TONKMETER RESOURCE FACILITY

TONKMETER RESOURCE FACILITY
LANDFILL AREA E
DETAIL DESIGN REPORT

Report No :P077d-05

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Figure 1: Site locality plan of the Tonkmeter Resource Facility showing Area E on the eastern side

1 INTRODUCTION

1.1 Background and Terms of Reference

Vereeniging Refractories (Pty) Ltd (Verref) obtained an Integrated Environmental Authorisation (IEA) No Gaut: 002/11-12/W0016 & Gaut: 002/11-12/E0228 dated 2 October 2017 (1) from the Gauteng Department of Agriculture, and Rural Development (GDARD) for the development of the New Tonkmeter Road Waste Disposal Facility (WDF) as a Class B landfill. On 30 March 2021, GDARD approved the detail design of Cell 1 and Cell 2 of Area E of the Tonkmeter Road Landfill and the associated drainage infrastructure. In 2024, Cell 1 was constructed up to the mid-slope bench together with the leachate tank and contaminated water pond. Construction of the upper slopes of Cell 1 was completed in 2026.

On 18 June 2024, GDARD, now the Gauteng Department of Environment (GDEnv), approved a change of Waste Management License (WML) holder, from Vereeniging Refractories (Pty) Ltd to Tonkmeter (Pty) Ltd, on the basis that both companies share the same ownership and that nothing about the facility itself would change.

The IEA included various conditions under clause 4 related to the “Scope of the Integrated Environmental Authorisation (Clause 4.1). Two of these conditions are considered unreasonably restrictive and Tonkmeter (Pty) Ltd intends applying for an amendment to the IEA. These are as follows:

- The final height of the landfill cells must not exceed 1 m above ground level;
- The Class B landfill may only receive non-recyclable general municipal waste.

In addition, Tonkmeter would like to amend certain conditions of the IEA that are either not applicable or should be amended to clarify the intention of conditions. Tonkmeter would also like to amend the Environmental Management Programme (EMPr) to remove / amend commitments made that are not applicable and to be in line with the above proposed amendments.

To support the IEA Amendment application, a detail report is required that addresses the various engineering issues associated the proposed amendment. In particular, the following amendments are being requested:

- The maximum final height of the landfill cells be increased to 25 m above ground level;
- The Class B landfill should be able to receive all Type 2 wastes, provided that they are compatible with the disposal of general municipal waste.

Tonkmeter (Pty) Ltd appointed Peter Legg Consulting (PLC) to update the detail design of the Area E development of the landfill, to address the increased height and Type 2 waste disposal.

This report covers the updated detailed engineering design of Area E of the Class B Tonkmeter Landfill.

1.2 Objectives

The overall project objective is to develop an environmentally sound and legally compliant landfill covering Area E of the Tonkmeter WDF.

More specifically, the objectives of this detail design are as follows:

- To develop detailed engineering designs (drawings) and documentation for the Area E landfill including the barrier system with associated leachate, contaminated water and storm water management systems to enable regulatory authority approval for the increased height and broader Type 2 waste stream; and
- To demonstrate that the environmental impact of Type 2 waste disposal within a quarry landfill has been adequately mitigated through the Class B barrier system and drainage management design.

The purpose of this Detail Design Report is to document the design criteria, assumptions and details of the proposed landfill development with increased height, for submission to the regulatory authority for approval.

1.3 Scope

The scope of the Detail Design relates to the development of Area E of the Tonkmeter Landfill for the disposal of Type 2 waste. The intention is to utilise the area previously mined for clay for manufacture of building and refractory bricks. The design needs to mitigate the potential impacts the facility could have on the environment, including the socio-economic and biophysical environments.

The design is based on current "best practice" as well as the regulatory requirements, specifically the DWA's "Minimum Requirements for Waste Disposal by Landfill" [3] document and the National Norms and Standards for Disposal of Waste to Landfill (GN.R. 636 in terms of section 7(1)(c) of the National Environmental Management: Waste, 2008 (Act 59 of 2008) [4].

Based on the foregoing, the scope for this report includes the following aspects:

- Regulatory requirements pertaining to the design and development of the landfill, including the specific requirements of the Waste Management Licence and the initial design approval for Cells 1 and 2 of Area E.
- Site description in terms of geographical and topographical setting, geology and soils, geotechnical and geohydrological conditions.
- Overall layout design of the proposed landfill facility including access, roads, landfill cells, contaminated water and leachate storage, and drainage systems.
- Detailed design of the following components of the Area E landfill:
  - Bulk earthworks and layer-works for the landfill cells;
  - Leachate and contaminated water drainage within the landfill cells, including collection and pump sumps;
  - Landfill barrier details;
  - Surface water drainage systems including diversion berms and drains for upslope run-off and contaminated water;
  - Bulk earthworks and layer-works for the contaminated water pond;
  - Pond barrier details and sump details (leakage and pump sumps);
  - Leachate, contaminated water and subsoil pumping system(s), including sizing and specifications for the required pumps;

- Detail design report and drawings;
- Technical specifications for the civil/earthworks construction and for the liner supply and installation;
- Construction Quality Assurance (CQA) Plan for the construction of the landfill and associated infrastructure, and particular the installation of the barrier system, to ensure the landfill is constructed in accordance with the approved design.

2 REGULATORY REQUIREMENTS AND SITE CLASSIFICATION

2.1 Integrated Environmental Authorisation

The IEA for the development of the Tonk Meter Road Waste Disposal Facility referred to in clause 1.1 above is for a Class B landfill facility, and contained a number of specific conditions relevant to the design and construction of the landfill. These include the following:

"***4.2 Construction and further Development***

a. Verref must, before commencement of the activity, submit the design drawing reports for approval which must demonstrate compliance with the National Norms and Standards for the Disposal of Waste to Landfill, 2013, in respect of requirements including stability, drainage at atmospheric pressure, service life, total solute transport leading to action leakage rates definition, chemical compatibility with geodrains and Geosynthetic Clay Liners and the independent construction quality assurance.

b. Verref must ensure that construction and further developments of new cells are undertaken in accordance with the Class B containment barrier design as indicated in the National Norms and Standards for the Disposal of Waste to Landfill, 2013, and meet the following requirements of the Department of Water and Sanitation: Engineering Services of 19 August 2013:

i. The creep collapse potential of the synthetic drainage material in both the under drains and leachate collection system (LCS) must be shown by Verref to meet the service life and post closure time (ie. 90 year) performance required;

ii. The service life of the LCS for Municipal Solid Waste with respect to precipitate and biofouling be demonstrated or alternatively replaced with prescriptive granular leachate collection system and filter.

iii. Liner temperature prediction is confirmed and monitored; and

iv. The factor of safety for stability on the side walls in particular be confirmed"

All the above conditions were adequately addressed in the detail design of Area E Phase 1 development of the landfill, and approved by way of GDARD's letter of conditional approval of the development of Cell 1 and Cell 2 at the Tonk Meter Resource Waste Disposal Area E Phase 1, dated 30 March 2021.

In August 2024, Tonkmeter submitted PLC Tech Memo P077-TM001-2024 to GDARD and DWS containing some design changes that were incorporated into in the construction of Cell 1A in 2024. No approval for these design changes was received from GDARD, however approval of the Construction Completion Report for Cell 1 was received from GDARD (and DWS) on 29 July 2025.

There are two further conditions contained in the IEA that are addressed in this report. These are:

"***4.1 Scope of the Integrated Environmental Authorisation***

b. The Integrated Environmental Authorisation is for the development and operation of a new, privately owned waste disposal facility and associated infrastructure at the remaining extent of Portion 81 of the Farm Rietfontein 128 IR, which will cover an area of approximately 108 hectares in extent

*with an estimated airspace below ground for the landfill of approximately 13,7 million cubic metres with **final cell height of 1m above ground level**.*
*c. The proposed Class B waste disposal site will allow the disposal of **only** non-recyclable general municipal waste. The proposal will include the development of the following infrastructures:"*

2.2 Landfill Classification

2.2.1 Landfill barrier

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

- National Environmental Management: Waste Act, 2008 (Act 59 of 2008) (NEMWA);
- Waste Classification and Management Regulations (GN R634 of 23 August 2013);
- National Norms and Standards for Assessment of Waste for Landfill Disposal (GN R635 of 23 August 2013);
- National Norms and Standards for Disposal of Waste to Landfill (GN R636 of 23 August 2013); and
- Minimum Requirements for Waste Disposal by Landfill (DWAF 1998) (MR).

The Tonkmeter WDF has been classified as Class B landfill in terms of R636. This means that the landfill will require a Class B barrier system and that it may only receive Type 2 waste.

GN R636 requires a Class B barrier for Type 2 waste, which includes municipal general waste. Figure 2 below shows the requirements for a Class B barrier, as per GN R636.

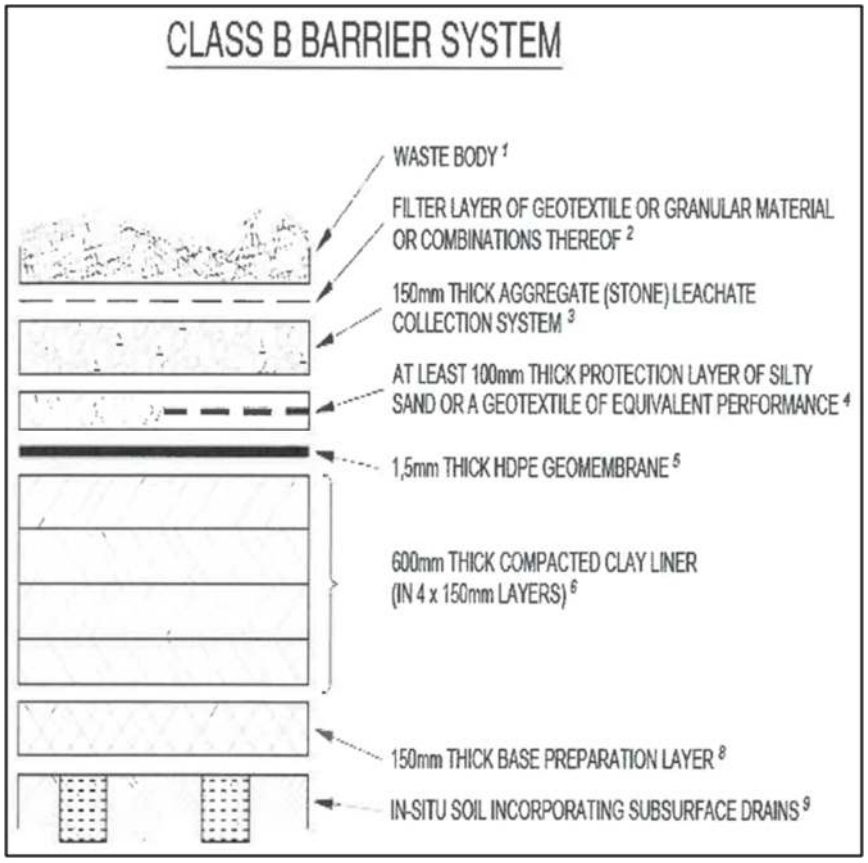


Figure 2: Class B barrier for Type 2 waste disposal

The regulations make provision for the use of alternative geosynthetic materials such as geocomposite drains for stone drains, geotextiles for sand protection layers and geosynthetic clay liners (GCL) for compacted clay liners, provided that the alternative materials can be shown to exhibit equivalent performance to the natural materials indicated.

It is noted that the barrier standards as included in GN R636 provide for containment commensurate with the type of waste as determined by GN R634. In the case of a Class B barrier, the landfill should be able to accept any solid waste that has been classified as Type 2 waste. The Tonkmeter Class B landfill should therefore receive Type 2 wastes, not only general municipal waste.

The operator of the Tonkmeter landfill should however take precautionary measures to ensure chemical compatibility of any industrial Type 2 wastes with the decomposing biological wastes within the landfill. This is necessary to prevent the emission of unacceptable odours from the landfill.

2.2.2 Size of waste stream

At the licencing stage, the forecast initial rate of deposition (IRD) was estimated to be about 20 000 tonnes per month. This translates to 923 tonnes per day based on 260 day per year disposal. This is greater than 500 tonnes per day so that the proposed landfill would be classified as a **Large (L)** landfill in terms of the Minimum Requirements for Waste Disposal by Landfill (1998).

3 SITE DESCRIPTION

3.1 Location, Size and Ownership

The entire site of the Tonk Meter Road WDF is owned by Verref and covers an area of 220 ha, although only 94 ha will be used for the development and operation of the landfill facility. Deep undermining, because of gold mining, has also occurred on parts of the site previously, but the depth of mining was confirmed to be in excess of 1 000 m, making it safe for development of a landfill. Land use surrounding the proposed site is rehabilitated mining areas (i.e. open spaces) to the north, the existing EMM Rietfontein landfill, an industrial area to the east, and residential areas of Kwa-Thema to the west and southwest. The site is located within a highly disturbed area because of previous mine tailings disposal and recovery, current clay mining and previous random waste disposal operations.

3.2 Existing site layout and servitudes

The layout of the site showing the different areas of operation and the various servitudes is shown on Figure 3.

The Rietfontein Road borders the site on the northern side. The main entrance into the Verref and Corobrik brick works is located from the Tonk Meter Road, to the east of the site. Internal roads provide access to the various areas within the site. A gravel track provides access to Area F from the Rietfontein Road.

