing basins, ablution facilities and other locations requiring water of potable quality. Non-potable water may be used for dust suppression, road wetting, construction activities, cover-soil conditioning, vehicle washing and other operational activities where potable quality is not required.

Where non-potable water is provided, the system shall be clearly labelled and physically separated from the potable system. Appropriate backflow prevention shall be provided to prevent accidental cross-connection. This separation is particularly important for the landfill facility because dust suppression may represent a significantly greater water requirement than domestic consumption. Such operational water should not compromise the availability of potable water or the dedicated fire reserve.

8.2.5.6 Wastewater Generation

Domestic wastewater will be generated from the administration facilities, ablution facilities, kitchen/canteen facilities, handwashing facilities and associated domestic activities. The domestic wastewater system shall remain completely independent from the landfill leachate and contaminated-stormwater systems. The 90% of the adopted water demand is assumed to return to the wastewater system. Based on the design water demand of approximately 5.175 m³/day, the resulting wastewater generation is approximately 4.66 m³/day.

The adopted design wastewater flow is therefore 4.7 m³/day.

Table 17: Wastewater Desing Values

Parameter	Adopted Value
Design population	50 persons
Design water demand	5.175 m ³ /day
Wastewater return factor	90%
Calculated wastewater flow	4.66 m ³ /day
Adopted wastewater flow	4.7 m³/day

The above wastewater flow excludes water used for activities that do not discharge to the domestic wastewater system, such as dust suppression and road wetting. The internal drainage system shall also consider peak instantaneous flows resulting from simultaneous use of toilets, wash basins, showers and other plumbing fixtures.

8.2.5.7 Septic Tank

In the absence of a municipal sewer connection, domestic wastewater shall be treated through a dedicated septic tank Figure 14 followed by sealed wastewater storage and off-site disposal by an approved vacuum tanker. No soakaway, infiltration trench, French drain, subsurface infiltration system or other ground-disposal system shall be permitted.

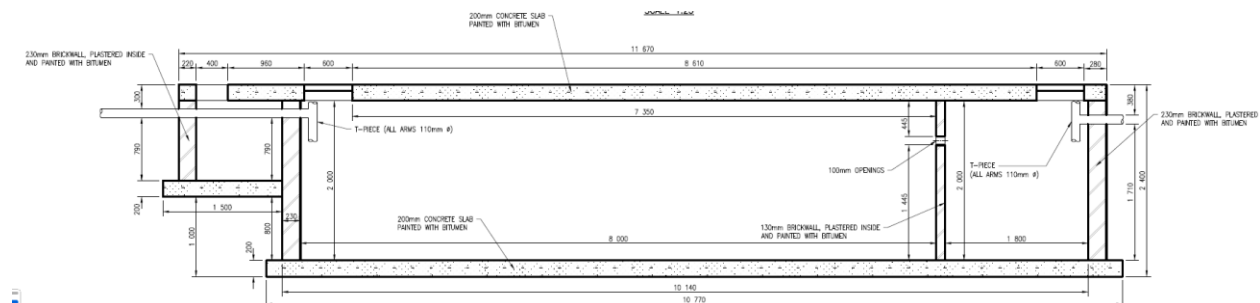


Figure 12: Proposed Septic Tank

The adopted design population is 50 persons, with an average wastewater flow of approximately 4.7 m³/day. A minimum preliminary liquid-retention period of 24 hours is adopted for the septic tank, resulting in a minimum hydraulic treatment volume of

approximately 4.7 m³. The hydraulic treatment volume alone does not provide adequate allowance for sludge and scum accumulation. A larger tank capacity is therefore required to maintain effective treatment conditions during operation.

A 10 m³ septic tank is consequently adopted for preliminary design. A two-compartment arrangement is recommended. The first compartment shall comprise approximately two-thirds of the total capacity, while the second compartment shall comprise approximately one-third. For the proposed 10 m³ tank, this corresponds to approximately 6.7 m³ in the first compartment and 3.3 m³ in the second compartment.

Table 18: Wastewater Design Criteria

Parameter	Adopted Design Criterion
Design population	50 persons
Average wastewater flow	4.7 m ³ /day
Preliminary retention period	24 hours
Minimum hydraulic volume	4.7 m ³
Adopted septic-tank capacity	10 m ³
Number of compartments	2
First compartment	Approximately 6.7 m ³
Second compartment	Approximately 3.3 m ³
Sludge/scum allowance	Included within tank capacity
Construction	Watertight
Access	Lockable inspection/desludging covers
Desludging	Approved vacuum tanker
Septic-tank desludging interval	Maximum 3 months initially
Final effluent	Sealed holding storage
Ground disposal	Not permitted

The three-month period shall be treated as the maximum initial septic-tank desludging interval. The actual desludging frequency shall be reviewed during operation based on measured sludge and scum accumulation. The septic tank shall be watertight and shall be located outside the active landfill footprint and outside the Pollution Control Dam footprint. The final tank dimensions shall be confirmed during detailed design and shall consider structural requirements, liquid depth, freeboard, inlet and outlet levels, sludge and scum storage, vehicle loading and access for desludging.

8.2.5.8 Sealed Wastewater Effluent Storage

As subsurface disposal of domestic wastewater is not permitted, effluent from the septic tank shall be conveyed to a dedicated, watertight and sealed wastewater holding facility. The wastewater holding facility shall be sized to provide approximately three months of storage before scheduled removal by an approved vacuum tanker.

At the adopted average wastewater generation rate of approximately 4.7 m³/day, approximately 423 m³ of wastewater will be generated over a 90-day period. A nominal 450 m³ sealed wastewater holding capacity is therefore adopted for preliminary design. The 450 m³ capacity provides approximately 96 days of nominal storage at the adopted average wastewater flow and provides a limited operational margin above the three-month storage requirement.

Table 19: Wastewater Design Criteria

Parameter	Adopted Design Criterion
Design population	50 persons
Average wastewater flow	4.7 m ³ /day
Target storage period	3 months / approximately 90 days
Wastewater generated over 3 months	Approximately 423 m ³
Adopted sealed storage capacity	450 m ³
Nominal storage duration	Approximately 96 days
Disposal method	Approved vacuum tanker
Subsurface disposal	Not permitted
Soakaway	Not permitted
Overflow	Not permitted
Construction	Watertight and sealed
Level monitoring	Required
High-level alarm	Required
Tanker access	Required

The final storage arrangement may comprise a single suitably designed tank or multiple interconnected tanks, provided that the total effective storage capacity is not less than the required design capacity. The holding facility shall be designed as a fully contained system and shall prevent leakage to the surrounding soil and groundwater.

The facility shall be incorporated:

- Watertight construction;
- Sealed joints and pipe penetrations;
- Suitable access and inspection covers;
- Level monitoring;
- High-level alarm;
- Suitable tanker connection points;
- Safe access for vacuum tankers;
- Adequate freeboard; and
- Emergency response provisions.

No overflow pipe, soakaway outlet, perforated pipe, infiltration trench or other uncontrolled discharge arrangements shall be provided. The wastewater holding facility shall be located outside the active landfill footprint and outside the Pollution Control Dam footprint. The final location shall also allow safe access for vacuum tankers without interfering with landfill operations, waste placement or contaminated-water management infrastructure.

8.5.9 Wastewater Collection, Emptying and Disposal

All effluent from the septic tank shall be retained within the sealed wastewater management system until removed by an appropriately authorised vacuum tanker. The initial operating requirement shall be for the sealed wastewater storage facility to be emptied at intervals of approximately three months, subject to actual wastewater generation and available storage capacity. The operator shall arrange tanker collection before the available storage capacity is exhausted.

Earlier collection shall be undertaken where:

- Wastewater generation exceeds the adopted average;
- The high-level alarm is activated;
- Available storage is approaching the maximum operating level;

- Tanker collection is delayed;
- The septic tank requires earlier desludging; or
- Any abnormal operating condition occurs.

Wastewater and septic sludge shall be transported by an appropriately authorised service provider to an appropriately authorized wastewater-treatment or disposal facility. Domestic wastewater shall not be discharged to:

- The surrounding ground;
- A soakaway;
- An infiltration trench;
- A French drain;
- A stormwater drain;
- The Pollution Control Dam;
- The landfill cell;
- The leachate collection system;
- Natural drainage channels; or
- Any other uncontrolled disposal area.

Records shall be maintained for all wastewater and septic-sludge removals, including the date of collection, volume removed, service provider and disposal destination.

8.2.5.10 Septic-Tank Operation and Maintenance

The septic tank and sealed wastewater holding facility shall be incorporated into the facility's routine operation, maintenance and environmental-management programme.

Routine inspections shall assess:

- Sludge accumulation;
- Scum accumulation;
- Liquid levels;

-
- Inlet and outlet conditions;
 - Tank integrity;
 - Pipework condition;
 - Wastewater holding-tank level;
 - High-level alarm;
 - Access covers; and
 - Vacuum-tanker connection points.

The septic tank shall initially be inspected and desludged at intervals not exceeding three months. The desludging frequency shall subsequently be reviewed based on actual operating conditions and measured sludge accumulation. The sealed wastewater holding facility shall be monitored regularly to ensure sufficient storage remains available for normal operation and to provide adequate warning before the maximum storage level is reached.

At a minimum, the following measures shall be implemented:

- Routine inspection of the septic tank;
- Monitoring of sludge and scum levels;
- Inspection of inlet and outlet structures;
- Inspection of wastewater transfer pipework;
- Monitoring of sealed holding-tank levels;
- Testing of the high-level alarm;
- Inspection for leakage;
- Removal of blockages;
- Prevention of excessive discharge of oils, solvents and harmful chemicals into the domestic wastewater system;
- Periodic septic-tank desludging by an appropriately authorised service provider; and
- Removal of stored wastewater by an appropriately authorised vacuum tanker.

Septic sludge and domestic wastewater shall not be discharged into the landfill cell, Pollution Control Dam, stormwater system, leachate collection system or any other uncontrolled disposal

area. Records of inspections, maintenance, desludging, wastewater removal and final disposal shall be maintained as part of the facility's environmental-management records.

8.2.5.11 Separation of Domestic Wastewater, Stormwater and Leachate

The domestic wastewater system shall be separate from the landfill stormwater and leachate-management systems.

The landfill drainage philosophy is based on maintaining separation between clean stormwater, contaminated stormwater, leachate and domestic wastewater. This reduces the potential for cross-contamination and prevents unnecessary increases in the volume of water requiring contaminated-water management.

The principal water streams shall be managed as follows:

Table 20: Water Streams and Their Management Systems at the Maletswai Landfill Facility

Water Stream	Source	Management System
Potable water	Treated groundwater	Potable storage and distribution
Domestic water	Borehole/storage	Offices, ablutions, kitchen and hygiene
Operational water	Non-potable source/storage	Dust suppression and operational activities
Domestic wastewater	Offices and ablutions	Septic tank and sealed wastewater storage
Clean stormwater	Undisturbed/upgradient areas	Diversion drains/channels
Contaminated stormwater	Active landfill areas	Contaminated-water collection and PCD
Leachate	Waste mass	Leachate collection and PCD
Fire water	Dedicated storage reserve	Fire-fighting system

Domestic wastewater shall not be discharged to the Pollution Control Dam. Likewise, landfill leachate and contaminated stormwater shall not be discharged to the septic tank or domestic wastewater storage system. Clean stormwater originating outside operational landfill areas shall be diverted around the facility where practicable. Stormwater generated within the active

landfill and other potentially contaminated areas shall be collected and directed to the designated pollution control dam

The Pollution Control Dam shall remain dedicated to contaminated water generated by landfill activities, while the septic tank and sealed wastewater storage system shall remain dedicated to domestic wastewater. Maintaining this separation is essential because the introduction of domestic wastewater into the PCD would unnecessarily increase both the hydraulic volume and organic loading of the contaminated-water system. Conversely, the introduction of landfill leachate into the domestic wastewater system could compromise septic treatment and increase environmental risk.