Eskom power lines (servitude width of 22 m) are located adjacent to the western border of the site, between the Kwa-Thema residential area and Areas D and F. The power line crosses over the north western corner of the site in Area F.

The access road to the first phase development of the EMM Rietfontein Landfill (i.e. Area A) separates Area E from Areas D and C. Adjacent to this road servitude is a servitude for the Rand Water pipeline. A sewer line exists between Areas F and C.

Verref have a registered servitude along the northern boundary of the study area which provides access to Area F during clay mining and landfilling in that area. The same servitude will also serve as an access road between Areas E and F during future operations of the landfill.

Verref have confirmed that the provincial road servitude along the northern boundary of the site is no longer valid since the proposed provincial road has been removed from the provincial road plans.

Area E is located on the eastern side of the property, within a recently mined clay quarry.

Cell 1 of Area E has been constructed and landfilling has reached the level of the interim bench.

3.3 Topography and drainage

The site is relatively flat and with higher-lying areas occurring to the south of the area. The lowest point is at an altitude of 1 612 m above mean sea level (amsl) (northern portion) and the highest point is 1 629 m amsl (southern portion). The natural topography of the study area has been changed due to past and current activities (i.e. clay mining and reclamation of gold tailings). Stormwater accumulated within the Area E quarry and has been pumped out of the two northeastern compartments to facilitate the construction of Cell 1 and Cell 2. Figure 4 shows a topographical plan of Area E with the Cell 1 as-built details.

The site falls within the catchment area of the Kleinblesbokspruit and is situated 800 m southwest of the Kleinblesbokspruit and 1 km northeast of the Withokspruit Rivers.

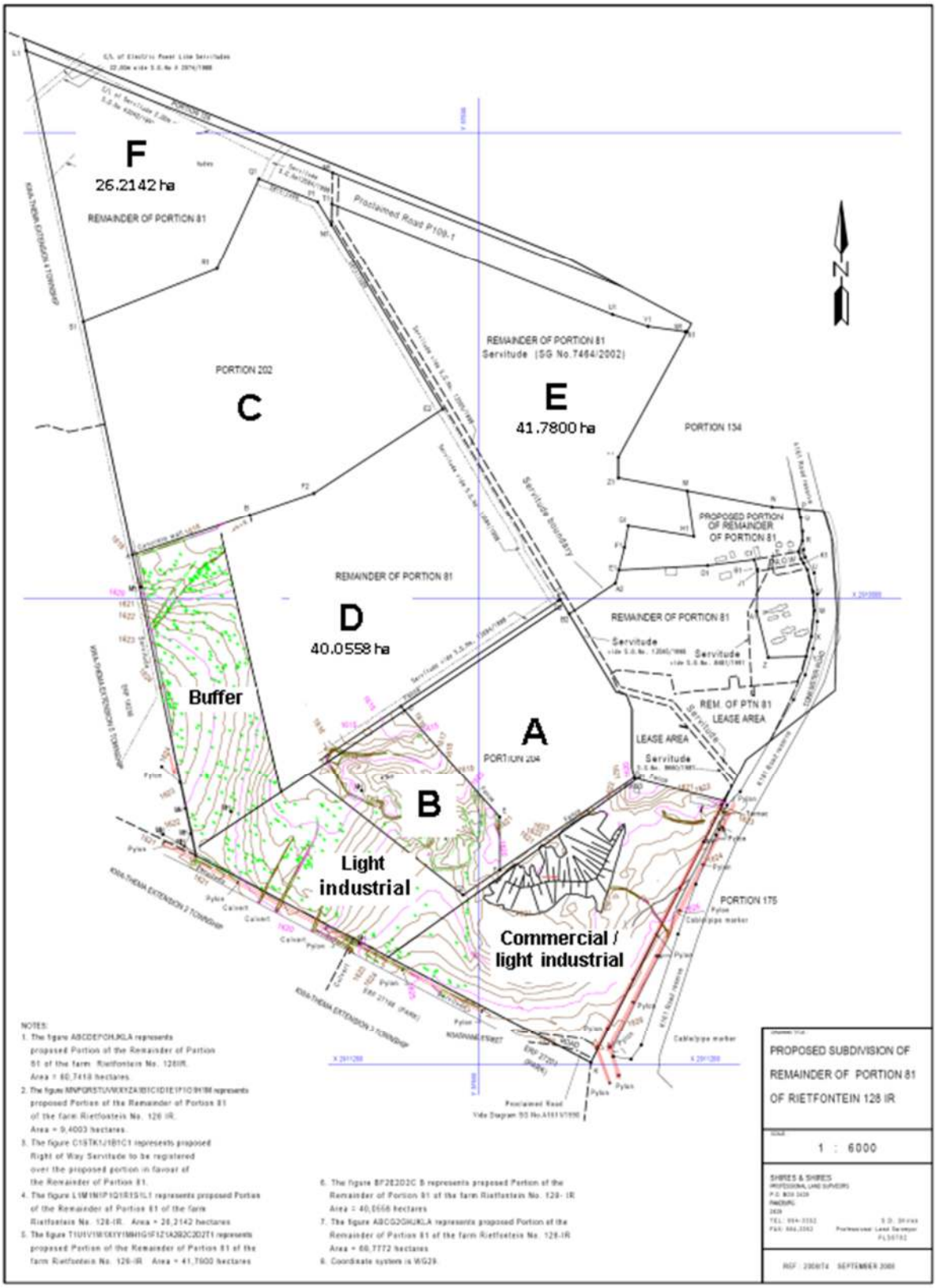


Figure 3: Site layout showing areas of operation and servitudes

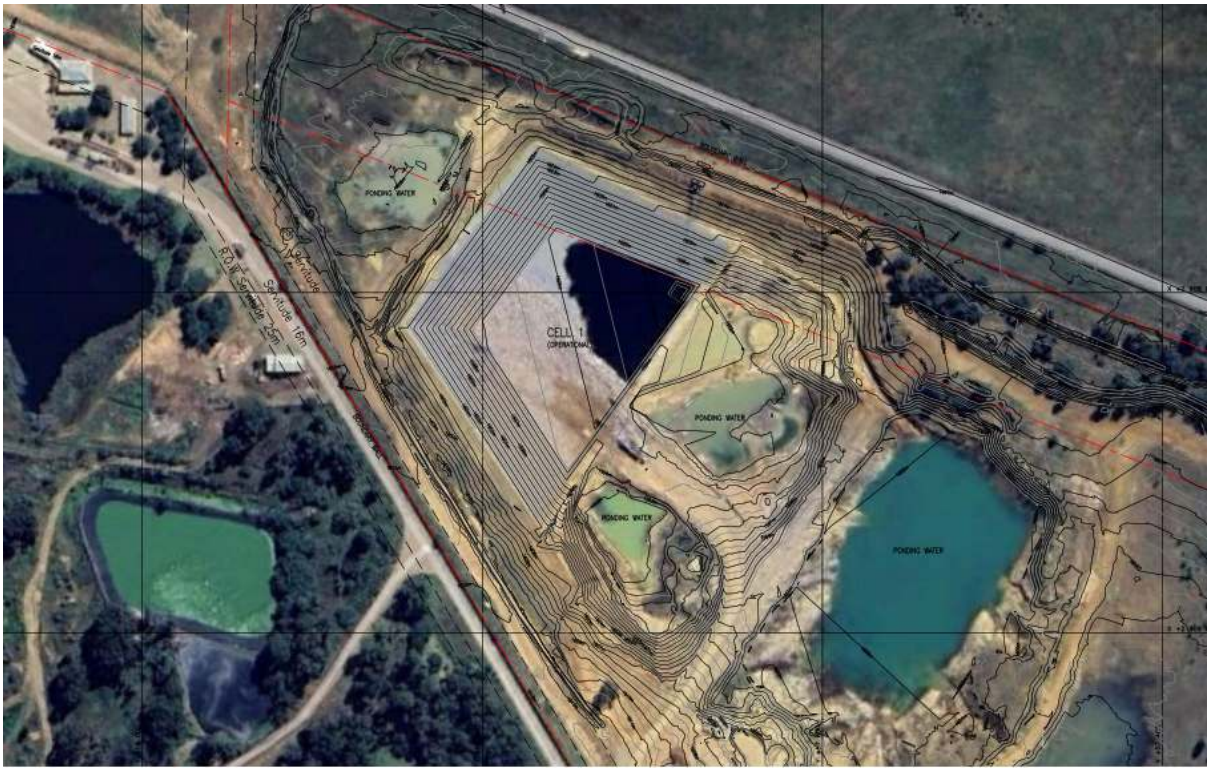


Figure 4: Area E topographical plan

3.4 Climate

The site falls within the summer rainfall region of the Highveld, with warm, wet summers, and mild, dry winters. Winds are predominantly northerly and north westerly during winter and spring (June to November), and north easterly and north-north easterly during summer (December to February).

Air temperatures show significant daily and seasonal variations, with mean temperatures at their maximum in December and January (25 to 27°C), and minimum in June and July (below 4°C). Frosts can be expected between May and September.

Mean annual rainfall is between about 600 and 800 mm, occurring mostly during the summer months between October and April. Intense thunderstorms can be experienced in the late afternoons, sometimes associated with hail. Average annual evaporation is about 1 700 mm. The site therefore has a definite negative climatic water balance.

3.5 Flora and Fauna

The site and its surrounds fall within the Tsakane Clay Grassland veld type, which occurs on flat to gently undulating plains with low hills. Where not removed by previous storage of gold tailings, clay mining or borrowing of road construction material, the vegetation is short and dense grasslands. Alien vegetation species occur within the site due to the land being disturbed by past and present mining operations. The most abundant alien species is the Eucalyptus tree, located mainly within the southwestern section of the study area.

No significant fauna activity is within the areas where mining operations are currently being undertaken. However, small fauna species may occur in previously disturbed areas of the site.

3.6 Geology, Soils and Groundwater

The site has been investigated extensively over several years by various parties. A detailed geological and geotechnical investigation that included a detailed review of previous work as well as additional fieldwork was carried out during 2011 by Dr J P Venter and is documented in the Geotechnical Report dated November 2012[5]. The following section represents a summary of the key aspects of the geotechnical report that affects the landfill design.

3.6.1 Geology

The area of the proposed Tonk Meter Road WDF is underlain by rocks of the Vryheid Formation and Dwyka Group of the Karoo Supergroup. These rocks overlie dolomite of the Chuniespoort Group and the Black Reef Formation of the Transvaal Supergroup. Rocks of the Witwatersrand Supergroup are present at depth. The mudrocks “clays” of the Vryheid Formation have been mined by Verref for the production of refractory bricks. Area E has been excavated by Corobrik to a depth of about 15 to 20 m over most of the area for the “yellow clay” which is used for the manufacturing of building and paving bricks.

Geophysical surveys carried out on lines traversing the site and percussion drilling on anomalies proved that the Karoo rocks are underlain at various depths by dolomite. In some areas dolerite/syenite has intruded into the Karoo rocks and the dolomite and these intrusive rocks are present as irregularly shaped “sills”. Geophysical data and visual observations also indicate the presence of a dolerite/syenite dyke just west of the Rand Water (RW) pipeline servitude striking in a north north-westerly direction.

3.6.2 Soils

A large part of Area E has been excavated for brick and refractory clays. A large portion of this area been “rehabilitated” with so called overburden material i.e. the ferrugenised overlying sands, ferrugenised mudrock and plastic mudrocks overlying the yellow shale (“clay”). Uncompacted backfill is therefore present in the backfilled areas. In some areas temporary dams have been constructed to retain stormwater.

The Dwyka Group sediments depths range from 17 m to 28 m depth, and the thicknesses of the Dwyka rocks vary from 2 m to 9 m.

The depth to dolomite bedrock varies from 20 m to 38 m from original ground surface level. Dolomite residuum is generally absent at the contact area with the Vryheid rocks or Dwyka.

Area E is overlain by Karoo mudrocks. The upper part of the soil and rock profile of the Karoo Supergroup rocks can be seen in the northern quarry face. The profile shows transported (aeolian) sandy material near surface underlain by sandy nodular ferricrete lower down. This changes into ferrugenised residual mudrock followed by mainly structureless reworked slightly ferrugenised clayey silt (completely weathered residual mudrock). All the above is referred to as “overburden” by the quarry operators and varies between 6 m and 8 m depth. This is followed by a soft to very soft rock shale or siltstone (“yellow clay”), in turn followed by a purplish shale (“plastic clay”) with the “fire clays” (a harder mudrock) further down. A borehole drilled just north of the benched face indicated that some carbonaceous shale underlies these sediments in this area. Figure 5 below shows the typical soil profile in the Area E clay quarry.



Figure 5: Area E quarry face profile and positions of samples taken in 2012

3.6.3 Geotechnical evaluation

Soil sampling and testing

As part of the earlier geotechnical investigations, numerous samples were taken for testing from the exposed bench faces in Areas E and F. The laboratory test results are discussed in detail in the Geotechnical Report [5] and the results for Area E are shown in Table 1 below.

As part of the detail design phase for Area E landfill, a composite sample of the yellow and purple residual mudstone clays were taken and subjected to remoulded (consolidated) undrained triaxial testing to determine shear strength parameters. In addition, a falling head permeability test was performed on a remoulded sample of the clay compacted to 95% Standard Proctor density.

Table 1. Summary of geotechnical test results from quarry samples from Area E (2012)

Trial hole	Depth (m)	Description	CBR at 95% 1*	LL 2*	PI 3*	LS 4*	% passing (mm)		GM 5*	Coefficient of permeability cm/s 6*
							0,075	0,002		
TS	0-0,4	Topsoil		26	10	4,0	66	19	0,41	
FS	1,5-2,0	Ferrugenised sand	8	30	14	7,0	22	8	2,17	
FC	4-5	Residual mudrock		44	20	9,5	76	42	0,36	$8{,}2 \times 10^{-7}$
YC(3)*	9-10	Mudrock		32	10	5,0	90	23	0,18	
PC(2)*	11-12	Plastic clay		43	17	8,0	96	49	0,06	$2{,}7 \times 10^{-8}$

Explanation:
*1 California Bearing Ratio
*2 Liquid limit
*3 Plasticity Index
*4 Linear shrinkage
*5 Grading modulus
*6 Coefficient of permeability at 95% Proctor density
*7 Yellow clay (3 samples)
*8 Plastic clay (2 samples)

Table 2: Summary of geotechnical test results for bulk clay sample from Area E (2020)

Sample source	Description	Sample depth (m)	Friction angle Ø' degrees	Cohesion (KPa)	Coefficient of permeability cm/s
Area E north face	Mudrock	10 - 15	29,4°	14,3	$6,7 \times 10^{-9}$

Excavatability of materials

All the excavations for the quarries have been done using large excavators. Some of the materials such as the ferrugenised hardpan ferricrete and the very soft to soft rock mudrocks probably classify as very soft or soft rock and may require power tools and/or ripping.

Materials

Most of the topsoil has been stripped in area E except for the site of the contaminated water dam. This material must be stockpiled for topsoiling outer slopes of berms and embankments.

Cover material

All the "overburden" material overlying the yellow clay, consisting of sand, ferricrete and ferrugenised mudrock can be used as cover material for sanitary landfilling operations. Large quantities of this material have been used in Area E for rehabilitation to the south of the Phase 1 development

Liner material

The permeability results show that the horizons with acceptable results for use as a compacted clay liner material is the residual mudrock and plastic clay with permeabilities between $8,2 \times 10^{-7}$ cm/s and $6,7 \times 10^{-9}$ cm/s. These clay materials occur below the ferrugenised mudrock layer.

Permeability of the underlying material

The materials from a shallow depth consist of ferrugenised or weathered Vryheid Formation mudrocks overlying sandstones and diamictites of the Dwyka Group. Both of these sedimentary strata have low permeabilities. The combination of low permeability weathered mudrock and weathered tillite provides an excellent natural barrier or aquiclude preventing contamination of the water table in the underlying dolomite, making this site very well suited for the purposes of waste landfilling.

Capping material

Clayey soils suitable for the construction of the final landfill capping can be selected and stockpiled if required. Investigations will be necessary to identify the location of suitable material.

Construction materials

The only road construction material identified is the ferrugenised sands, gravels or hardpan situated below the windblown sand layer. The material is however, highly variable and detailed investigations are required to define an area with material of a required quality.

A large stockpile of Witwatersrand rocks which belongs to Afrisam is present south of Area A. Crushed and screened material has been used in leachate collection and leakage detection layers in landfill cells on Areas A and C of the Rietfontein landfill.

3.6.4 Dolomite Stability

A dolomitic stability assessment was carried out in 2010 and 2011 by geophysical traverses and percussion drilling on identified anomalies. The study concluded that the hazard risk for the development of sinkholes and subsidences is low and the site was classified as Inherent Hazard Class (IHC) 1. The site is suitable for the development of waste disposal facilities. The dolomite stability assessment report was submitted to the Council for Geoscience and a letter supporting the proposed Waste Disposal Facility was received (see Appendix A).

In addition, a further dolomite stability review for Area E was carried out in July 2026 by specialist Conulting Engineering Geologists, IntraSolutions. The findings of this dolomite stability review are included hereunder, and the full report is included in Appendix B.

The document presents a review of dolomite stability conditions at the Tonkmeter Waste Disposal Facility, Area E in the context of present Facility Development proposals which entails increasing the height of landfill cells from 1 m above ground to 25 m above ground and associated infrastructure.

The Facility as a whole is a Class B lined landfill designed to provide a reliable alternative to constrained municipal facilities in the area. Area E is 47,3 hectares in extent. The Council for Geoscience supported the development of the larger site as a Waste Treatment Facility in comments dated 23 May 2013, referenced F3849-2 Rem Porton 81, Rietfontein 128-IR. This support is understood to remain in place provided certain conditions are met.

Area E is located on Karoo age deposits, which blanket dolomite rock at intermediate to great depth. In the past clays were quarried from this site. The shallow groundwater levels reported in the 2011 and 2013 boreholes are confirmed by the 2013 geohydrological study to represent a perched aquifer. The groundwater level of the deeper aquifer is 20-40 m deep at an elevation of 1570-1590 m amsl.

Based on available data and current best practice, Area E represents a **low to medium** susceptibility of sinkhole and subsidence formation with respect to ingress of water and a **low** susceptibility of sinkhole and subsidence formation with respect to anthropogenic groundwater level drawdown, i.e. Inherent Hazar Class 1/2//1. The Design Hazard Class is 2. In light of the land use, the Dolomite Area Designation is D3.

3.6.5 Undermining

Deep underground mining activity is a source of low intensity earth tremors in the area, however the Tonk Meter Road WDF site is not within the three areas of greatest seismic hazard in South Africa. The relatively low natural seismic activity and the fact that no major faults have been identified or are mapped in the area, indicates that the potential for reactivation of existing faults or shear zones in the area is low. No geological structural features were found to be present in the vicinity of the site.

Available information and the “shallow” drilling down to the Dwyka Group rocks and dolomite showed that there is no coal undermining. An enquiry to the Department of Mineral Resources, Gauteng revealed that the only mining that has taken place beneath the area is on the Main Reef of the Witwatersrand Supergroup, at a depth of more than 1 000 m below surface.

3.7 Geohydrology

This section includes a summary of the main results of the geological and geohydrological investigation of Portions 81, 202 and 204 of the farm Rietfontein 128 IR, carried out by R Meyer "An Assessment of the Geohydrological Conditions at the proposed Tonk Meter Road Landfill Site, Springs, Gauteng province" dated February 2013 [(6)].

3.7.1 Groundwater

A hydrocensus of the entire site and the surrounding area indicated that in more than 50% of the boreholes drilled (more than 100), no water strikes were reported. In several of the deep boreholes (>50 m deep), no water was encountered during drilling and the boreholes were completely dry at completion of drilling. Most reported water strikes occurred in mudstone of the Vryheid Formation and diamictite (tillite) of the Dwyka Group. Several water strikes were reported at the contacts between different formations or with intrusive dolerite. Only six water strikes in the dolomite were reported. Except for some slight dolomite weathering, no indications or evidence of karstification of the dolomite was observed in any of the 60 boreholes that intersected dolomite.

The majority of the boreholes (more than 90%) had a blow or tested yield of <0.5 ℓ/s. The highest reported blow yield (0.9 ℓ/s) occurred at the intersection with a coal seam in the Vryheid Formation.

Inspections of the clay quarry on Area E revealed that the side walls are dry for most of the year supporting the conclusion that the mudstone of the Vryheid Formation has a low permeability and does not host large amounts of groundwater to support sustainable high yielding boreholes. The water accumulating in the lower sections of the quarry is attributed to rainfall and inflow of storm water during summer months.

The water level measurements collected during the 2011/2012 drilling programmes as well as those from earlier surveys, indicate the presence of two distinctly different ground water level situations:

- A shallow perched water level associated with the mudstone and carbonaceous shale of the Vryheid Formation and dolerite/diabase intrusions; and
- A deeper water level associated with the tillite and dolomite of the Dwyka and Malmani Groups respectively.

The depth of the shallow water level in Area E is approximately 20 m below ground level. Water level depths in the Dwyka and Malmani Group rocks range between approximately 30 m to 40 m below ground surface in Area E.

Similar to the water level distribution for the Vryheid Formation, the Dwyka/dolomite water levels indicate a groundwater movement towards the central and western part of the site, with eventual outflow towards the west.

3.7.2 Groundwater quality

Water quality of both the shallow and deeper groundwater is generally of a good quality. However, boreholes on and close to the areas previously used for gold mining tailing dams have a high electrical

conductivity and low pH, higher concentrations of for example sulphate (SO_4), chloride (Cl) and iron (Fe), that was leached from the large tailings dams and the remnants still present in certain areas.

The two exploration boreholes on the northern side of the Area E quarry are no longer functioning since the quarrying operations extended into this area. Aadditional pairs of monitoring boreholes have been established on the northern and western sides of Area E, to monitor groundwater quality in both aquifers. This will ensure the early detection of any contamination originating from the existing Rietfontein landfill and/or the new Area E landfill sites and that appropriate action can be taken to eliminate or contain possible further contamination.

3.7.3 Aquifer classification

The three criteria recommended by DWS for the classification of aquifers, (i) potential sustained yield, (ii) water quality, and (iii) significance of the aquifer, were used to classify the aquifers in the study area. In terms of groundwater significance and based on the borehole yields and water quality information, the aquifers associated with the Vryheid Formation, and Dwyka and Malmani Groups are classified as "non-aquifers" with low potential sustained yield.

3.8 Discussion

The site is significantly disturbed as a result of previous and current mining activities, so that the development and operation of a formal landfill operation will improve the site in the long term.

From a geological and geotechnical perspective, the deep low permeability residual soils favour the development of a landfill facility. Despite the site being underlain by dolomite at depth, the low permeability sediments provide adequate protection for the groundwater. In addition, it was determined that the hazard for the development of sinkholes and subsidences is low and is therefore suitable for the development of a landfill facility. There is adequate material for daily cover operations as well as suitable clay material for the construction of the compacted clay component of the landfill barrier system.

From a geohyrological perspective, the site is underlain by two aquifers, both of which are classified as “non-aquifers” with low potential sustained yield. The groundwater quality has already been adversely affected by the previous mine tailings activities. The site is therefore considered to be suitable for the development of a landfill facility.

4 DETAIL DESIGN OF THE LANDFILL FACILITY

4.1 Introduction

The detail design Phase 1 development of Area E landfill of the Tonk Meter Road WDF was submitted in October 2020 to GDARD for approval. Conditional approval was obtained by GDARD in March 2021. In August 2024, PLC on behalf of Tonkmeter submitted a Tech Memo to GDRARD that addressed many of the conditions of approval, and also included various design changes. Approval of this Tech Memo and authorisation to construct Cell 1 with the contaminated water pond and leachate tank was received from GDARD in March 2025. Following construction of Cell 1A and its associated infrastructure, approval of the construction completion report was received from GDARD in July 2025. Construction of Cell 1B, completion of the lining from the interim bench up to surface, was completed in September 2026.

This design report for Area E of the Tonkmeter Landfill is presented with a view to providing the details for approval of the increased landfill height and complete Type 2 waste stream, for the further landfill development of Area E. The design is largely based on the current approved design for Area E. A construction quality assurance (CQA) plan also forms part of the submission to the regulatory authorities.

The general objective of the landfill design is to provide a cost-effective, yet environmentally sound and socially acceptable waste disposal facility. More specifically, the design presented is aimed at minimising the potential for pollution of the environment, particularly the soil, ground water and surface water bodies, as well as the air. Due attention is therefore given to the site-specific aspects identified during the on-site investigations.

The overall design makes provision for the phased development of the site, as determined by the waste disposal needs and capital investment by Tonkmeter. This report focuses on the detail design of the complete Area E of the Tonkmeter Landfill.

The layout and details of the design proposed for the Tonkmeter Landfill are shown on Drawing No's P077E-300 to P077E-304 included as Appendix A of this report.

4.2 Constraints and factors affecting the design

Taking into consideration the expected waste disposal operations, the physical conditions of the site, and discussions with the Verref personnel, there are several factors that affect the design philosophy for the Tonkmeter Landfill. These are as follows:

- The design needs to comply with the requirements of the National Norms and Standards for Disposal of Waste to Landfill (GN R636) for a Class B barrier system and the Minimum Requirements for Waste Disposal by Landfill (DWAF 1998) for a G:L:B^+ landfill.
- This design caters for the complete development of Area E of the Tonkmeter Landfill as well as associated infrastructure such as the leachate, contaminated water and subsoil drainage extraction systems, contaminated water pond and leachate tank, as well as access and service roads.
- The residual mudrock soils exhibit very low permeabilities and are therefore suitable for supplementing the barrier system.
- Due to the abundance of overburden from the mining operations, there is more than adequate soil for use as cover in the landfilling operations.

- The Area E quarry currently has compartments of ponded stormwater which will have to be pumped out before commencement of construction the cells in those areas.
- The design caters for a final landfill height of 25 m above natural ground level.
- The separation of clean storm water, contaminated runoff water and leachate is critical in the design of a facility of this nature, especially with the cells being below ground level.