8.2.6 Electricity Provision

Electricity will be supplied by a connection to the municipal distribution grid. Power will serve the weighbridge, leachate pumping systems, offices, and security installations. A standby diesel generator will provide backup to ensure uninterrupted operation of critical systems during load shedding or power outages.

8.2.7 Staff and Administrative Facilities

Administrative and support facilities will include a control office, staff ablution block, waste area, changing rooms, and a small meeting room. These will be located near the entrance for easy access and oversight of incoming vehicles and operations. The layout will be designed to separate staff areas from the active landfill footprint for safety.

8.2.8 Plant and Equipment Maintenance

A dedicated maintenance yard and workshop will be provided for daily servicing of landfill equipment such as compactors, dozers, and loaders. A fuel storage facility, compliant with SANS standards, will be established for refueling operations. Parking bays and a washing area will be included to ensure plant readiness and minimize breakdown

9. OPERATIONAL STRATEGY

9.1 Overview

The operational strategy for the Maletswai Landfill is designed to ensure that the facility is managed in compliance with the National Environmental Management: Waste Act (Act 59 of 2008) and the 2024 Norms and Standards for Disposal of Waste to Landfill. Operations will focus on minimizing environmental impacts, maintaining health and safety, and extending landfill life through best-practice waste placement, monitoring, and progressive rehabilitation.

9.2 Access Control

Access to the site will be strictly controlled. A single gated entry point with a weighbridge, security office, and guard presence will regulate incoming vehicles. Only authorised waste collection vehicles and contractors will be permitted. A visitors' logbook and vehicle checks will be maintained to prevent illegal dumping and ensure waste loads meet acceptance criteria.

9.3 Waste Control and Monitoring

9.3.1 Waste Pickers and Informal Reclaimers

Waste picking will not be permitted on active disposal cells due to safety risks. Recycling initiatives will be formalized at a dedicated Materials Recovery Facility (MRF), where reclaimers can operate under municipal management.

9.3.2 Acceptance Criteria

Only general waste consistent with the site's permit classification of Class B will be accepted. Hazardous, infectious, or industrial waste requiring special treatment will be prohibited. Random load inspections will ensure compliance.

9.3.3 Record Keeping

All waste entering the site will be weighed and recorded. Records will include vehicle ID, origin, waste type, and tonnage. This information will feed into the South African Waste Information System (SAWIS) and annual reporting to the regulator.

9.3.4 Auditing Requirements

Quarterly internal audits and annual external audits will verify compliance with licence conditions. These audits will review operational records, monitoring data, and environmental management practices.

9.3.5 Gas Monitoring Systems

Although climatic conditions suggest limited gas generation, perimeter gas monitoring wells will be installed to detect methane and carbon dioxide migration. Monitoring will be conducted quarterly, with results included in compliance reports.

9.4 Water Quality Monitoring Program

9.4.1 Baseline Conditions

Baseline water quality will be established prior to operation through sampling of groundwater, surface water, and leachate.

9.4.2 Surface Water Monitoring

Upstream and downstream points along nearby drainage lines will be sampled bi-annually for pH, EC, nitrates, and heavy metals.

9.4.3 Groundwater Monitoring

At least three boreholes (one upgradient, two downgradient) will be monitored quarterly. Parameters will include major ions, heavy metals, and microbiological indicators.

9.4.4 Leachate and Contaminated Water Monitoring

Leachate volumes will be measured daily, with quarterly laboratory analysis for chemical composition. Any contaminated runoff from the site will be managed through lined stormwater ponds.

9.4.5 Reporting Requirements

All water quality results will be compiled into a bi-annual monitoring report submitted to the Department of Forestry, Fisheries and the Environment (DFFE) and shared with the Walter Sisulu Municipality Council.

9.4.6 Recycling Initiatives

The landfill will integrate recycling by establishing a small-scale MRF near the entrance. Materials such as plastics, cardboard, glass, and metals will be separated and diverted from landfill disposal. Partnerships with local cooperatives will enhance job creation while reducing landfill airspace demand.

9.5 Waste Deposition Operations

9.5.1 Cell Operation Procedures

Waste will be placed in thin layers (≤ 0.5 m) and compacted using landfill compactors to achieve high density (0.8 t/m^3). The working face will be minimized to reduce windblown litter and infiltration.

9.5.2 Daily and Intermediate Cover

A minimum of 150 mm of soil cover will be applied daily, and 300 mm of intermediate cover will be placed on inactive areas to limit smell, vermin, and fire risk.

9.5.3 Wet Weather Operating Cell

A dedicated wet-weather operating cell shall be provided to maintain safe and continuous landfill operations during periods of heavy rainfall when normal landfill working areas may become saturated, unstable or inaccessible. The wet-weather cell shall be located and prepared to provide a stable working platform and shall be connected to the internal landfill access-road network.

The cell shall be provided with a suitably compacted gravel wearing course and all-weather access surfacing designed to accommodate the anticipated operational traffic under wet

conditions. The operating platform shall be maintained in a condition that allows waste collection and placement vehicles, compaction equipment and other essential landfill equipment to access the working area safely during periods of adverse weather.

The wet-weather cell shall be operated as a temporary working area when rainfall or saturated ground conditions prevent effective operation of the normal active cell. Waste shall be placed and compacted in controlled lifts within the designated wet-weather operating area, with the working face maintained to the minimum practical size to limit exposure of waste to rainfall.

Surface runoff from the wet-weather operating area shall be controlled and prevented from mixing with clean stormwater. Water generated within the active waste-placement area shall be managed as contaminated water and directed to the designated leachate and contaminated-water management system.

The wet-weather cell shall be inspected following significant rainfall events to assess the condition of the access surface, working platform, drainage infrastructure and surrounding areas. Any rutting, erosion, ponding or loss of gravel material shall be repaired before the area is returned to normal operation.

The provision of the wet-weather operating cell will improve operational resilience during periods of high rainfall, reduce the risk of vehicles becoming immobilised, maintain waste-placement and compaction activities, and minimise uncontrolled movement of contaminated water from the active landfill area.

9.5.4 Site Drainage and Run-off Control

Stormwater diversion berms and cut-off drains will be constructed around active cells to prevent surface water ingress. Collected runoff will be channeled to lined attenuation ponds before controlled discharge.

9.6 Resource Requirements

9.6.1 Plant and Machinery

Essential equipment includes a landfill compactor, bulldozer, front-end loader, and water cart for dust suppression. Leachate pumping equipment and tankers will also be maintained.

9.6.2 Staffing Requirements

The site will be managed by a Landfill Supervisor, supported by machine operators, weighbridge clerks, environmental monitoring staff, and security personnel. Training in occupational health, waste handling, and emergency response will be mandatory.

9.6.3 Nuisance Control (Noise, Dust, Odor, Vermin)

Dust will be controlled with water carts and vegetative cover on inactive areas. Odors will be reduced by rapid covering of waste. Litter fences will capture windblown debris, and vector control measures (trapping and baiting) will be applied. Operating hours will be limited to daylight to reduce noise impacts on nearby communities.

9.6.4 Implementation Schedule

Phase 1 infrastructure and the first cell will be constructed immediately after licensing, with operations commencing within 12 months. Cells will be filled sequentially at five-year intervals, with monitoring and rehabilitation implemented progressively.

9.6.5 Rehabilitation Program

Rehabilitation will be progressive, with final cover applied to each cell upon completion. The final cap will consist of a compacted clay or GCL layer, geomembrane, drainage layer, and vegetative soil cover. Vegetation will stabilize slopes and integrate the site into the surrounding landscape.

9.6.6 Closure Plan

Closure will involve decommissioning infrastructure, final capping of all cells, groundwater monitoring for a minimum of five years, and conversion of the site into a controlled green

space. The plan will be refined closer to the end of life to incorporate updated regulatory requirements and available technologies.

9.7 Discussion

The operational strategy for Maletswai Landfill demonstrates a balance between engineering practicality, environmental protection, and socio-economic benefits. With structured access control, careful monitoring, and phased development, the site can operate safely for its 20-year design life. Recycling initiatives and progressive rehabilitation will further enhance sustainability, while continuous auditing and reporting will ensure compliance with the regulatory framework.

10 CONCLUSION & RECOMMENDATION

The preliminary design for the licensing of a new landfill site in Maletswai, Eastern Cape, is an important response to the growing waste management needs area. The existing landfill, primarily used for domestic waste disposal, has reached its airspace capacity, and it is in the process of closure. Additionally, due to the current poor waste management practices, such as burning, not covering the waste, which are practiced at the landfill site, they are outdated and environmentally unsafe.

The proposed landfill design considers legislative requirements and strives to optimize waste management operations, ensuring compliance with environmental regulations. The proposed landfill is classified as a Class B Liner Landfill Site and "G: S: B-" based on waste type, size, and potential climatic water balance. It falls into the "Small" size class with a calculated Maximum Rate of Deposition (MRD) of 66.1 tons per day. The landfill's calculated airspace capacity and lifespan are estimated, with approximately 20 years of operational life anticipated.

The design incorporates liner and capping systems, considering the absence of natural clay capping material, and relies on geosynthetic clay liners (GCLs). The final capping of the landfill will involve topsoil planted with local vegetation. Effective leachate management is emphasized, with a system of perforated pipes and layers for leachate collection, valves, and emergency

overflow mechanisms. Continuous monitoring and sampling of leachate are crucial for quality assessment.

Surface water management strategies are in place to prevent surface water ingress, erosion, and on-site flooding, including stormwater cut-off trenches. In conclusion, the proposed new landfill site aligns with environmental regulations, addresses waste disposal needs, and is designed for long-term viability, considering factors such as infrastructure, future growth, and environmental impact mitigation.

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ANNEXURE CONSTRUCTION QUALITY ASSURANCE (CQA) PLAN

1 INTRODUCTION

This Construction Quality Assurance (CQA) Plan defines the procedures and requirements for the inspection, monitoring, testing, and documentation of construction activities associated with the pollution control barrier system at the Maletswai Landfill Facility.

The CQA Plan ensures that all construction works are undertaken in accordance with the approved Construction Drawings, Technical Specifications, and applicable regulatory requirements, including the National Environmental Management: Waste Act (NEMWA, 2013) and relevant South African National Standards (SANS 10409, SANS 1200, and SANS 1526).

The objective of this CQA Plan is to provide independent verification that construction materials, installation procedures, and workmanship comply with the design intent and required engineering performance standards.

Specifically, this plan establishes:

- Roles and responsibilities of all parties involved in the CQA process
- Qualification requirements for CQA personnel
- Inspection and testing procedures
- Sampling and laboratory testing requirements
- Material approval processes
- Documentation and reporting protocols
- Procedures for addressing non-conformance

The CQA process is independent of Quality Control (QC), which remains the responsibility of Contractors, Manufacturers, and Installers.

2.2. PARTIES AND RESPONSIBILITIES

The Owner and license holder of the facility is the Maletswai Local Municipality (or appointed entity).

The Owner is responsible for:

- Appointment of consultants, contractors, and CQA personnel
- Overall project governance and regulatory compliance

2.1 Owner/Operator

The Project Manager shall act as the official representative of the Owner/Employer and will be responsible for the overall coordination, management, and supervision of construction activities at the facility. This includes full oversight of site operations, as well as the coordination of both construction and Construction Quality Assurance (CQA) activities. The Project Manager shall ensure that all work programmes submitted by the various Contractors are effectively integrated and aligned with the overall project schedule, and that linkages between different work streams are properly managed.