4.3 General site layout

Based on the aforementioned requirements and constraints, the overall layout of Area E of the Tonkmeter Landfill and the access road from the site entrance has been developed as shown on the attached drawings in Appendix A. Figure 6 below indicates the current layout of the operational Cell 1, as well as the partially backfilled quarries to the southeast.

Figure 7 shows the proposed overall development layout for Area E.

Initially two cells will be developed and lined. Disposal will commence in Cell 1 with the Cell 2 acting as a contaminated water buffer pond from where water will be pumped into the contaminated water pond near the entrance to the site. Once landfilling in Cell 1 approaches final height, Cell 3 will be constructed Cell 2 will be landfilled, with Cell 3 operating as the contaminated water buffer pond.

Contaminated water and leachate will drain towards sumps within the operating cells from where it will be pumped into either the contaminated water pond or the above ground leachate tank. Storm water will be diverted around the excavated area by way of the surface perimeter berm.



Figure 6: Area E current development

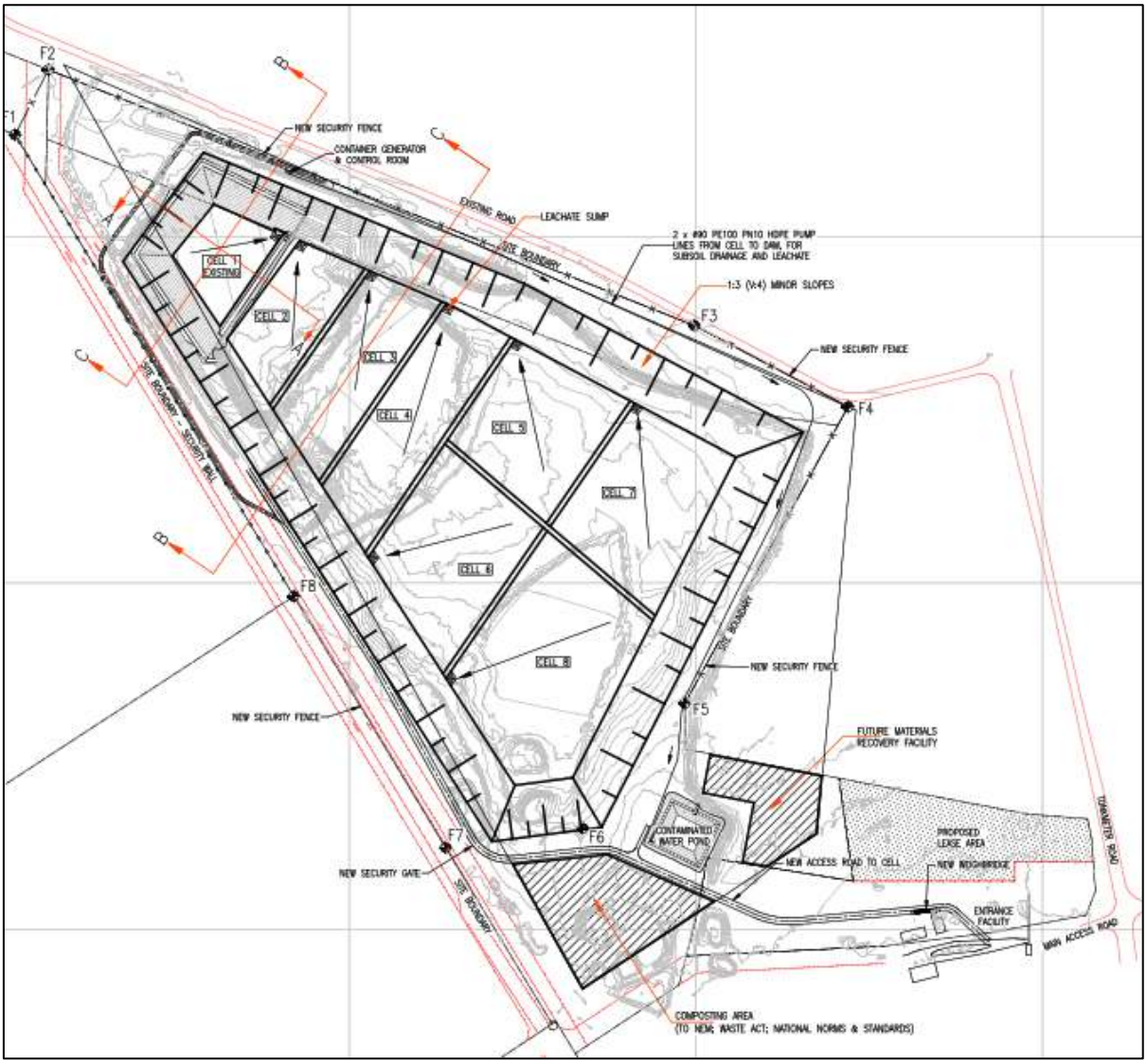


Figure 7: Area E overall site layout

4.4 Services and infrastructure

4.4.1 Access and Roads

Access to the site is from Tonk Meter Road using the existing entrance to the Verref property. A new gravel road has been constructed from the Verref entrance area along the north side of the buildings to join up with the existing gravel road running in a north westerly direction along the RWB servitude. This road has been upgraded to cater for the increased traffic load of the waste delivery vehicles.

Incoming vehicles are checked at the existing Verref gatehouse for the type of waste being delivered. Potentially hazardous waste loads are redirected to an appropriate site to ensure that waste disposed of on Tonkmeter Landfill complies with the waste stream allowed for disposal in a Class B landfill. From there, the vehicles are directed to the active tipping area at the waste cell in Area E.

4.4.2 Entrance Facility

A new weighbridge has been installed at the entrance area. Incoming loads are monitored and logged by means of dedicated weighbridge software to ensure proper recording of waste categories, waste

delivery vehicle details and waste mass. This system is also be used to bill the customers using the landfill.

The existing vacant Verref offices and buildings have been upgraded and used for the landfill staff facilities, including administration offices, ablution facilities, showers, locker room, mess room and kitchen.

Similarly, the existing Verref maintenance workshop is used for maintenance of the landfill equipment and vehicles.

Resource recovery of recyclable materials is being undertaken in one of the existing Verref buildings on site.

Provision is made for the establishment of a formal materials recovery facility (MRF) and a composting facility in the areas demarcated in Figure 7.

4.4.3 Fencing and security

The entire perimeter of Area E as well as the access road from the weighbridge is fenced with a 2.1 m high concrete palisade fence to prevent unauthorised access. Lockable vehicle access gates are to be provided at the entrance.

4.4.4 Water

Potable water for domestic use is obtained from the existing Verref facilities.

Water required for operations is obtained from the open excavations and the contaminated water pond should the quality be acceptable. For dust control purposes on the lined waste disposal cell area, water from the contaminated runoff water ponds will be used for dust suppression. Alternatively, accumulated water rainwater from southern portion of the Area E clay quarry can also be used.

4.4.5 Electricity

There is a three phase 380-volt electrical supply at the Verref facilities, however a generator has been installed at the Area E Phase 1 landfill for operating the various pumps. Solar powered security lights have been installed around the cell.

4.5 Landfill Geometric Design

Based on the foregoing constraints, the overall layout of Area E has been developed as shown on Drawing Nos P077E-301 to P077E304 and in Figure 7 above. The layout and arrangement have been determined according to topography, drainage requirements, practicality and operational requirements.

Adjacent cells are to be developed from the northern section of Area E clay quarry. Each cell adjacent to the operating cell will act as an interim contaminated runoff buffer pond, from where the contaminated water will be pumped and stored in a lined contaminated water pond near the entrance of the site. An access ramp will be constructed down into each developed cell for waste delivery vehicles.

4.5.1 Cell development

The cells will be developed be shaping the inside slopes of the current quarry side walls to internal slopes of 1:3 (V:H) with a 4,5 m wide bench that serves as an intermediate anchor trench half way up

the slope. This bench will also be used to divert clean upslope runoff into the open quarry to the south to reduce the amount of water entering the waste body, until such time as the landfilling operations reach this level. Thereafter the upper slopes will be lined to surface.

The floor of the quarry will be shaped to drain at 2% slope towards two sumps located in each cell on the eastern side. An internal berm 2 m high is to be constructed across the prepared floor of the cells to separate adjacent cells. A high berm is constructed across the south eastern side of Cell 2 to separate the cell from the quarry behind it. A surface perimeter berm is to be constructed around the top of the cells at ground level. This will prevent surface runoff from entering the cells and will also accommodate the upper liner anchor trench. The bulk of the cell earthworks will be constructed using the residual mudstone clayey soils compacted in layers to 95% Proctor density at OMC to OMC +2%.

The plan layout for Cell 1 and Cell 2 is shown in Figure 8 below. A long section through Cell 1 and Cell 2 is shown in Figure 9, while a cross-section through Cell 1 is shown in Figure 10.