The Project Manager shall further be responsible for the resolution of all quality assurance issues that arise during the construction of the barrier system and shall be involved in any decisions that may affect the long-term performance and operation of the facility. In this regard, the Project Manager plays a critical role in ensuring that construction activities are carried out in accordance with the approved design, specifications, and regulatory requirements.

For the purposes of this CQA Plan and the Project Specifications, the “Project Manager” shall refer to the individual or entity identified as such on the title and signatory page of this document. The Owner/Employer shall appoint a suitably qualified and experienced Project Manager with a Civil Engineering background and relevant experience in managing similar infrastructure or waste management projects.

The Project Manager shall also be responsible for managing and monitoring project schedules using appropriate project management tools. The use of Microsoft Project shall be a minimum requirement,

while the use of additional tools such as Asana, or equivalent platforms, is recommended to enhance coordination, communication, and task management across all parties involved in the project.

2.2 Project Manager

Responsible for coordination, scheduling, and resolving construction issues.

2.3 Design Engineer

The Design Engineer, also referred to as the “Designer” or “Engineer,” shall be the individual or organisation responsible for developing the design and preparing the Construction Drawings and Specifications for the project. The Design Engineer shall ensure that all designs meet applicable regulatory requirements, relevant standards, and accepted engineering practices. In addition, the Design Engineer shall review and approve any design changes, amendments, or clarifications that may arise during construction to ensure that the integrity and performance of the facility are not compromised.

For the purposes of this CQA Plan and the Project Specifications, the term “Design Engineer” shall refer to the party identified as such on the title and signatory page of this document. A suitably qualified consulting engineering firm, registered with Consulting Engineers South Africa (CESA), shall be appointed by the Employer to undertake this role. The appointed Design Engineer may also fulfil the role of CQA Engineer and Resident Engineer for the project.

A designated Engineer’s Representative shall be present on site on a full-time basis throughout the construction period. This will enable continuous supervision of the works, ensure adherence to design requirements and specifications, and support effective day-to-day management of construction activities.

A Resident Engineer will be stationed on site for the duration of construction.

2.4 CQA Engineer and Monitors

The CQA Engineer and CQA Monitor(s) shall be responsible for the implementation of this Construction Quality Assurance (CQA) Plan, including all required testing, monitoring, inspection, documentation, and reporting activities. They shall ensure that all construction works are carried out in accordance with the approved Construction Drawings, Specifications, and applicable regulatory requirements. The CQA Engineer shall act as the client representative and shall be responsible for reviewing and certifying the final construction report.

All CQA activities, including implementation and reporting, shall be conducted under the direct supervision of a professional engineer or technologist registered with the Engineering Council of South Africa (ECSA) in the civil engineering discipline. For the purposes of this CQA Plan and the Project Specifications, the term “CQA Monitor” shall refer to the individual or entity identified as such on the title and signatory page of this document.

The Engineer’s Site Representative, who may also serve as the CQA Engineer, shall be stationed on site on a full-time basis and shall report to the Design Engineer for verification. This representative shall be responsible for overseeing and coordinating all quality assurance activities on site. This includes supervising testing conducted by the Contractor and Geosynthetics Installer, as well as independent third-party laboratory testing. In addition, the CQA Engineer shall be responsible for maintaining site diaries, recording inspections, reviewing contractor reports and logs, overseeing the collection and transportation of samples, and reviewing laboratory test results to ensure compliance with project requirements.

2.5 Geosynthetics Manufacturer

The geosynthetics manufacturer(s), hereafter referred to as the “Manufacturer,” shall be responsible for the production and supply of all geosynthetic components specified in this CQA Plan. In accordance with the Competition Act (Act 89 of 1998), the Manufacturer shall operate independently from the Geosynthetics Installer unless otherwise approved. Each Manufacturer shall be required to pre-qualify

and demonstrate the capability to supply materials that fully comply with the Project Specifications, as outlined in the relevant appendices and certified construction drawings.

The Manufacturer shall be appointed by the Employer through a formal tender process and shall provide all necessary documentation, guarantees, sureties, and evidence of suitably qualified personnel to ensure that the works are carried out in a professional and timely manner. Certificates of conformance shall be provided for all geomembrane rolls delivered to site to confirm compliance with the required quality standards.

All geomembrane materials shall strictly comply with the requirements of SANS 1526:2015 and relevant international standards such as GRI-GM13 (as amended). The proposed liner supplier's details shall be submitted as part of the tender documentation, and any subsequent changes to the approved supplier shall require prior written approval from the Design Engineer.

The geomembrane shall be manufactured in the largest practical sheet sizes to minimise the number of joints, with a minimum seam-free width of 5 metres. The production process shall be carefully controlled to ensure consistent, high-quality sheets that are homogeneous and free from defects such as blisters, bubbles, pinholes, or irregular edges. Each batch of material shall be supported by production data and test reports, with certificates of conformance issued at a minimum frequency of once per production shift to demonstrate compliance with the specified standards.

2.6 Geosynthetics Installer

The Geosynthetics Installer, also referred to as the "Geosynthetics Installation Contractor" or "Installer," shall be responsible for the proper installation of all geosynthetic components, including the HDPE geomembrane and associated geotextiles, in accordance with the approved Construction Drawings and Project Specifications. In line with the requirements of the Competition Act (Act 89 of 1998), the Installer shall operate independently from the Manufacturer unless otherwise approved.

The Installer shall be appointed by the Employer through a formal tender process and must pre-qualify by demonstrating compliance with the requirements outlined in the Construction Specifications and

Tender Documents. As part of the tender submission, the Installer shall provide details of suitably qualified personnel, including a Supervisor or Construction Manager who will be responsible for providing full-time technical oversight and guidance to the installation team. This representative shall attend all site meetings and act as the official spokesperson for the Installer throughout the project.

All welding technicians employed by the Installer shall be appropriately trained and certified by recognised industry bodies such as the International Association of Geosynthetic Installers (IAGI), the TRI (Austin, Texas), or an equivalent approved authority. The competency and performance of welding personnel shall be subject to ongoing evaluation, and the CQA Engineer, through the Project Manager, shall reserve the right to reject any technician whose performance is deemed unsatisfactory.

The Installer shall ensure that all installation activities are carried out in strict accordance with the specified requirements, maintaining high standards of workmanship to ensure the integrity and long-term performance of the liner system.

2.7 Civil/Earthworks Contractor

The Civil/Earthworks Contractor shall be responsible for the execution of all earthworks, civil works, pipework, and concrete works in accordance with the requirements of **SANS 1200** and the approved Construction Drawings and Specifications. All construction activities shall be carried out to a standard that ensures compatibility and proper interface with the liner system, including the HDPE geomembrane and any Geosynthetic Clay Liners (GCLs).

Compaction of the Pollution Control Dam (PCD) embankment shall be undertaken in layers not exceeding 300 mm in thickness, using a pad-foot vibrating roller, and shall achieve a minimum of 95% Standard Proctor Density ($\pm 2\%$), at a moisture content within the range of 0% to +3% of Optimum Moisture Content (OMC). The compacted clay liner shall be constructed in two separate layers of 150 mm each, resulting in a total compacted thickness of 300 mm, to ensure the required level of impermeability.

The Civil Contractor shall be appointed by the Employer through a formal tender process and shall be responsible for delivering all works in accordance with the contract specifications. As part of the contractual requirements, the Contractor shall provide appropriate sureties and guarantees, as well as a comprehensive Occupational Health and Safety Plan, Environmental Management Plan, detailed construction programme, and cash flow projections. In addition, the Contractor shall submit the qualifications and curriculum vitae of key site personnel, together with a list of all plant and equipment to be utilised on the project.

The Contractor shall also be responsible for appointing a qualified surveyor to establish and control all required levels, grades, and alignments for excavation and construction works, as well as to prepare accurate as-built drawings. All surveying activities shall be conducted under the supervision of a registered Professional Surveyor. The accuracy of levels and grades shall be verified by the Resident Engineer or an independent surveyor to ensure compliance with the design requirements.

2.8 Independent CQA Laboratory

The Construction Quality Assurance (CQA) Laboratory shall be independently appointed by the Employer and shall operate under the direction and management of the CQA Engineer. The laboratory shall function as an impartial entity and shall have no affiliation with the Civil Contractor, Geosynthetics Installer, Manufacturer, or any material suppliers involved in the project.

The laboratory responsible for geosynthetics testing shall be appropriately accredited, preferably through the Geosynthetics Research Institute Laboratory Accreditation Program (GRI-LAP) or an equivalent recognised accreditation body. The independence of the CQA Laboratory is essential to ensure unbiased testing, verification of material compliance, and the integrity of all quality assurance data generated during construction.

3. MEETINGS

Meetings shall be held during the construction of the project to enhance coordination among the various parties involved. Meetings will include a pre-construction meeting, progress meetings, and

resolution meetings if necessary

3.1 Pre-Construction Meeting

allowing the evaluation of tenders and the formal award of contracts to the appointed Contractors and Geosynthetics Installer/Manufacturer, a pre-construction meeting shall be convened on site prior to the commencement of any construction activities. Attendance at this meeting shall be mandatory for all key project stakeholders, including the Design/CQA Engineer and CQA Monitor, Project Manager, Contractor, Geosynthetics Installer/Manufacturer, Owner (Employer), and any other parties designated by the Owner.

The primary purpose of the pre-construction meeting is to ensure that all parties have a clear understanding of the Construction Quality Assurance (CQA) Plan, project requirements, and their respective roles and responsibilities. The meeting shall address and clarify, inter alia, the responsibilities and authority of each party, established communication channels, and coordination procedures.

A site inspection shall be conducted to familiarise all parties with the project layout, including the location of construction works, access routes, laydown and stockpiling areas, site offices, borrow areas (if applicable), and environmentally sensitive or restricted “no-go” areas such as wetlands and watercourses. In addition, the project schedule and individual work programmes submitted by the Contractor and Geosynthetics Installer shall be reviewed and aligned, and construction battery limits shall be clearly defined.

The meeting shall further include a review of the Construction Drawings, Specifications, applicable testing standards, and the CQA Plan, as well as the Occupational Health and Safety (OHS) Plan and the Environmental Management Plan (EMP).

The proceedings of the pre-construction meeting shall be formally recorded by the CQA Engineer, and minutes of the meeting shall be distributed to all attendees for record and reference.

3.2 Progress Meetings

Weekly progress meetings shall be held throughout the construction period to ensure effective coordination and communication between all parties involved in the project. These meetings shall be attended by the Contractor and their Construction Manager, the CQA Monitor, the CQA Engineer (or the Engineer's Representative), the Geosynthetics Installer, and any other representatives designated by the Employer.

The CQA Monitor shall be responsible for organising and chairing these meetings. The primary purpose of the progress meetings shall be to review construction progress and address any issues affecting the quality, schedule, or execution of the works. This shall include a review of the previous week's activities and achievements, as well as a discussion of upcoming scheduled work.

In addition, the meetings shall serve as a platform to identify and assess any defects, construction challenges, or potential risks. Appropriate solutions and corrective actions shall be discussed and agreed upon. Where necessary, any proposed amendments to the design arising from site conditions or construction challenges shall be reviewed and referred to the Design Engineer for approval.

The status and results of all CQA field and laboratory testing shall also be reviewed to ensure ongoing compliance with project specifications. All proceedings of the progress meetings shall be formally documented by the CQA Monitor, and minutes shall be distributed to all attendees for record and follow-up actions.

3.3 Resolution Meetings

Special resolution meetings shall be convened where unforeseen problems arise, or where there is a potential risk of such issues occurring during the construction period. In the event that any discrepancy, defect, or concern is identified by any of the project parties, the CQA Monitor or CQA Engineer shall be notified immediately, and a meeting shall be arranged without delay to address the matter.

These meetings shall be attended by the CQA Monitor, the CQA/Design Engineer (or the Engineer's Representative), the Project Manager, the Contractor, the Geosynthetics Installer, and any other relevant parties as deemed necessary. The purpose of the resolution meeting shall be to formally identify and discuss the issue, evaluate possible solutions and alternatives, and agree on the most appropriate course of action.

Once a solution has been agreed upon, the implementation strategy shall be defined, including responsibilities, timelines, and any required adjustments to the construction programme. The potential impact on project timelines shall also be reviewed, and an action plan shall be developed and implemented accordingly. The agreed solution shall be assessed to ensure that it remains compliant with the applicable design standards, specifications, and regulatory requirements.

All resolution meetings shall be formally documented and chaired by the CQA Monitor or CQA Engineer. Minutes of the meeting shall be recorded and distributed to all attendees for implementation and record purposes.

4. EARTHWORKS AND CONTAINMENT SYSTEM REQUIREMENTS

All construction activities associated with the NEMWA (2013) compliant containment barrier system, including specified earthworks and related infrastructure, shall be carried out strictly in accordance with the approved Project Drawings, Construction Specifications, and Contract requirements. This CQA Plan establishes the monitoring and testing framework necessary to verify that all works meet the required design standards and regulatory obligations.

The earthworks quality assurance programme shall include the testing and verification of soil and rock materials utilised during excavation, subgrade preparation, and the construction of the containment barrier system. Quality assurance inspections and testing shall be conducted throughout all critical stages of construction, including excavation of the subgrade, placement and compaction of engineered fill, and installation of liner system components.

All earthworks, concrete works, and associated civil construction shall comply with the detailed specifications provided in the Construction Drawings and contract documentation. The quality assurance programme shall assess both the suitability of materials prior to construction and their performance characteristics after placement, in accordance with the requirements outlined in this plan.

Continuous observation and testing shall be undertaken during the construction process to ensure compliance with specified standards and to identify any deviations at an early stage. In addition, appropriate concrete testing shall be conducted for structural elements such as overflow spillways and silt traps, particularly where these structures interface with the HDPE liner system, to ensure adequate strength, durability, and compatibility with the liner installation.

5. CONSTRUCTION MONITORING AND TESTING

The CQA Monitor shall be responsible for observing and testing all components of the works to verify compliance with the Construction Specifications and approved Project Drawings. The CQA Engineer shall review the activities and findings of the CQA Monitor to ensure that all quality assurance requirements are adequately implemented and maintained.

Any substandard test results, non-compliant visual inspections, or unsuitable materials identified during construction shall be recorded by the CQA Monitor and immediately brought to the attention of the Design Engineer. Appropriate corrective actions shall be implemented to prevent any potential underperformance of critical components, particularly the compacted clay liner system.

An independent surveyor shall be appointed to carry out surveys and visual verification of the works to ensure that all construction is executed in accordance with the specified levels, grades, and alignments as indicated on the Construction Drawings. Survey verification shall be undertaken at key stages during construction to confirm compliance.

All construction activities shall be subject to continuous observation and testing by the CQA Monitor to ensure adherence to Project Specifications. The Design Engineer shall review the monitoring results

and identify any deficiencies in construction methods or materials that may negatively impact the performance of the NEMWA-compliant containment barrier system. Visual inspections and survey verification shall be conducted for specific layers throughout the construction process to confirm that materials are placed in accordance with the required lines and grades.

The CQA Monitor and Design Engineer shall provide the Project Manager with adequate notice of the anticipated completion of construction components to allow for timely review and acceptance of CQA documentation, thereby avoiding delays to the Contractor.

Specific CQA monitoring and testing shall be conducted for the following components:

- Engineered fill
- Subgrade preparation, including subsurface drainage
- Compacted clay liner (CCL)
- Anchor trench backfill
- Soil protection layer
- Drainage gravel and leachate collection system (LCS) drainage layer
- Operations soil layer or initial waste layer

In addition, the CQA Monitor and CQA Engineer shall oversee the independent CQA Laboratory in carrying out detailed inspections and testing, including:

- Verification of compaction layer thickness, moisture content, and dry density (including nuclear/Troxler testing) for engineered fill and PCD embankments
- Monitoring compaction and material requirements of the clay liner constructed in two layers of 150 mm each
- Inspection of trenching and backfilling, including gradients and installation of perforated underdrainage pipes
- Concrete testing, including slump and cube strength tests for silt traps and overflow structures

-
- Verification of final surface finishes of concrete and earthworks to prevent damage to the HDPE geomembrane and ensure proper interface

Furthermore, the CQA Monitor and Design Engineer shall observe the installation of aggregate base layers (serving as geomembrane protection) and HDPE piping systems to confirm compliance with the Project Drawings and Specifications.

6. EARTHWORKS TESTING REQUIREMENTS

The CQA Monitor shall be responsible for observing and documenting all subgrade preparation activities prior to the placement of engineered fill. This shall include monitoring the stripping of vegetated soil and growth media, which shall be stockpiled in designated areas as directed by the Owner. The CQA Monitor shall also ensure that appropriate dust control measures are implemented throughout the construction process.

Visual inspections shall be conducted during excavation to identify any moisture seepages, soft or excessively wet zones, or unstable slopes that may compromise the integrity of the subgrade. The CQA Monitor shall verify that the prepared subgrade is free from soft, organic, or otherwise deleterious materials and that it is firm, stable, and compliant with the Project Specifications, including required compaction density or California Bearing Ratio (CBR) values.

In addition, the CQA Monitor shall confirm that the subgrade is suitable to support the installation of overlying geosynthetic layers in accordance with the Project Specifications. Materials used for engineered fill and anchor trench backfilling shall be sourced from approved excavation areas within the landfill cell or from designated clay stockpiles.

CQA observation and testing shall be undertaken during construction to ensure that both materials and workmanship comply with the specified requirements. The type and frequency of testing shall be as outlined in Table below, although testing frequencies may be increased where site conditions or material variability warrant additional verification. Any additional testing requirements shall be recommended by the CQA Monitor and approved by the Design Engineer.

Test Designation	ASTM	Compacted Clay	Subgrade
Visual-Method Soil Classification	D2488	Continuous	1 per Soil Type
Moisture-Density	D1557	1 Test	1 Test
Atterberg Limits	D4318	1 per Lift	NA
Nuclear Moisture Density	D6938	4 Tests Per Lift	3 Tests
Moisture Content	D2216 or D 4643	4 Tests Per Lift	1 Test
Sand Cone Test/Drive Cylinder Test	D1556 D2937	1 Per 20 Nuclear	NA
Hydraulic Conductivity	D5084 (5 psi)	2 per Lift	NA
Field-Scale Infiltration Test & Permeability or	D3385 11 (2020)	Two Stage Borehole Permeameter	NA
Alternatively Double ring Infiltrometer or	D3385	1 per lift	
Guelph Meter test	D5126 – 16e1	1 per lift	

All tests shall be conducted on a systematic grid basis to ensure adequate spatial coverage across the works. In areas where large volumes of fill are placed within confined spaces, the testing frequency shall be increased as necessary to ensure that each layer of placed material is adequately assessed for compliance.

7. CONCLUSION

This CQA Plan provides a structured framework to ensure that the Maletswai Landfill barrier systems are constructed in accordance with design specifications, regulatory requirements, and best engineering practice, thereby ensuring long-term environmental protection and facility performance

ANNEXURE B GEOTECHNICAL REPORT

ANNEXURE C LANDFILL INTERNAL AUDIT REPORT TAPLATE

ANNEXURE D COMPLIANCE MONITORING TAMPLATE

ANNEXURE D RISK MANAGEMENT PLAN

ANNEXURE E Sensitivity Analysis of Waste Growth Rate and Airspace Requirements

1 Introduction

A sensitivity analysis was undertaken to evaluate the impact of different waste growth rates on the long-term airspace requirements of the proposed Maletswai Landfill. The purpose of the analysis is to assess the robustness of the landfill design under varying waste generation scenarios and to determine whether the proposed airspace allocation remains adequate should future waste volumes deviate from the projected baseline growth rate.

The base-case design assumes an annual waste growth rate of **1.5%**, which was applied to the projected waste volume for the first year of operation (2027). Additional scenarios of **1.0%, 2.0%, and 3.0%** annual growth were evaluated.

2 Design Assumptions

Parameter	Value
Base year waste volume (2027)	15,960 m ³
Design life	20 years
Cover-to-waste ratio	1:4
Airspace factor	1.25
Base growth rate	1.5%
Sensitivity growth rates	1.0%, 2.0%, 3.0%

3 Methodology

Annual waste volumes were projected using:

$$V_n = V_0(1 + r)^n$$

Where:

V_n = Waste volume in year n (m^3)

V_0 = Initial waste volume ($15,960 m^3$)

r = Annual growth rate

n = Number of years

The cumulative waste volume over the 20-year design period was calculated using the geometric series equation:

$$V_T = V_0 \left(\frac{(1 + r)^{20} - 1}{r} \right)$$

Total airspace requirements were then determined by applying the cover material allowance:

$$A_T = 1.25 \times V_T$$

Where:

A_T = Total airspace required (m^3)

4 Base Case Calculation (1.5%)

Step 1: Calculate Growth Factor

$$(1 + 0.015)^{20} = 1.346855$$

Step 2: Calculate Total Waste Volume

$$\begin{aligned} V_T &= 15\,960 \left(\frac{1.346855 - 1}{0.015} \right) \\ &= 15\,960 \times 23.1237 \\ &= 368\,705 m^3 \end{aligned}$$

Step 3: Calculate Total Airspace

$$A_T = 368\,705 \times 1.25$$
$$= 460\,881m^3$$

5 Low Growth Scenario (1.0%)

Step 1: Growth Factor

$$(1.01)^{20} = 1.22019$$

Step 2: Total Waste Volume

$$V_T = 15\,960 \left(\frac{1.22019 - 1}{0.01} \right)$$
$$= 15\,960 \times 22.019$$
$$= 351\,423m^3$$

Step 3: Total Airspace

$$A_T = 351\,423 \times 1.25$$
$$= 439\,279m^3$$

6 High Growth Scenario (2.0%)

Step 1: Growth Factor

$$(1.02)^{20} = 1.48595$$

Step 2: Total Waste Volume

$$V_T = 15\,960 \left(\frac{1.48595 - 1}{0.02} \right)$$
$$= 15\,960 \times 24.2975$$
$$= 387\,788m^3$$

Step 3: Total Airspace

$$A_T = 387\,788 \times 1.25$$

$$= 484\,735\text{m}^3$$

7 Extreme Growth Scenario (3.0%)

Step 1: Growth Factor

$$(1.03)^{20} = 1.80611$$

Step 2: Total Waste Volume

$$V_T = 15\,960 \left(\frac{1.80611 - 1}{0.03} \right)$$

$$= 15\,960 \times 26.8703$$

$$= 428\,850\text{m}^3$$

Step 3: Total Airspace

$$A_T = 428\,850 \times 1.25$$

$$= 536\,063\text{m}^3$$

8 Summary of Results

Growth Rate	Total Waste Volume (m ³)	Total Airspace Required (m ³)	Difference from Base Case
1.0%	351,423	439,279	-4.7%
1.5% (Base Case)	368,705	460,881	0%
2.0%	387,788	484,735	+5.2%
3.0%	428,850	536,063	+16.3%

9 Conclusion

The sensitivity analysis demonstrates that the total airspace requirement is moderately sensitive to changes in the assumed waste growth rate. Under the adopted base-case growth rate of **1.5%**, the required landfill airspace is **460,881 m³** over the 20-year design life. A lower growth rate of **1.0%**

reduces the required airspace to approximately **439,279 m³**, while a higher growth rate of **3.0%** increases the requirement to approximately **536,063 m³**.