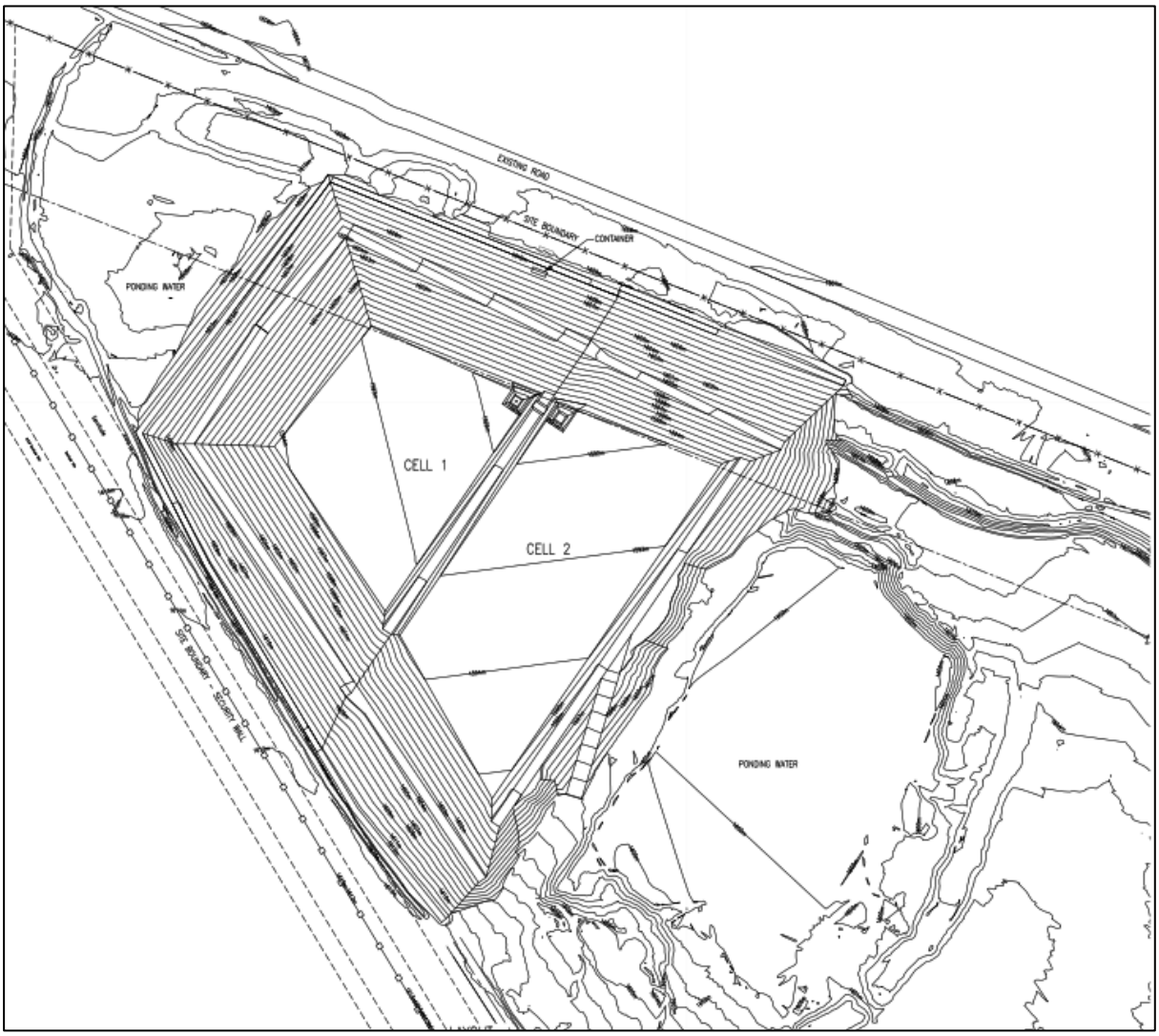


Figure 8: Cell 1 and Cell 2 layout

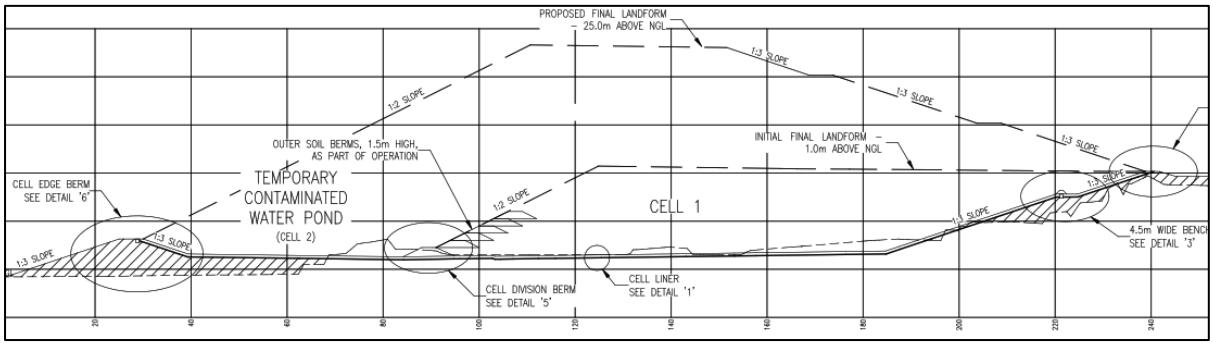


Figure 9: Section through Cell 1 and Cell 2

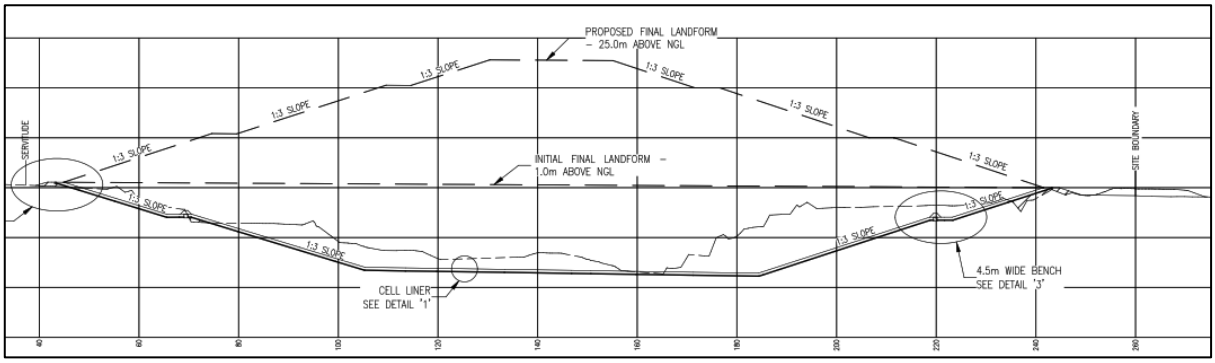


Figure 10: Cross-section through Cell 1

4.6 Subsoil drainage

A subsoil drainage system will be installed below the liner to drain any sub-surface seepage water from beneath the liner to the subsoil collection sump, from where the water will be pumped either to the environment if it clean water, or to the leachate tanks if any leakage of leachate through the liner system is detected.

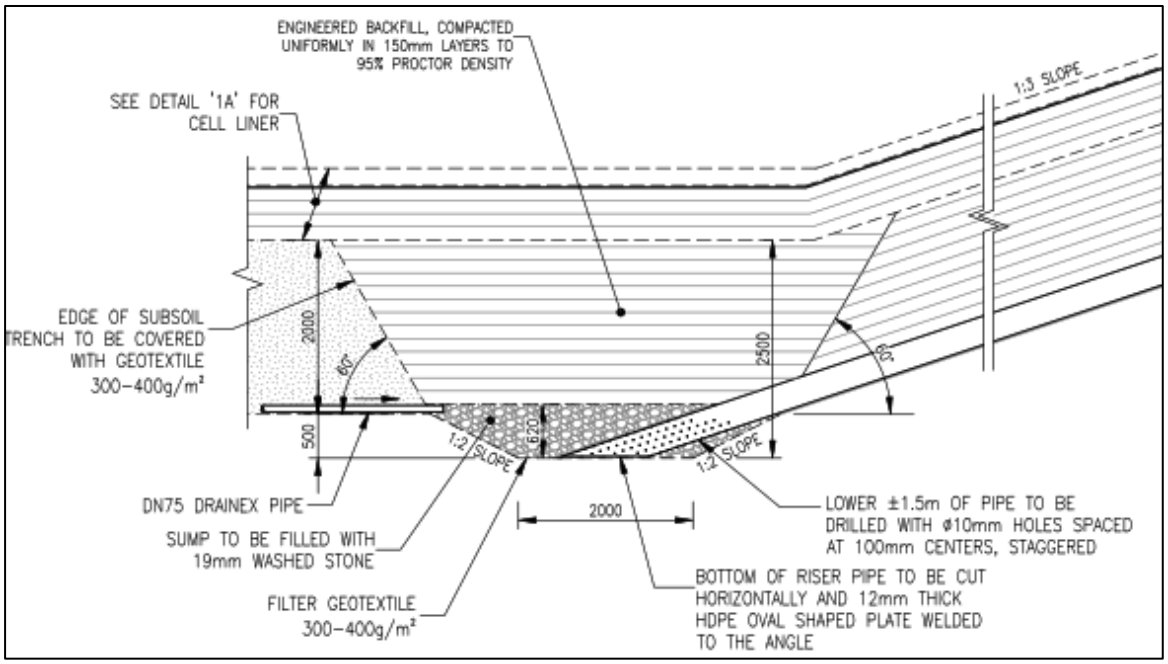


Figure 11: Subsoil drainage sump and extraction riser pipe

The subsoil system will consist of a herring bone system of slotted HDPE pipes with 2m high vertical "fins" of HDPE geonet, all wrapped in filter geotextile and set in 2 m deep trenches filled with river sand as shown in Figure 13. These pipes all drain to the collection sump from where subsoil seepage is extracted using a borehole pump installed within an inclined riser pipe (see Figure 11). The as-built layout of the subsoil drains in Cell 1 is shown in Figure 12.

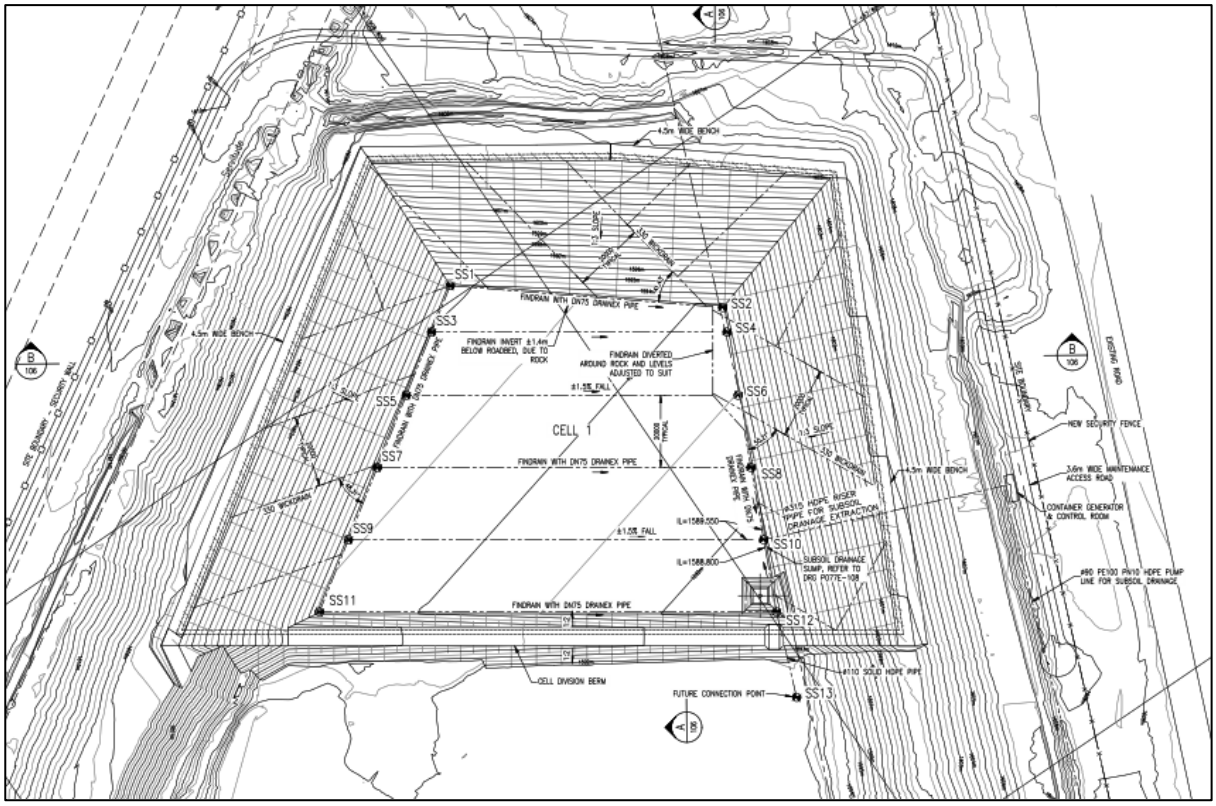


Figure 12: Cell 1 as-built subsoil drainage layout

4.7 Landfill Barrier Design

The Class B equivalent barrier as installed in Cell 1 is shown in Figure 13 below.

- The base of the excavation was ripped, graded to falls and compacted to 95% Standard Proctor density at a moisture content between 0 and +2% of optimum moisture content.
- The 2 m deep subsoil "fin" drains were installed as described above.
- The compacted clay liner (CCL) was constructed in 4 x 150 mm thick layers compacted to 98% Standard Proctor density at a moisture content between 0 and +2% of optimum moisture content.
- The geomembrane liner is a 1,5 mm thick HDPE dual textured geomembrane on the base and single textured on the side slopes that complies with GRI-GM13 (2025), and with a minimum asperity height of 0,4 mm.
- The liner protection geotextile is a nonwoven continuous filament needle-punched polyester or polypropylene geotextile with a unit mass of 1 500 g/m^2.
- The leachate collection and drainage layer consists of 200 mm thick layer of washed single sized 28-38 mm stone.

- The leachate drainage system on the base consists of a herring-bone system of drilled 90 dia HDPE class PN10 pipes at 10 m centres, connected to drilled 160 dia HDPE class PN10 collector pipes that discharge into the leachate sump.
- The filter and separation geotextile between the leachate collection layer and the overlying waste is a UV stabilised nonwoven needle punched geotextile with a unit mass of approximately 300 g/m^2.
- A 1m thick pioneering layer of waste was placed over the filter and separation geotextile.

For Cell 2 and the remainder of Area E development, the CCL is to be replaced with a reinforced geosynthetic clay liner (GCL) in accordance with GRI-GCL3 (2019) with minimum sodium bentonite content of 3 700 g/m^2.

The proposed Class B equivalent barrier for the remainder of Area E cells is shown in Figure 14 and Figure 15 below.