The results indicate that the adopted design airspace provides a reasonable planning basis and that the proposed landfill layout should remain adequate under a range of foreseeable waste growth scenarios. The analysis further highlights the importance of periodic waste quantity reviews during landfill operation to confirm that actual waste generation trends remain consistent with the design assumptions.



MALETSWAI LOCAL MUNICIPALITY

GEOTECHNICAL INVESTIGATION FINAL REPORT

**GEOTECHNICAL INVESTIGATIONS REPORT FOR: ALIWAL NORTH LANDFILL
SITE**



EARTHINVLAB

OFFICE
Plot 209 Tienie Street Andeon Zandfontein 0183 Tell: 012 372 3023 Cell: 061 473 1190 Fax: (086 241 3304) e-mail: admin@earthinv.co.za

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Name & Title	Purpose
Imbawula Civil Projects	Information and Retention
Maletswai Local Municipality	Information and Retention

Approval

The signatories hereof, being duly authorized thereto, by their signatures hereto confirm their acceptance of the contents hereof.

Name	Designation	Signature	Date
Lukhanyo Gqobo	Engineering Geologist (Pr.Sci.Nat)		03/06/2026

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1. INTRODUCTION AND TERMS OF REFERENCE

Earthinv Lab was appointed by Imbawula Civil Projects to carry out a geotechnical investigation to identify subsurface conditions that will affect the implementation of the construction of Aliwal North Landfill Site. The geotechnical data, discussions and recommendations of this report includes a field reconnaissance, data review and field explorations. The investigation comprised a desktop study, site walkover and fieldwork. The information in this report will inform the design and construction precautions to be considered in the implementation of the project, thus reducing the risk of structural failure and construction damage where such adverse conditions may occur.

1.1. Purpose and scope of work

This ground investigation was carried out in order to establish the site-specific geotechnical properties of the subsurface material below the proposed development and to provide geotechnical evaluation and recommendations needed for the detailed design of the construction of Aliwal North Landfill Site as stated below:

- Review existing geologic maps and regional seismic and geological data.
- Assess the groundwater level underneath the site at the time of investigation, if encountered.
- Review available subsurface information in the project vicinity
- To provide an appropriate site classification according to the TRH14 Classification System
- Evaluation of potential construction constraints and development of possible mitigation.
- Overview of existing structures within the area for any relevant information that could assist in the investigation.
- Advise on appropriate, practicable and cost-effective conceptual planning and design options for the development.
- Excavation conditions.

Imbawula Civil Projects responsible for the design of the construction of Aliwal North Landfill Site. This report addresses the required field works needed and provides the parameters to facilitate the design, as well as all applicable South African regulatory requirements, which includes:

- SAICE (Site Investigation Code of Practice)
- SAIEG (Guidelines for Soils and Rock Logging in South Africa)
- Relevant SANS Codes of Practice

1.2. Available Information

- ❖ Geological Map 3026 Aliwal North 1:250 000
- ❖ Locality Plan

2. SITE INFORMATION

2.1. Site Description

The site for the proposed development is located in the western region of Aliwal North CBD within the Maletswai Local Municipality in Eastern Cape Province. The study area which is approximately 6.5 hectares and located approximately 6km north west of Aliwal North CBD. The coordinates from the center of study area in Longitude and Latitude, Datum MAP 64 are 30°41'28.25"S, 26°39'48.11"E with an elevation of 1325m above the sea level.

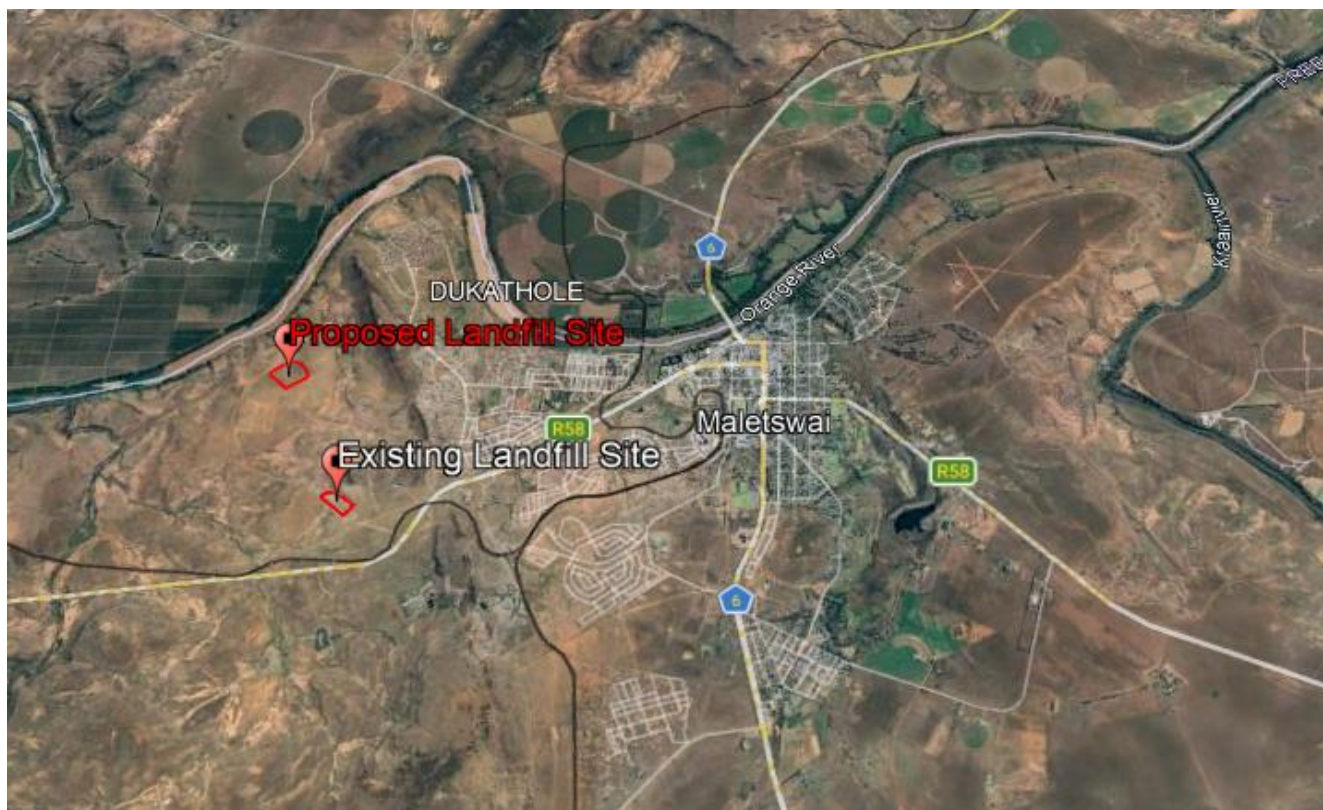


Figure 1 – Site Locality Map

2.2. Topography

The topography is gently sloping towards the northerly direction where there is an existence of the perennial Orange River. The study area is occupied by short grass and short sparsely arranged trees. The lowest altitude on site is on the northern region with 1325m above the sea level and the highest altitude is at the southern region with 1327m above the sea level.

2.3. Existing Infrastructure

During our investigations there were no underground services observed within the proposed area that could be of hindrance during construction as it was a greenfield. It is recommended that all way-leaves or service detection be performed prior to any commencement of works/excavation, to avoid services interruption. Way-leaves may be attained as shown on below table 1, to ascertain that there are no existing services:

Photos 1 to 4 show the existing services and surrounding infrastructure.



Photo 1 – Electric Power line on the study area.



Photo 2 – Existing vegetation on site.



Photo 3- Garbage on site.



Photo 4- Test pit on the site area.

When planning of construction approaches, we propose wayleaves be attained as shown on below **Table 1**, in order to ascertain there are no existing services:

Table 1: Services Information

Name of Institution	Type of Service
Eskom	Electricity
Telkom	Telecommunication
Local Municipality	Electricity
Local Municipality	Water & Sanitation
Local Municipality	Roads, Transport & Civil Works

Attaining the above information will also assist in knowledge of future developments planned (if any), to avoid clashing of development or infrastructure related to human settlement.

2.4. Climate

According to meteoblue the "mean daily maximum" (solid red line) shows the maximum temperature of an average day for every month for Aliwal North. Likewise, "mean daily minimum" (solid blue line) shows the average minimum temperature. Hot days and cold nights (dashed red and blue lines) show the average of the hottest day and coldest night of each month of the last 30 years. Aliwal North receives about 702mm of rain per year, with most rainfall occurring during summer. It receives the lowest rainfall (15mm) in July and the highest (109mm) in January. The mean daily temperatures for Aliwal North ranges from 1°C in June and 0°C July in January 30°C and December 29°C.

Aliwal North

30.69°S, 26.71°E (1325 m asl).
Model: ERA5T.

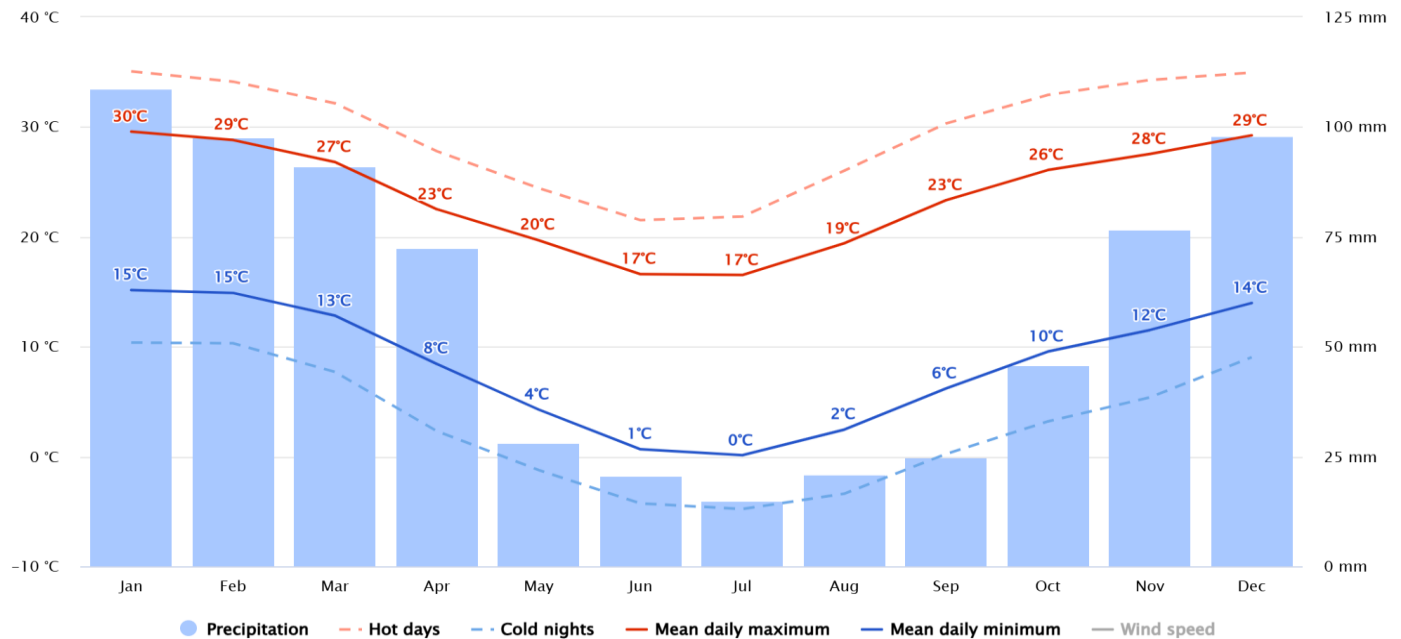


Figure 2 – Aliwal North Climate

Climate determines the mode of weathering and rate of weathering. The effect of climate on the weathering process (i.e., soil formation) is determined by the climatic N-value defined by Weinert.

The N-value for Aliwal North is 2 and 5, which implies a sub humid climate. Chemical decomposition and the chemical alteration of a rock can therefore be expected. This can lead to the transformation of certain minerals into some type of clay.

“These materials tend to have relatively high plasticity and are moisture sensitive. Basic igneous rocks are often not durable and prone to degradation in service. Careful attention should be paid to the internal and external drainage of pavement”. LVSR Guidelines

Climate Data indicates that construction would be better suited between the months of August and September, as there is less rain which can obstruct a construction program. The months of June and July are in particular most favorable as there is very little or no rain.

Table 2: Weinert N Value Versus Climate

Climate Zone	Arid	Semi-Arid	Sub Humid	Humid
Weinert N-Value	>10	5-10	2-5	<2

Mean Annual Rainfall (mm)	< 250	250-500	500-800	800
---------------------------	-------	---------	---------	-----

3. METHOD OF INVESTIGATION AND OBSERVATIONS

3.1. Field Investigation

The site investigation was conducted on the 08th May 2026, which included total of seven (7) test pits and seven (7) DCP tests. These test pits were excavated within the area of proposed developments. Coordinates of the test pits were determined using a hand-held Garmin MAP 64 GPS instrument on the South African grid, while excavations were done using an excavator, following the safety procedures as set out in the SAICE Code of Practice (2003, updated 2007), as well as the Occupational Health and Safety Act No. 85 of 1993. Soil profiles were described by an engineering geologist based on the standard method proposed by Jennings et al. (1973). Table 3 gives a summary of test pit profiles, while profile logs are attached as Annexure E. The map showing test pits positions are enclosed as Annexure C while Annexure D contains the Site Photos.

Table 3: Test pits summary information

Sample No	Sample Depth(m)	Total Depth (m)	GPS Coordinates		Remarks
			X	Y	
TP1	1.5-2.7	2.7	30°41'25.45"S	26°39'48.46"E	No groundwater seepage, no refusal
TP2	0.0-0.7	2.9	30°41'30.07"S	26°39'44.54"E	No groundwater seepage, no refusal
TP3	1.0-2.85	2.85	30°41'32.13"S	26°39'49.80"E	No groundwater seepage, no refusal
TP4	-	2.9	30°41'28.98"S	26°39'53.40"E	No groundwater seepage, no refusal

Center	0.8-2.8	2.8	30°41'28.96"S	26°39'49.10"E	No groundwater seepage, no refusal
PCD	0.7-3.0	3.0	30°41'25.49"S	26°39'52.43"E	No groundwater seepage, no refusal
Landfill	-	1.9	30°42'12.40"S	26°40'12.37"E	No groundwater seepage, no refusal

3.2. Laboratory Testing

Soil samples of the in-situ soils were retrieved and delivered to our SANAS accredited soil testing laboratory to determine the material classification. The following tests were undertaken:

- Foundation indicators
- Maximum Dry Density (MDD)
- California Bearing Ratio (CBR)
- PH & Conductivity
- Bulk density and soil activity (Shrink / Swell Potential)
- Permeability

The tests were conducted to determine materials suitability for foundation Indicators for the proposed development. Detailed analysis and interpretation of the laboratory results will be included in this final report.

Table 4: Laboratory Test Results for Foundation Indicators

Test Position	Depth (m)	Material description	Atterberg limit			PERCENTAGE FINER THAN (mm)			GM	CLASSIFICATION		
			LL	PI	LS	0.075	0.42	2.00		AASHTO	UNIFIED	TRH14
1	1.5-2.7	Light brown silty sand	25	3	1.4	38	96	97	0.68	A-4	ML	G7
2	0.0-0.7	Dark brown silty sand	-	NP	0.0	69	100	100	0.31	A-4	ML	-
3	1.0-2.85	Orange brown blotched white	34	6	3.4	61	91	94	0.54	A-1-b	CL	G9

		silty sand										
Center	0.8-2.8	Orange brown silty sand	33	3	1.3	70	98	99	0.33	A-4	ML	G9
PCD	0.7-3.0	Orange brown silty sand	33	6	3.3	52	99	99	0.50	A-4	CL	G10

GM-Grading Modulus

LL-Liquid Limit

PI-Plasticity Index

LS-Linear Shrinkage

The maximum dry density that can be obtained by compaction is dependent upon the soil type and arrangement of various particles making up the soil structure with relative movement of soil particles being affected by the water content. Each soil type has its own optimum moisture content for certain compacted effort and the effect of increasing the compactive effort is to increase the maximum dry density and decrease the optimum moisture content. Table 5 shows the maximum and minimum values obtained for maximum dry density and OMC for samples obtained from excavated test pits. The variation in maximum dry density and OMC shown on the table are due to the variation in particle size and number of fines in the soil mass.

CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The variation in CBR values are the results of differences in size and shape of the particles, the prevailing moisture within the soil, grading of the particles, how loose the grains are, presence of hard rock fragments and fines. CBR test was conducted on recovered samples from profiled test pits to characterize the strength and bearing capacity of the obtained samples and these were used as a guide complementing CBR obtained from DCP testing.

Table 5: Compaction Properties

Tests	Maximum	Minimum
MDD (kg/m³)	2023	1743
OMC %	18.1	11.9
CBR @93%	18	2
CBR @95%	22	3
CBR @98%	27	4

Laboratory results show low values for obtained CBRs @ 93%, 95% and 98%. According to TRH14, material sampled from excavated test pits on site have been classified as G7 to less than a G10 class. Parameters obtained indicate that material have fair to poor subgrade properties in accordance with the AASHTO soil classification. See Annexure G: Laboratory Test Results for ease of reference.

3.3. Hydrometer Analysis

Hydrometer analysis is the process by which fine graded soil, silts, and clays are graded. It also determines the specific gravity of the suspension, and the specific gravity depends upon the mass of the solids present which in turn depends on the particle size. This test is carried out to quantitatively determine the particle size distribution for soil particles of size smaller than 0.075 mm.

Table 6: Hydrometer Test Results

Test Pit	Depth of Sample (m)	Overall, PI	Activity
1	1.5-2.7	2.58	Low
2	0.0-0.7	NP	Low
3	1.0-2.85	5.53	Low
Center	0.8-2.8	2.80	Low
PCD	0.7-3.0	5.99	Low

The samples activity factor is low which means low soil ground movements with changing moisture content and applied weight. Refer to Annexure G for the Laboratory Test Results”.

3.4. pH & Conductivity

From the journal of Applied Sciences Research, vol. 8, no. 3, pp. 1739-1747, 2012 ("Relationship between Soil Properties and Corrosion of Carbon Steel") by M. N. Norhazilan and Y. Nordin, resistance of an electrolytic solution, as represented by the saturated soil paste or water, is that property of the solution which opposes the flow of an electric current. The resistance is dependent on the amount and nature of dissolved ion content (TMH 1 second addition). The potential of hydrogen (pH) 8.2-8.3 was measured, which is neutral to alkaline. The tested values conductivity is 0.004 to 0.009 S/m indicating that the materials are generally not corrosive to very corrosive. The tabulated results in **Table 7**, conforms to the limits as specified in **Table 8**

Table 7: Measured pH & Conductivity

TP	Depth (m)	pH Value	Conductivity s/m
1	1.5-2.7	8.3	0.004
2	0.0-0.7	8.7	0.005
PCD	0.7-3.0	8.2	0.009

Table 8: Conductivity & Corrosiveness of the Soil

Conductivity(mS/cm)	Corrosiveness
Greater than 0.5	Very corrosive
0.5-0.2	Corrosive
0.2-0.1	Mildly corrosive
Less than 0.1	Generally, not corrosive

4. GEOLOGY

The Geological Map Series, sheet number 3026 Aliwal North, published at a scale of 1:250 000 by Department of Mineral and Energy Affairs indicate that the area is underlain by alluvium deposits of Quaternary age, See Annexure B – Geological Map Extract.

5. HYDROGEOLOGY

There was no groundwater encountered during excavation work. It is important that the design of the storm water management system allow for the drainage of accumulated surface water from the platforms and into the storm water system or natural drainage lines.

6. GEOTECHNICAL EVALUATION

6.1. Test Pits

Total of seven (7) test pits were excavated and profiled by an engineering geologist, with detailed analysis carried out based on the field works and interpretation of laboratory results obtained from the representative soil sample collected during the ground investigation around the study area. The following soil horizons were encountered on site for the seven (7) excavated test pits.

6.1.1. Colluvium Material/Top Soil Material

These are naturally transported materials from the hill wash or gravity. This material was encountered at all the test pits a maximum depth of 0.8m. This material was described as silty sand and clayey sand. The consistency of this material is medium dense. The structure of the observed colluvium material is intact.

6.1.2. Pedogenic Material

The pedogenic material is a hardened soil or sedimentary material that forms due to the cementation of soils particles by minerals like iron oxides, calcium carbonate, or silica. This material consists of calcrete. This type of material is normally encountered between the transported soils and underlying residual material or bedrock. This material was only encountered at all the test pits except at PCD and Dump wast at a maximum depth of 30m. The encountered material was described as silty sand. The consistency of this material was medium dense to dense, and the observed structure was intact.

6.2. Dynamic Cone Penetrometer (DCP) Tests and Analysis

The dynamic cone penetrometer (DCP) tests were carried out adjacent to the trial pits during the field investigations. This method of testing was used to estimate the in-situ CBR for the subgrade layers.

The dynamic cone penetration (DCP) test is conducted by driving a 20 mm diameter, 60° cone into the ground by an 8 kg hammer. The hammer is lifted by hand and dropped a distance of 575 mm and the results are expressed as the penetration rate (PR) in mm per blow.

The following model that has been adapted from “The use and interpretation of the dynamic cone penetrometer (DCP) test” by P Paige-Green and L Du Plessis refers:

If $DN > 2$ mm/blow $CBR = 410 \times DN^{-1.27}$

If $DN < 2$ mm/blow $CBR = (66.66 \times DN^2) - (330 \times DN) + 563.33$

DN (the rate of cone penetration)

The results are tabulated on the graph indicating a stable founding material with an average CBR of 12.0% and a UCS of 162 % at all test pits.

The following model that has been adapted from “The use and interpretation of the dynamic cone penetrometer (DCP) test” by P Paige-Green and L Du Plessis refers:

Bearing capacity (kPa) = $3426.8 \times DN^{-1.0101}$

The average bearing capacity has been calculated at 234 KPa.

Table 9: DCP Ave CBR, UCS & Bearing Capacity

TP	Ave CBR	Ave UCS	Ave Bearing capacity
1	8,6	102	153
2	13,4	153	224
3	15,6	175	254
4	11.1	131	193
Center	8.0	98	149
PCD	14.0	152	218
Dump Waste	31.4	324	449

6.3. Excavatability

The excavation characteristics of the different soil horizons encountered have been evaluated according to the South African Bureau of Standards standardized excavation classification for earthworks (SABS-1200D) and earthworks (small works-SABS-1200DA). The soil on site can be classified as soft to intermediate excavation.