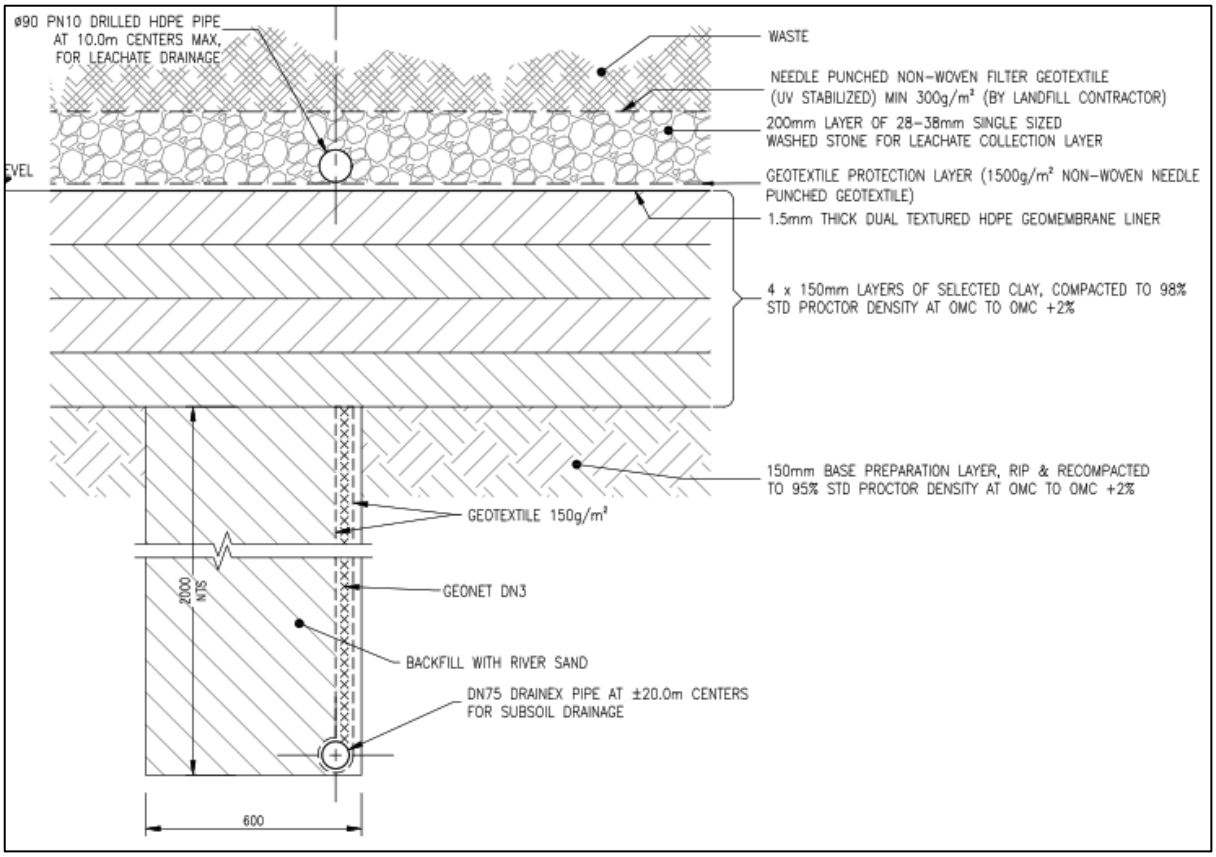


Figure 13: Class B barrier for Cell 1 of Area E landfill base

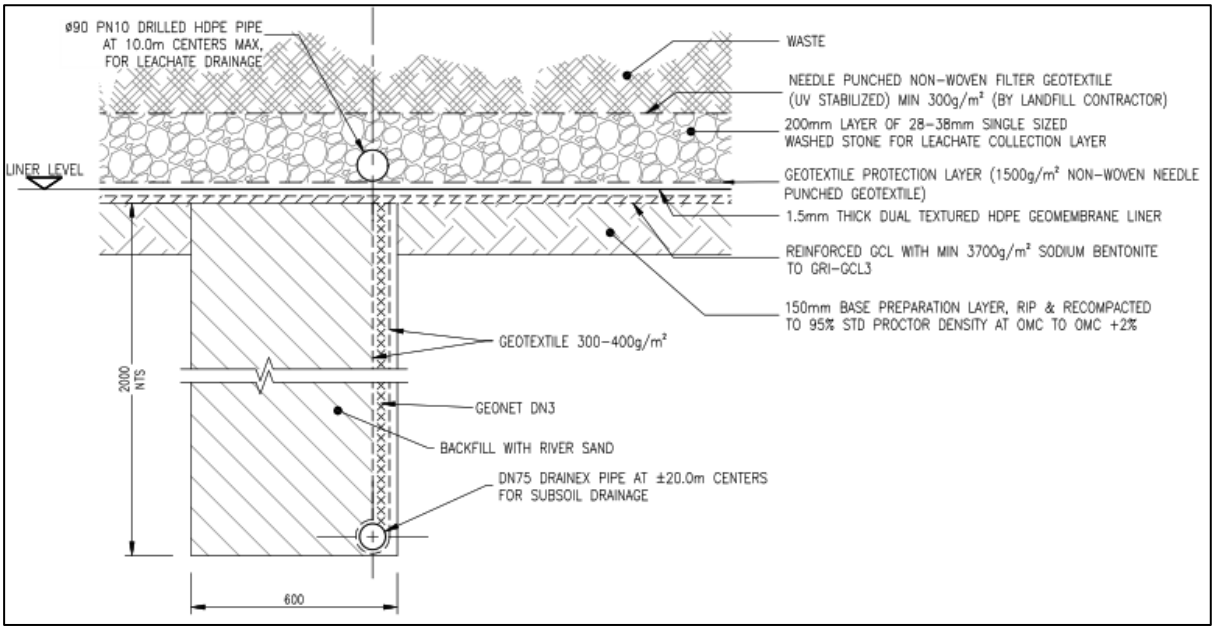


Figure 14: Class B equivalent barrier for remainder of Area E landfill base

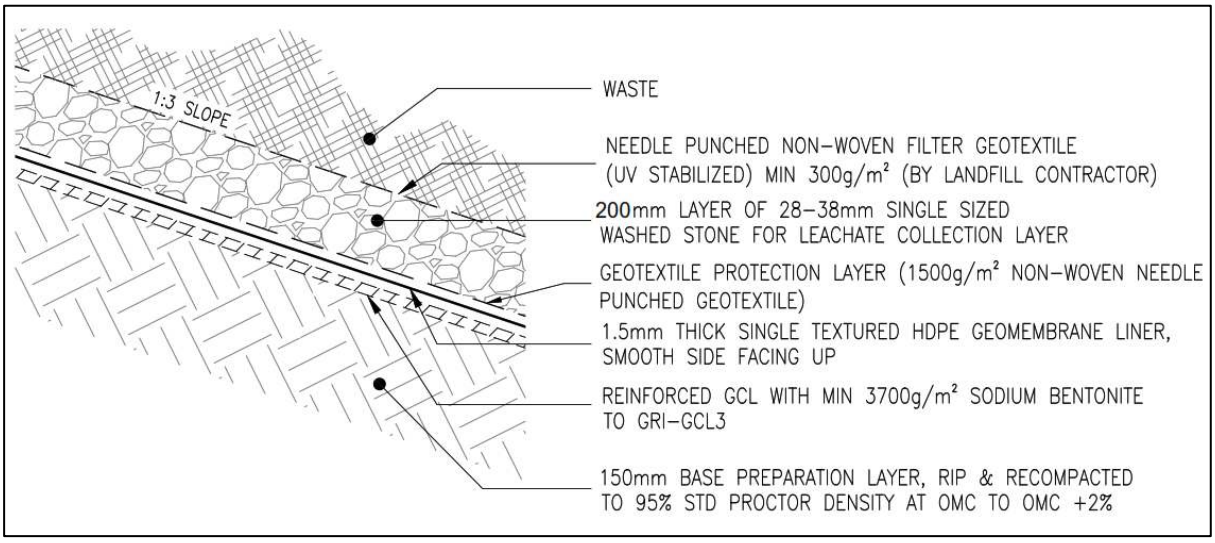


Figure 15: Class B barrier for remainder of Area E landfill side slopes

4.7.1 Barrier service life assessment

The service life of a geomembrane liner can be defined as the length of time the liner continues to act as an effective hydraulic barrier for the purposes of the site under consideration. This will depend upon the circumstances at the site and, for groundwater quality, the acceptable amount of leakage of specific contaminants. One of the main factors influencing the service life of a geomembrane in a basal liner such as beneath the Tonkmeter landfill is elevated temperature. In the case of an exposed geomembrane the main factor affecting service life is UV exposure as well as temperature.

In the case of the Tonkmeter landfill, it is unlikely that the basal liner will experience temperatures greater than 35°C, if there is no accelerated biodegradation as result of leachate recirculation through the landfill. Based on research work conducted by the Geosynthetics Institute in Philadelphia, USA, (GRI White Paper #6, 8 February 2011 - Geomembrane Lifetime Prediction: Unexposed and Exposed Conditions) a good quality HDPE geomembrane that is not exposed to UV, will have a service life in excess of 106 years at 35°C. This is based on the HDPE geomembrane complying with the

requirements of specification GRI-GM13 (Rev 14 – 2025). This service life is considerably longer that the expected site life of the Tonkmeter Area E landfill which should not exceed 30 years plus a 30-year maintenance period after closure.

The liner protection geotextile to be installed is a nonwoven continuous filament needle-punched polyester geotextile. Based on recent durability laboratory test work conducted by SKZ Germany on Kaytech's Bidim polyester geotextiles, the liner protection geotextile is expected to have a service life more than 80 years at 35°C.

4.7.2 Estimated leakage rates

Since a Class B barrier does not have a formal leakage detection system, it is not necessary to determine the action leakage rate. However, the estimated leakage rate through the liner system has been calculated based on the method proposed by Rowe (2012) "Short and Long-term leakage through composite liners. 7th Arthur Casagrande lecture" [7]. This method is based on using Rowe's equation and input based on the following estimated parameters:

- Assume that all defects are repaired after ELLS, so the cell barrier starts with zero leakage.
- Long-Term Assumption: It is assumed that 1 defect per hectare per wrinkle could occur over time.
- Liner System Components (Cell 1):
  - 1.5 mm geomembrane
  - CCL with a permeability (k) of $1{,}0 \times 10^{-8}$ m/s
  - Or GCL with a permeability (k) of $5{,}0 \times 10^{-10}$ m/s (as per GRI-GCL3)
- Wrinkle Characteristics:
  - 200 mm width of wrinkle
  - Length of interconnected wrinkles between 10 m (excellent contact) and 20 m (good contact)
  - Hydraulic Head: A head loss of 800 mm across the composite liner system, assuming a 200 mm leachate depth.
- Liner Contact Quality: Based on continuous CQA during the construction of the barrier, controlled placement of the stone layer, and the ELLS results, good to excellent liner contact was achieved.

Seepage Estimation (CCL):

Using the above parameters, the estimated leakage rate of leachate through the Cell 1 liner system is projected to vary between **39 ℓ/ha/day** and **78 ℓ/ha/day** over the landfill's lifespan.

Seepage Estimation (GCL):

Using the above parameters, the estimated leakage rate of leachate through the Cell 1 liner system is projected to vary between **12 ℓ/ha/day** and **24 ℓ/ha/day** over the landfill's lifespan.

Note that the quality of the water in the subsoil drainage system will be monitored routinely as part of the operational monitoring protocol for the landfill, to determine whether there is any leachate leakage through the liner system

4.7.3 Geotextile as Liner protection layer substitute for 100 mm sand layer

Due to the practical problems associated with the placement of a protective soil layer over an installed geomembrane without risk of damage to the geomembrane, the soil layer on the base of the landfill is

replaced with a heavy protection geotextile (nonwoven continuous filament needle-punched polyester or polypropylene geotextile with a unit mass of 1 500 g/m^2). The 200 mm leachate collection layer of 38 mm single sized washed aggregate will be placed over the geotextile protection layer. It is believed that this alternative will restrict the strain in the geomembrane to less than 3%, under a possible maximum pressure of 400 KPa.

Note that, as part of the CQA Plan for Cell 1, a static load test was carried out on the installed barrier system with the 1 500 g/m^2 protection geotextile and the 38 mm stone, under a 24 hour sustained load of 400 KPa. This load represents a landfill waste height of 40 m at a landfill density of 10 KN/m^3. The test result yielded a maximum strain of 0,99% according to the method of Tognon et al. This is significantly less than the maximum strain threshold of 3%.

4.7.4 Geosynthetic Clay Liner (GCL) substitute for Compacted Clay Liner (CCL)

Due to the absence of sufficient clay to construct the approved CCL component of the Class B barrier for the remainder of Area E landfill development, it is intended to substitute the CCL with a geosynthetic clay liner (GCL) of equivalent performance.

In terms of the National Norms and Standards for Disposal of Waste to Landfill (GN R636, 2013) and the Guideline for Pollution Control Barrier System Design issued by the Department of Water and sanitation in 2021, the maximum allowable permeability for the compacted clay liner component of a Class B landfill barrier is 10^{-7} cm/s (10^{-9} m/s). The most suitable substitute for a natural compacted clay liner would be to use a geosynthetic clay liner (GCL). The GCL consists of a sodium bentonite clay, sandwiched between two geotextiles. The swelling property of the sodium bentonite when hydrated provides the GCL its low permeability and self-healing properties. Under ideal conditions, the GCL generally has a saturated permeability of less than 10^{-11} m/s. On average the GCL is approximately 6 mm thick. This very thin and very low permeability clay is assumed to provide permeability and attenuation equivalent to the four compacted clay layers (CCL) specified in the Class B barrier system, and it is therefore often acceptable to replace the CCL with GCL.

However, the equivalency of the GCL with a CCL could be affected if the performance of the GCL is significantly reduced; for instance due to inorganic chemical attack or cation exchange. It is therefore necessary to assess the chemical compatibility of a GCL prior to its usage in the barrier system.

GCLs have been widely used in municipal solid waste landfills worldwide as replacements for CCLs. The average permeability (k) of a pre-hydrated GCL is in the region of 2 x 10^{-11} m/s for typical confining stress levels. Depending on stress levels and levels of hydration, as well as possible cation exchange of the sodium bentonite within the GCL, k may increase by up to two orders of magnitude but will still be very close to the acceptable limit of 1 x 10^{-9} m/s for a Class B barrier CCL.

In terms of equivalency, For a 600mm thick CCL, the permittivity (ψ) = k/t (where t =thickness of the liner) = 1.67x10^{-9} s^{-1}.

For a GCL to be equivalent to a 600 mm CCL in terms of hydraulic conductivity,

k_{max} for the GCL = ψ.t(gcl) = 1.67x10^{-9} s^{-1} x 0.01 = 1.67x10^{-11} m/s.

The hydraulic conductivity of a 4 000 g/m^2 GCL is typically ≤ 2.56x10^{-11} m/s so that a standard GRI-GCL3 GCL with 4 000 g/m^2 of sodium bentonite is considered to be equivalent to a CCL for a Class B barrier.

The use of a GCL to replace the 600 mm CCL is therefore considered to be acceptable. In order to provide support for the GCL and a source of moisture for hydration of the GCL, it is intended to construct a 150 mm layer of clayey soil compacted to 95% Proctor density at a moisture content between optimum and optimum +3%.

Note that wastes containing significant concentrations of calcium and/or magnesium will be avoided so as to mitigate the risk of cation exchange of the sodium bentonite within the GCL.

4.7.5 Leachate drainage pipes

The leachate drainage layer on the base of the landfill has a herring bone system of 90 mm dia drilled HDPE pipes Class PN10 pipes at 10 m spacing, that feed into collector mains of 160 mm dia drilled HDPE pipes Class PN10 pipes, that discharge into the leachate extraction sump at the lowest point of the cell. All the pipes will be butt welded together and tested after welding. The leachate conveyance pipes are to be Class PE100 to SANS 4427-1 (2008).