6.4. Stability of trenches

The test pits excavations were all vertical and there was no evidence of side wall collapse during excavation of the test pits, therefore any trenches excavated to within the limits of the tests pit depths are expected to be stable. However, in cases where water ingress is encountered, or the slopes are left open for an extended period, there could be instability problems. In such case(s), the excavated trenches would have to be battered to stable angles or shored to avoid sidewall collapse.

6.5. Earthworks

It is recommended that all earthworks be carried out in accordance with SABS1200DM (latest version). All unsuitable fill, i.e., garden refuse and general waste, should be removed from the site. All vegetation should be cleared from the areas over which fills are to be built. In addition, the upper topsoil should be removed and stockpiled for later use as topsoil.

6.6. Drainage

The most important factor in the promotion of a stable site is the control and removal of both surface and groundwater from the site. It is important that the design of the stormwater management system allow for the drainage of accumulated surface water. Such water should be directed towards the natural drainage lines. Disposal of stormwater should in any case conform to the Local Authority's requirements. Points of discharge of piped stormwater should be carefully designed to limit erosion.

6.7. RECOMMENDATIONS

Under no circumstances should foundations be placed in/on untreated natural subsoils unless it has been specifically engineered to support structural foundations. In general, good practice requires that the in-situ moisture content of the founding horizons below the structure be maintained. The document, SANS 10400-H has reference for the design of the foundations.

The findings of these investigations are constant with the notion that the site is underlined by alluvium deposits.

Due to the collapsable grain structure associated with alluvium sands, it is recommended that certain precautions should be taken to prevent structural damage. According to NHBRC, this site will classify as a C1. No perched water levels were present during the investigation and no clay material suitable for the use as liner material was encountered on site.

Liner material for the construction of the landfill liners must be imported or a GCL should be used.

Proper surface drainage needs to be designed and constructed to prevent excessive erosion.

7. General Precaution Measures

The following Safety Precaution Measures should be adhered to during the construction:

- Excavations guarded/barricaded/lighted after dark in public areas
- Soil dumped at least 1m away from edge of excavation
- On sloping ground soil dumped on lower side of excavation
- Keep heavy equipment at least 1 meter from trench edges to avoid surcharge loads.
- Know where underground utilities are located.
- Inspect trenches at the start of each shift.

- Inspect trenches following a rainstorm.

8. REFERENCES

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9. Brandt, M.B.C., Bejaichund, M., Kgaswane, E.M. Hattingh, E. and Roblin, D.L. 2005. Seismic history of Southern Africa. Seismologic series 37. Council for Geoscience of South Africa, Pretoria, 32 pp.

Annexure A Locality Map

Annexure B

Geological Map

Annexure C

Test Pit Positions

Annexure D

Site Photos

Annexure E

Test Pit/Soil Profiles

Annexure F

DCP Probes and Analysis

Annexure G

Laboratory Test Results

Annexure H Topographical Map



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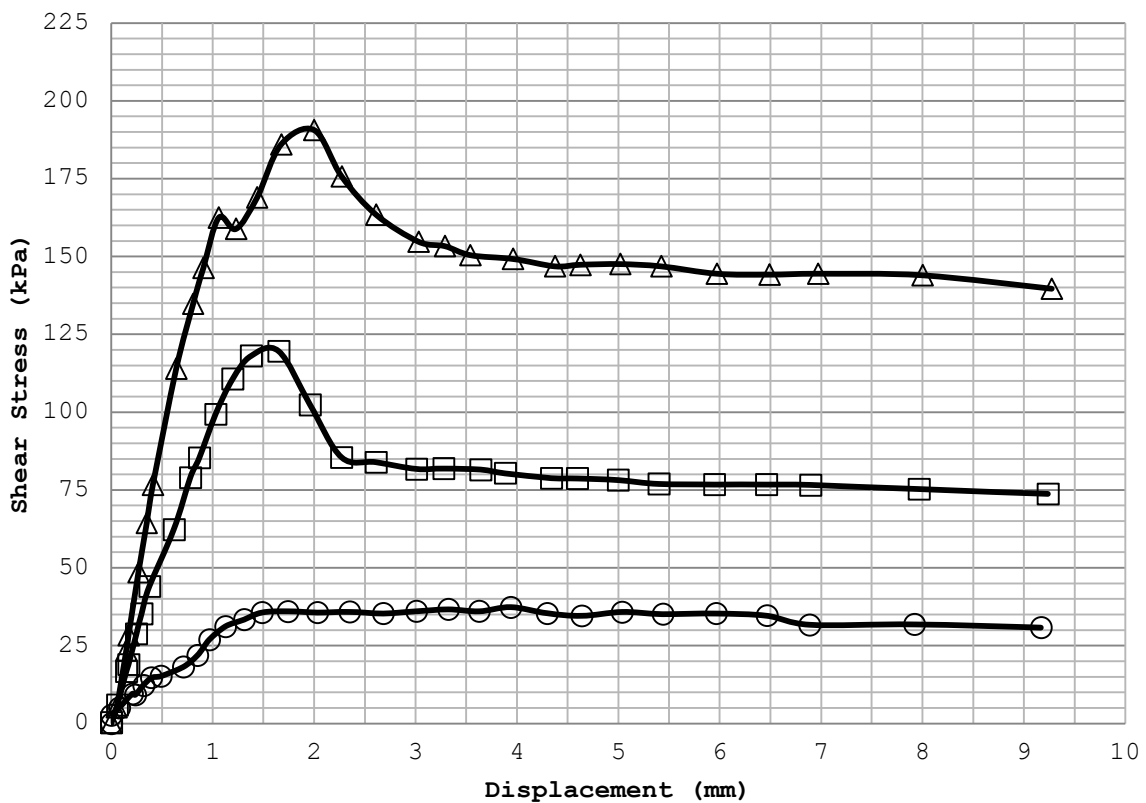
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Monument Park
0181

PO Box 26272
Monument Park
0105

Tel/Fax 012 346 7586
Cell: 082 375 3003
bennie@geotesting.co.za

P1/2

Project:	Imbawuh
Sample Number:	Centre
Sample Position:	0.8-2.8m
Test:	DIRECT SHEAR TEST
Preparation:	Remoulded
Lab Number:	26/119
Test Date:	29 May 26



Normal Stress (kPa)	50.0	100.0	200.0
Shear Stress (kPa)	37.4	119.6	190.6
Displacement (mm)	3.9	1.7	2.0



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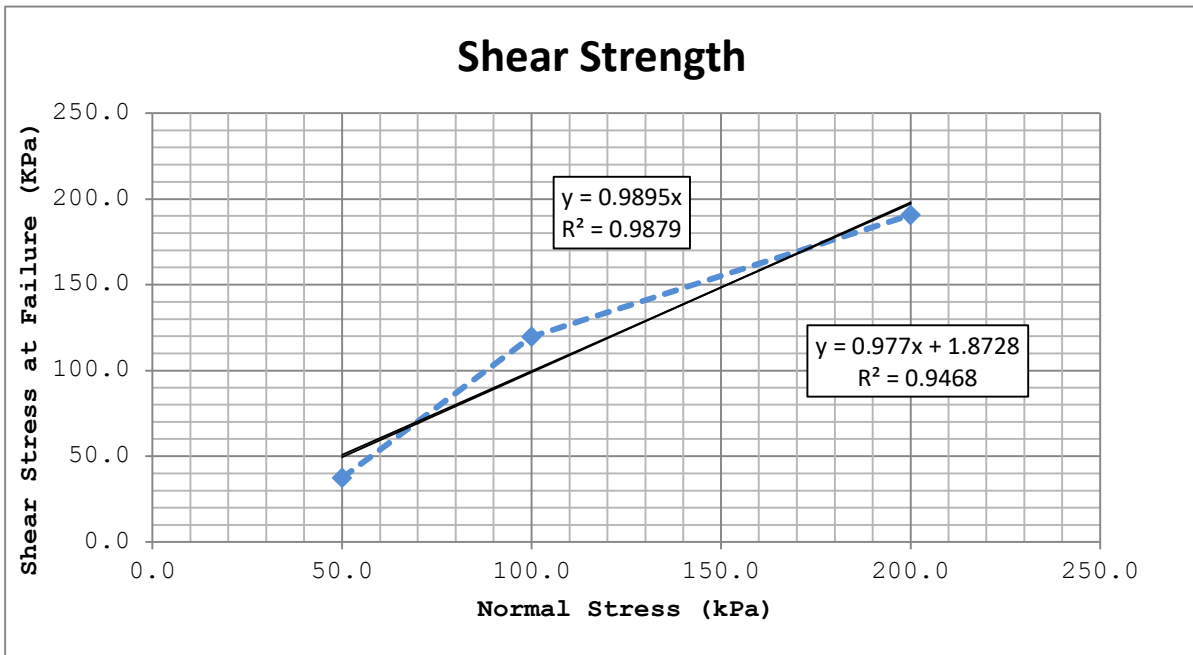
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Monument Park
0105

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Cell: 082 375 3003
bennie@geotesting.co.za

P2/2

Project:	Imbawuh
Sample Number:	Centre
Sample Position:	0.8-2.8m
Test:	DIRECT SHEAR TEST
Preparation:	Remoulded
Lab Number:	26/119
Test Date:	29 May 26



Linear trendline fit intersecting zero:

Apparent Cohesion Intersect : 0.0
Angle of Shear Resistance ϕ : 44.7 °

Linear trendline fit intersecting 3 data points:

Apparent Cohesion Intersect : 1.9
Angle of Shear Resistance ϕ : 44.3 °

Sample:	1	2	3
Initial Moisture:	13.8%	13.4%	13.8%
Initial Bulk Density (kg/m ³):	2152.4	2164.7	2153.1
Initial Dry Density(kg/m ³):	1890.8	1908.7	1891.6



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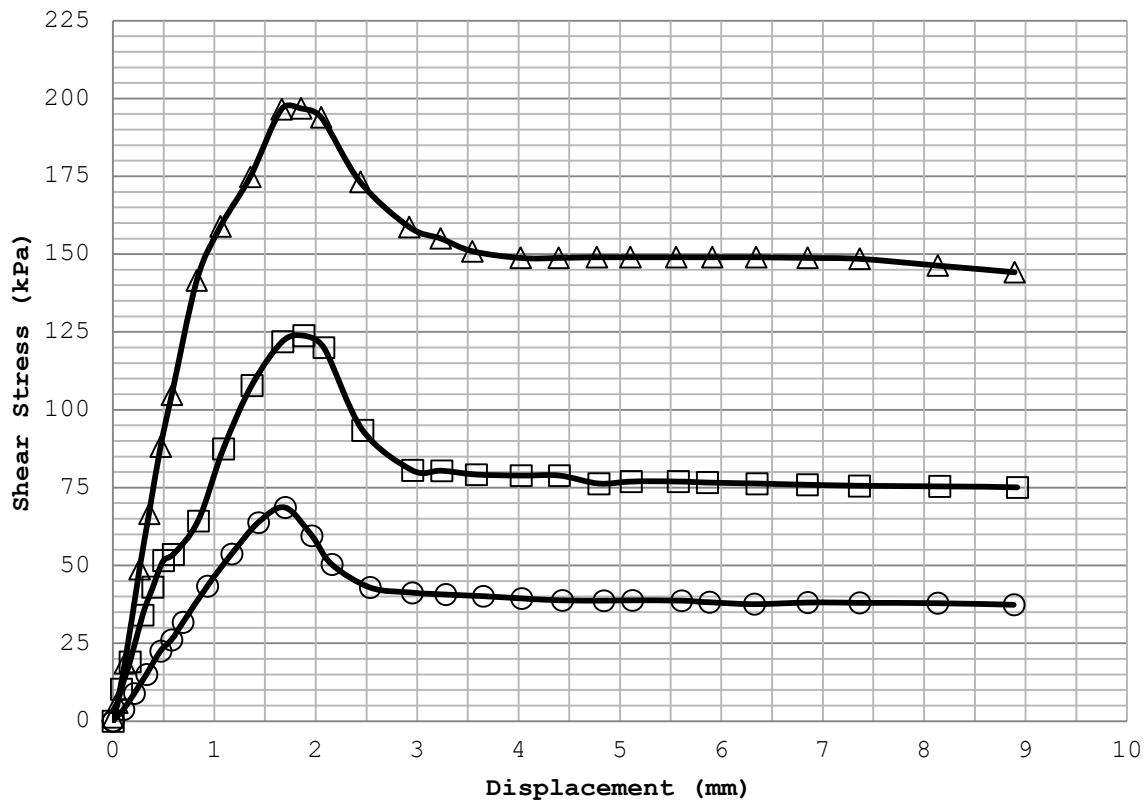
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Monument Park
0105