HDPE pipes are known to be chemically resistant and commonly used in leachate conveyance applications at landfills due to their expected long service life. The PE100 pipe is the highest stress value pipe that can be specified to ensure the best possible service life. Furthermore, because a much more crystalline formula is used in the manufacturing of HDPE pipes compared to geomembranes, they are limited by strain and not temperature.

As long as the leachate extraction pump system operates, the leachate conveyance pipes should operate at atmospheric pressure.

4.7.6 Leachate drainage layer veneer stability

The veneer stability of the leachate drainage layer on the 1:3 side slopes as well as the adequacy of the anchor trench has been assessed. Using the tensile strength of the protection geotextile at less than 10% strain, the FoS against sliding down the smooth geomembrane is 1,60. The FoS for the anchor trench resistance against pull-out is 1,61.

The stability of the leachate drainage layer on the side slopes has therefore been confirmed. This has also taken account of construction equipment working on the slope (8T Dozer).

During the construction of Cell 1, no veneer stability problems were experienced during the placement of the stone layer.

4.8 Leachate management system

The leachate management system for Area E consists of the following components:

- A leachate collection and drainage system for each cell;
- A leachate sump, penstock and pump riser pipes for each cell;
- A leachate pump system for each cell; and
- Leachate storage tank that serves all cells.

Any leachate emanating from the waste in the landfill would report to the 200 mm thick leachate collection and drainage layer, and from there it will drain under gravity via the leachate drainage pipes into the leachate sump in the lowest corner of each cell. The leachate sump consists of a 1 200 mm diameter structured walled HDPE prefabricated manhole that is founded on a 1 m thick reinforced

concrete base with the liner extending below the concrete base. The manhole is surrounded with stone aggregate. The leachate sump is fitted with two 315 dia HDPE inclined riser pipes up to ground level as pump sleeves. The lower end of each pipe is drilled to allow leachate to flow into the pipe and the bottoms are blanked off with HDPE plate. As the level of the landfill rises, the leachate sump will be raised as a vertical penstock using additional lengths of the same HDPE structured walled pipe. Drilled to allow leachate to drain into the sump. The purpose of the penstock is to drain away surface water from the waste body in the event of an intense rainfall event and also to provide a redundancy for pumping out leachate should the inclined pump riser pipes become unusable.

The concrete base for the penstock system has been designed to prevent the penstock tower from punching through the liner because of down-drag on the penstock tower from the consolidating waste body.

Leachate will be removed from the landfill by pumping it from the sump by means of a submersible pump inserted down one of the riser pipes to the leachate storage tank (see Section 4.10). Stored leachate will then be discharged in a controlled manner from the leachate tanks into the nearby sewer, with the necessary authorisation of the local authority.

Details of the leachate sump and penstock are shown in Figure 16 below and on Drawing P077E-210.

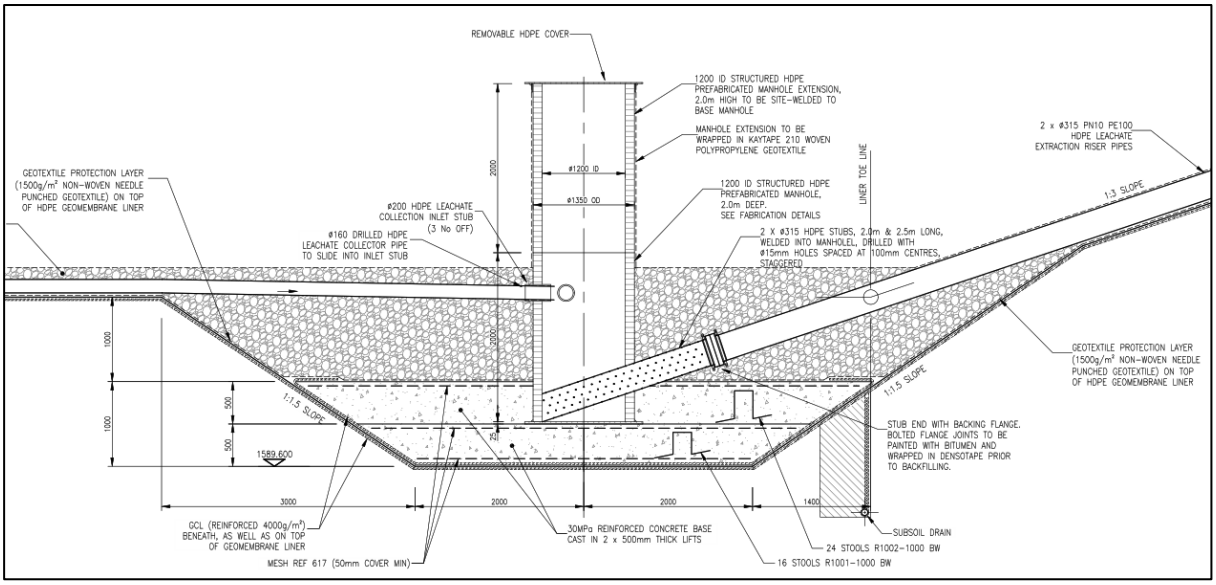


Figure 16: Leachate sump, penstock and pump riser pipes

4.9 Contaminated water management system

The contaminated water management system consists of the following components:

- A cell that acts as a contaminated water buffer pond;
- A contaminated water sump, penstock and pump riser pipe;
- A contaminated water pump system; and
- A contaminated water pond.

The first component of the contaminated water system is the adjacent unused cell which for the purposes of the design of Phase 1 will be Cell 2. Surface runoff from the waste body in Cell 1 will flow into Cell 2 from where it will be pumped into the contaminated water pond. Initially while the landfill level in Cell 1 is below the internal berm between Cell 1 and Cell 2, contaminated water and leachate

from Cell 1 will be pumped from the Cell 1 leachate sump. Water collecting in Cell 2 will be clean initially and can be pumped out to the environment. Once the waste level extends above the internal berm level, contaminated water will be pumped from Cell 2 into the contaminated water dam, and leachate from Cell 1 will be pumped into the leachate tanks. The sump, penstock and pump riser pipe in Cell 2 is identical to the one in Cell 1.

4.10 Contaminated water pond and leachate storage tank

The contaminated water dam and leachate tanks are located on surface on the north west side of Cell 1. The leachate storage tank is an HDPE lined steel tank with a capacity of 238 000 ℓ. This tank has been installed on the bunded and lined area adjacent to the contaminated water pond, so that any leakage that might occur from the tank will drain into the pond.

Refer to Figures 17 and 18 and Drawings P077E-115 and 116 for the layout and details of the contaminated water dam and leachate tank.

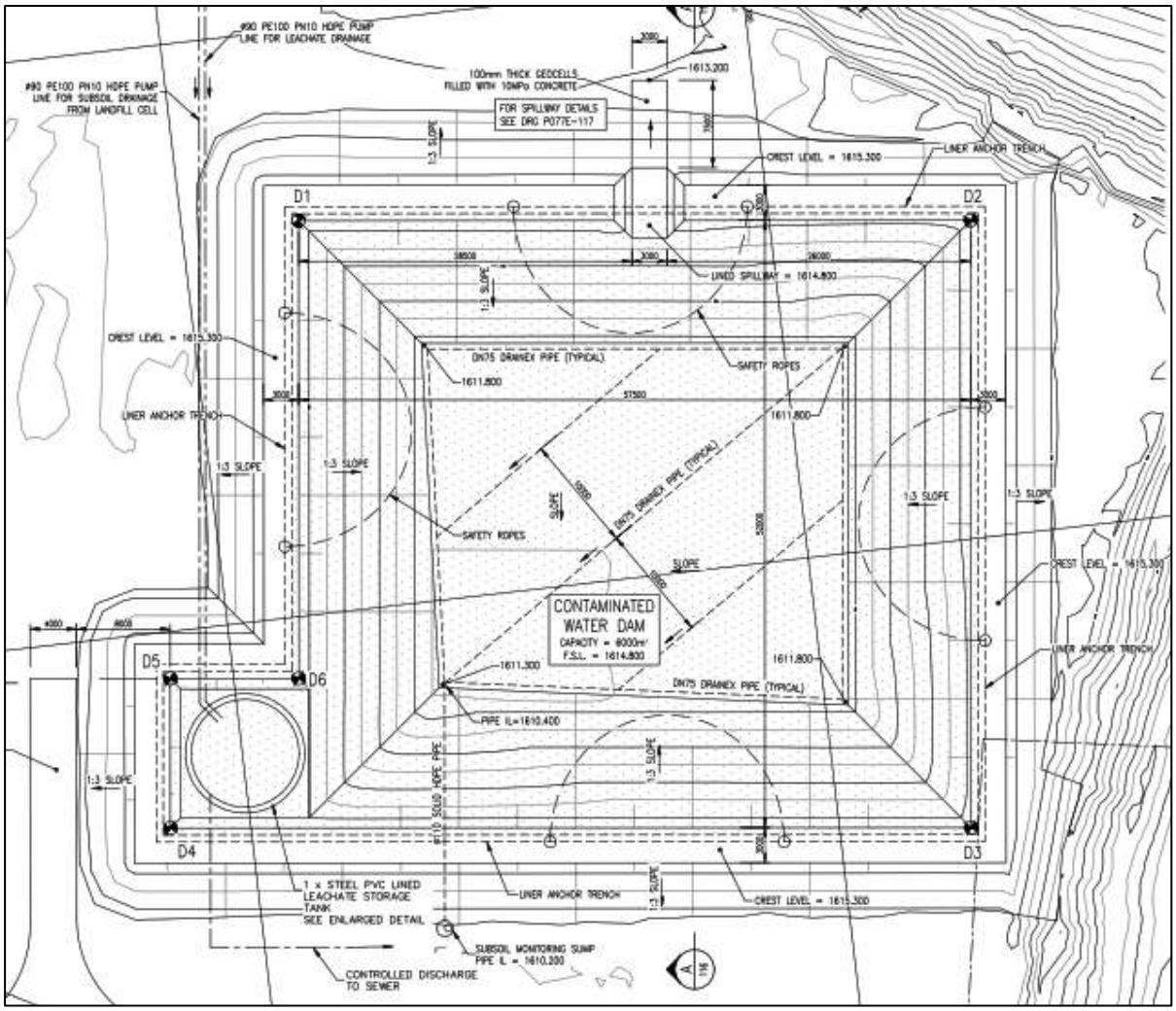


Figure 17: Layout of Contaminated water pond and leachate tank

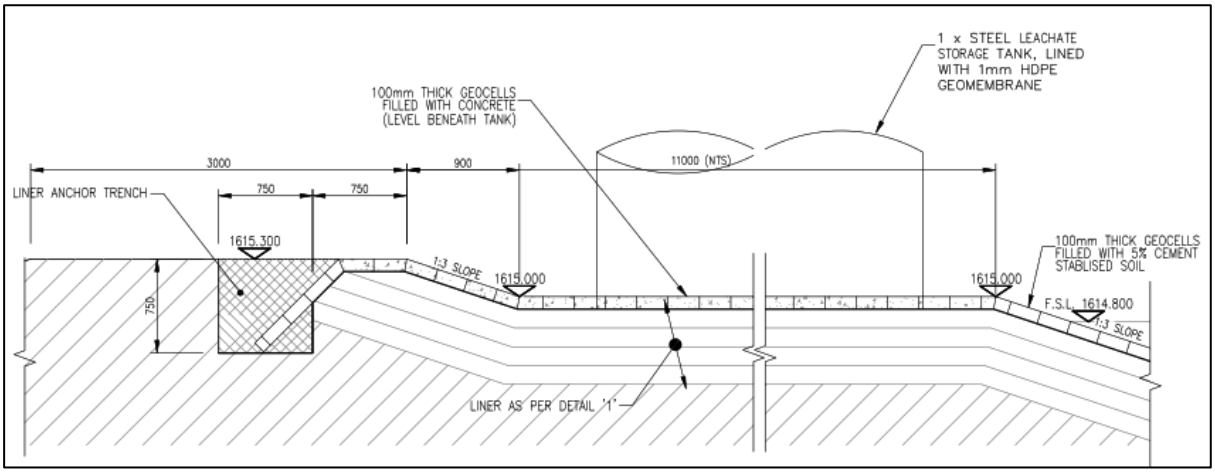


Figure 18: Leachate tank bunded area

The contaminated water pond has been sized to contain the 1 in 50 year recurrence interval 24 hour duration storm run-off from two consecutive cells.

Mean annual rainfall = 700 mm

1 in 50 yr 24 hr storm = 160 mm

Area of Cells 1 & 2 = 49 400 m^2

C (waste and pond) = 0,7

Require storage volume = **5 530 m^3**

Capacity of Cell 2 at 2m depth = 13 500 m^3

Capacity of contaminated pond = **5 490 m^3**

Cell 2 therefore provides adequate buffer storage capacity for the design storm.

The liner design for the contaminated water pond is also a Class B equivalent barrier but modified to provide protection of the liner. Refer to Figure 19 below.