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Cell: 082 375 3003
bennie@geotesting.co.za

P1/2

Project:	Imbawuh
Sample Number:	PCD
Sample Position:	0.7-3.0m
Test:	DIRECT SHEAR TEST
Preparation:	Remoulded
Lab Number:	26/120
Test Date:	03 Jun 26



Normal Stress (kPa)	50.0	100.0	200.0
Shear Stress (kPa)	68.6	123.8	196.8
Displacement (mm)	1.7	1.9	1.9



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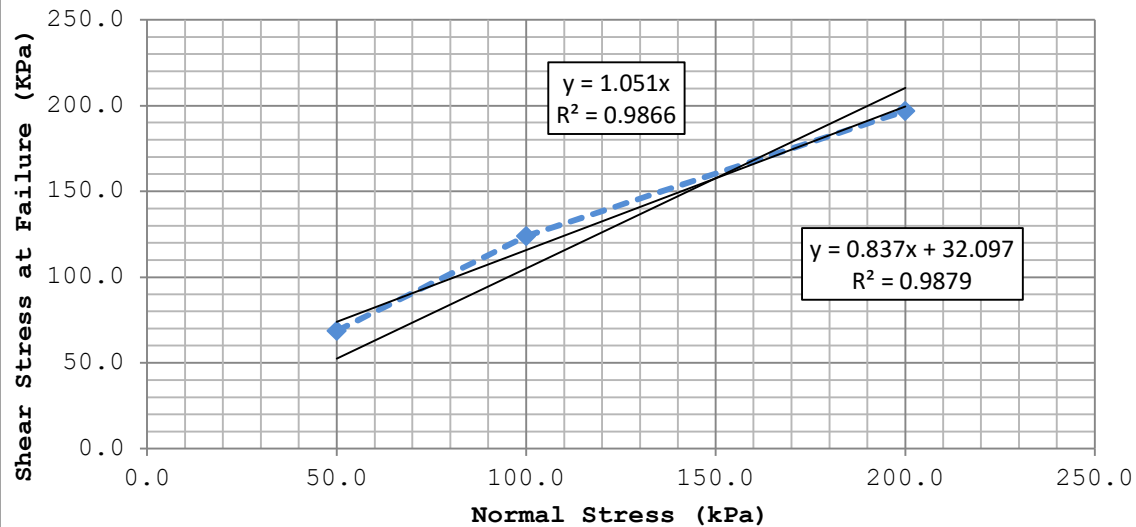
PO Box 26272
Monument Park
0105

Tel/Fax 012 346 7586
Cell: 082 375 3003
bennie@geotesting.co.za

P2/2

Project:	Imbawuh
Sample Number:	PCD
Sample Position:	0.7-3.0m
Test:	DIRECT SHEAR TEST
Preparation:	Remoulded
Lab Number:	26/120
Test Date:	03 Jun 26

Shear Strength



Linear trendline fit intersecting zero:

Apparent Cohesion Intersect : 0.0
Angle of Shear Resistance ϕ : 46.4 °

Linear trendline fit intersecting 3 data points:

Apparent Cohesion Intersect : 32.1
Angle of Shear Resistance ϕ : 39.9 °

Sample:	1	2	3
Initial Moisture:	12.9%	12.7%	12.8%
Initial Bulk Density (kg/m ³):	2413.8	2403.4	2399.1
Initial Dry Density (kg/m ³):	2138.2	2132.9	2127.2



du Plessis
Civil Engineering

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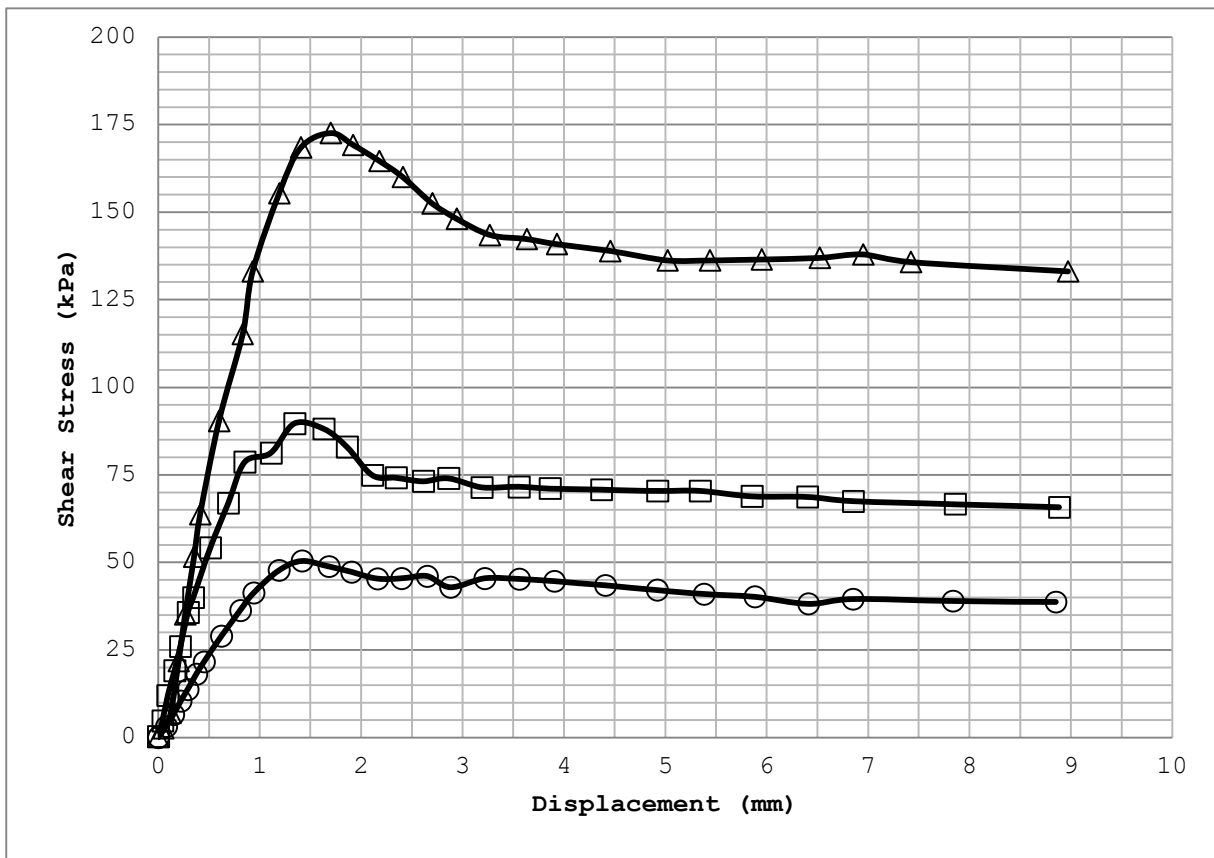
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0181

PO Box 26272
Monument Park
0105

Tel/Fax 012 346 7586
Cell: 082 375 3003
bennie@geotesting.co.za

P1/2

Project:	Imbawuh
Sample Number:	TP03
Sample Position:	1.0-2.85m
Test:	DIRECT SHEAR TEST
Preparation:	Remoulded
Lab Number:	26/118
Test Date:	22 May 26



Normal Stress (kPa)	50.0	100.0	200.0
Shear Stress (kPa)	50.4	89.7	172.6
Displacement (mm)	1.4	1.3	1.7



du Plessis
Civil Engineering

Reg. No: cc 200004833323

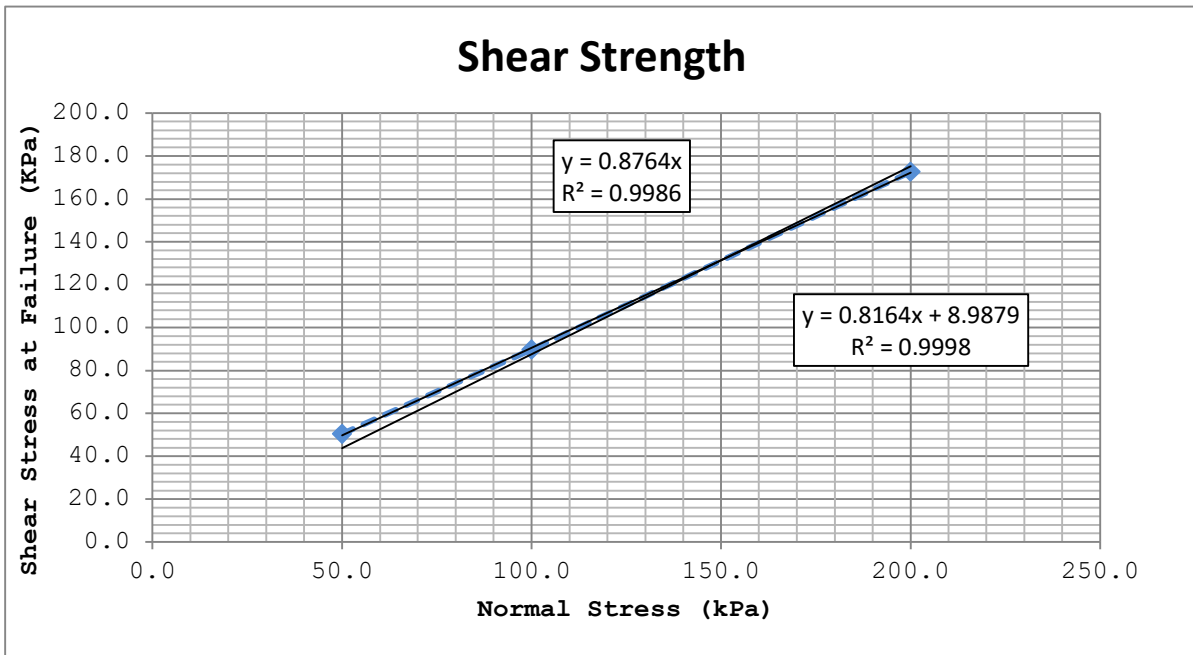
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P2/2

Project:	Imbawuh
Sample Number:	TP03
Sample Position:	1.0-2.85m
Test:	DIRECT SHEAR TEST
Preparation:	Remoulded
Lab Number:	26/118
Test Date:	22 May 26



Linear trendline fit intersecting zero:

Apparent Cohesion Intersect : 0.0
Angle of Shear Resistance ϕ : 41.2 °

Linear trendline fit intersecting 3 data points:

Apparent Cohesion Intersect : 9.0
Angle of Shear Resistance ϕ : 39.2 °

Sample:	1	2	3
Initial Moisture:	13.2%	12.7%	12.6%
Initial Bulk Density (kg/m ³):	2142.3	2140.3	2142.7
Initial Dry Density(kg/m ³):	1891.7	1899.2	1903.3