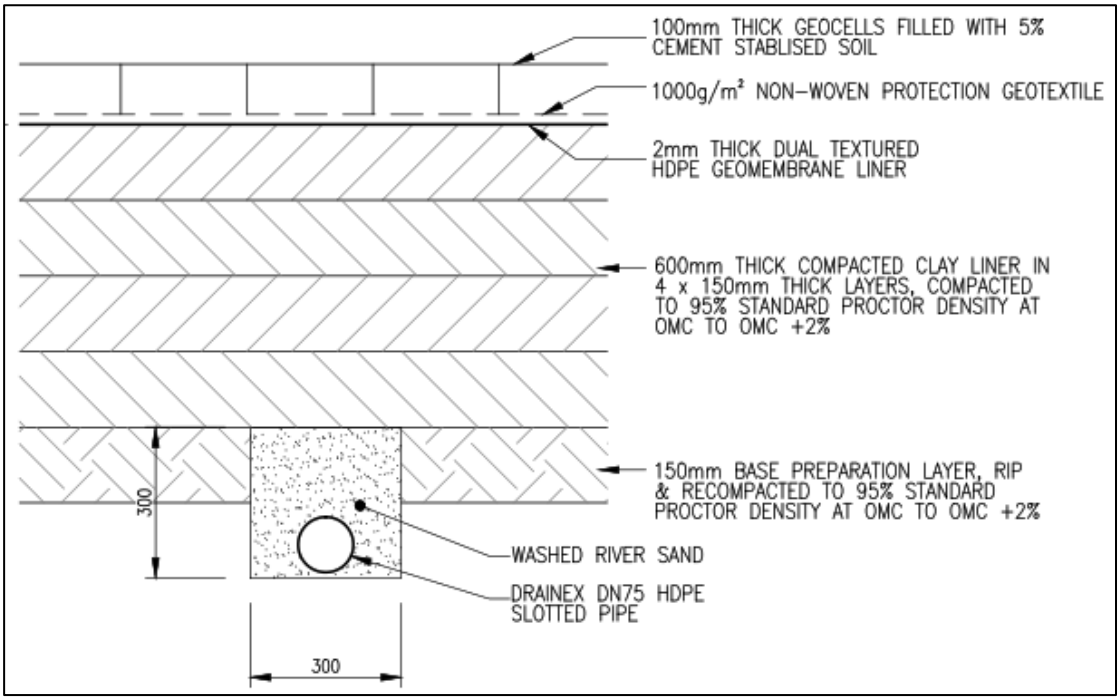


Figure 19: Contaminated water pond liner system

To protect the geomembrane liner from mechanical damage, theft and vandalism, the liner system is covered with a 100 mm layer of geocells filled with 5% cement stabilised soil. This will also protect the geomembrane liner from UV degradation. In the leachate tank bunded area, the geocells are filled with concrete.

The contaminated water pond also has a subsoil drainage system that discharges into a monitoring manhole located on the southern side of the pond.

The pond is fitted with safety ropes. The geocells cover provides animal exit routes in the corners of the pond.

4.11 Surface water drainage

There are two main surface water drainage systems associated with the Area E development as indicated below:

- Surface water / storm water diversion berm and drain;
- Bench Drain.

Surface storm water runoff is to be prevented from entering the landfill cells by means of the perimeter berm around the top of the cells. The perimeter berm will be constructed by compacting selected material to 95% Proctor density. The berm will be 3 m wide on the crest, 1 m high and have 1:3 side slopes to match the side slopes of the landfill. See Figure 20 and drawing P077E-204 for detail. In addition, a grader-cut earth drain 0.5m deep and 1m wide alongside the access and maintenance roads will divert stormwater away from the landfill.

In addition, runoff from the surface of the waste body above ground will be diverted by means of a perimeter drain created within the perimeter berm so that contaminated runoff will drain into the adjacent liner cell to be pumped out to the contaminated water pond (see Figure 21).

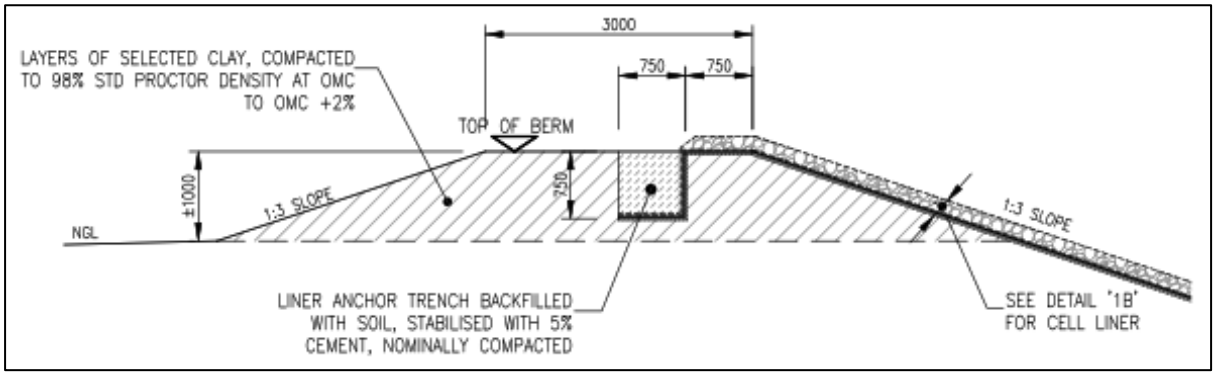


Figure 20: Perimeter berm details

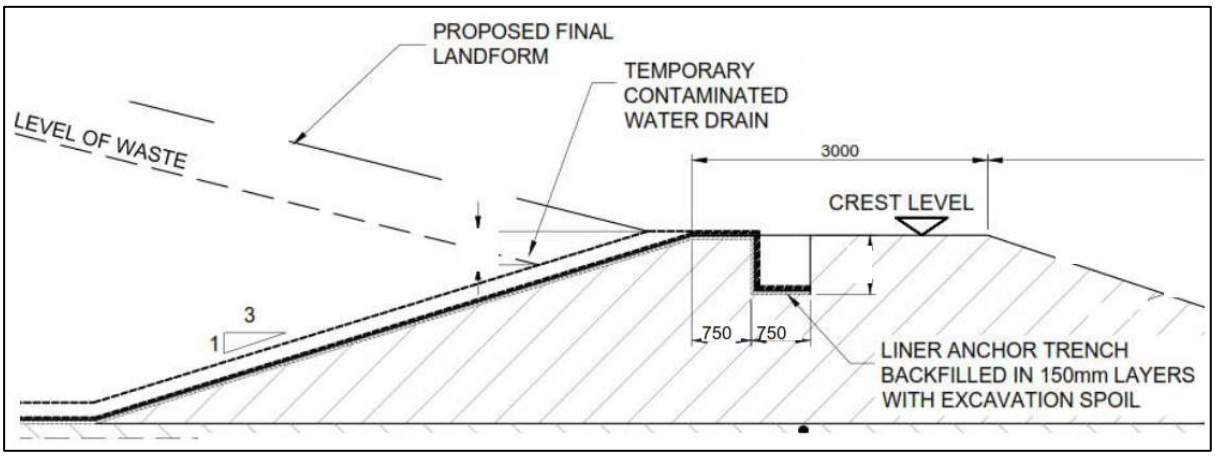


Figure 21: Temporary contaminated water drain along the toe line of the landfill

4.12 Landfill Stability

4.12.1 Introduction

The short- and long-term stability of the Phase 1 landfill has been assessed. The stability analysis of Cell 1 of the Tonkmeter Landfill has been based on the as-built design geometry, the installed barrier system and the proposed final landform shape (see Figure 19 below). The most critical section for slope stability analysis is through Cell 1 in a northwest-southeast direction, which represent the open operational slope of the landfill at 1:2 (V:H). However, the outer slope of the landfill above ground level at 1:3 (V:H) has also been assessed. The stability assessment has considered the landfill at a final height of 25 m above ground level, with a total landfill waste depth of 40 m.

The slope stability analysis has been carried out using assumed waste material properties (density and shear strength) obtained from a study of the literature and interface shear test results from recent similar projects.

4.12.2 Geotechnical conditions

The geotechnical investigation confirmed that the base of the cell is underlain by a soft rock mudrock. The mudrock is overlain by weathered shale and siltstone (clay). The overlying transported material has been removed as part of the quarrying operations.

Groundwater seepage was encountered in the base of the excavated quarry. Although it is removed through the subsoil system, the stability analysis has considered a water level above the barrier.

Triaxial testing was conducted on a remoulded sample of the clay that was used for the CCL in the barrier system.

4.12.3 Geometry

The critical slope geometry for the interim operating slope of 1:2 (V:H) is represented as indicated in Figure 22 below. Figure 19 also indicates the outer slope geometry at 1:3 (V:H).

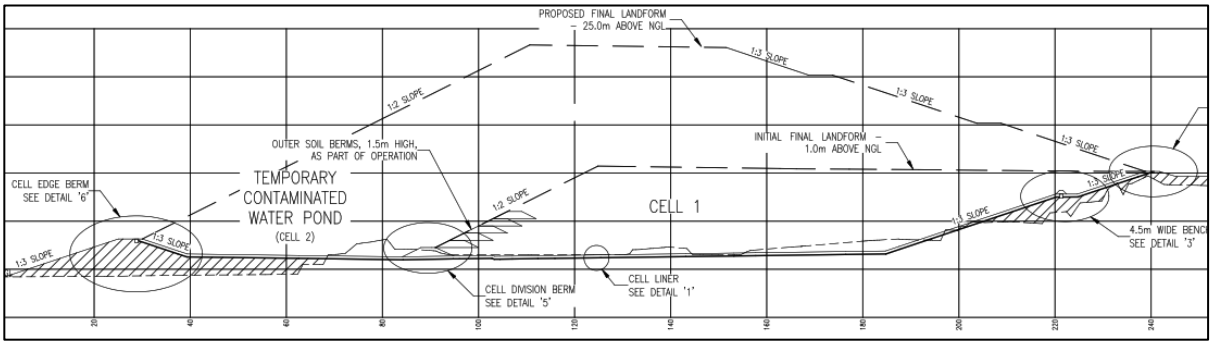


Figure 22: Critical sections for slope stability analyses

4.12.4 Seismicity

According to the Seismic Hazard Map of South Africa (Kijko et al., 2003), the peak ground acceleration is 0,1g for the Tonkmeter Landfill area. The peak ground acceleration may be described as the maximum acceleration of the ground shaking during an earthquake, which has a 10% probability of being exceeded in a 50-year period. An analysis has been performed for a peak ground acceleration of 0,15g as well.

4.12.5 Materials properties

From the geotechnical investigations and testing, the shear strength properties of the residual clay have a friction angle of φ' = 29° and cohesion of c' = 14kPa. These values are used for the material in the modelling as presented in Table 3.

The waste material to be disposed of on the general waste landfill facility will consist mainly of municipal solid waste (MSW), as well as other general Type 2 wastes. Due to the variability in the composition MSW it is very difficult to test its shear strength. Based on the literature, MSW material typically has a cohesion ranging 0 to 100kPa, with Bray et al (2009) recommending a characteristic value of c' = 15kPa for slope stability purposes, 25kPa from Edincliler et al (1996) and 10kPa by Naveen et al (2014). Similarly, for the friction angle, the literature values ranged from 36°, 35° and 25° for the aforementioned three sources respectively. An approximate average of the above values has been adopted as shown in Table 3 below which summarizes the parameters used in the modelling.

Table 3: Summary of Materials Shear Strength Parameters

Material	Cohesion c' (KPa)	Friction angle ø' (degrees)	Unit weight γ (KN/m^3)
Residual clay	14	29	18
Waste body	15	30	11

4.12.6 Liner system

For this stability assessment, a study of available literature has been done, and interface shear results from recent testing as well as from a large data base of test results has been accessed. This information has been used to determine conservative interface strength parameters for the general waste liner system, as shown in Table 4 below. Note that the recent interface shear test results from the same geosynthetic materials used in the Tonkmeter barrier system have also been referenced .

Table 4: Liner Interface Strength Parameters

Interface	Friction Angle (peak) ø (º)	Adhesion (peak)kPa	Friction Angle (residual) ø (º)	Adhesion (residual)kPa
Stone - GTX	40	0		0
GTX – Smooth GM (slopes)	14	0	11	0
GTX – Textured GM (base)	21	0	12	0
Textured GM – GCL	21	14	12	0
GCL (woven) - CCL	29	14	12	0

4.12.7 Phreatic surface

In the absence of any external water sources (surface and subsurface) the phreatic level is generally governed by normal and storm rainfall falling onto the site. Any excess pore water pressure arising from this is normally controlled and drawn down by the leachate drainage system. It has been assumed that this under drainage system will be operating optimally and expected to continue to function adequately over time. A worst-case scenario has also been analysed for a phreatic level at the top of the eastern division berm (2 m above the lowest level of the liner).

4.12.8 Stability analysis and evaluation

The final outer slopes of the landform is 1:3 (V:H). The internal operating slope of the waste body is 1:2(V:H).The most critical slope is the internal operating slope of 1:2 (V:H), that is 38 m above the eastern toe. Geo5 Slope stability software was used to determine the global factor of safety factor for the landfill cell slopes. The analysis was undertaken using the Bishop, Spencer, Janbu and Morgenstern-Price methods, which require satisfying equilibrium of forces and moments acting on individual slices. Circular as well as block failure surfaces along the liner interface were assessed. Figure 23 shows the circular and block failure surfaces for the internal 1:2 (V:H) slope.

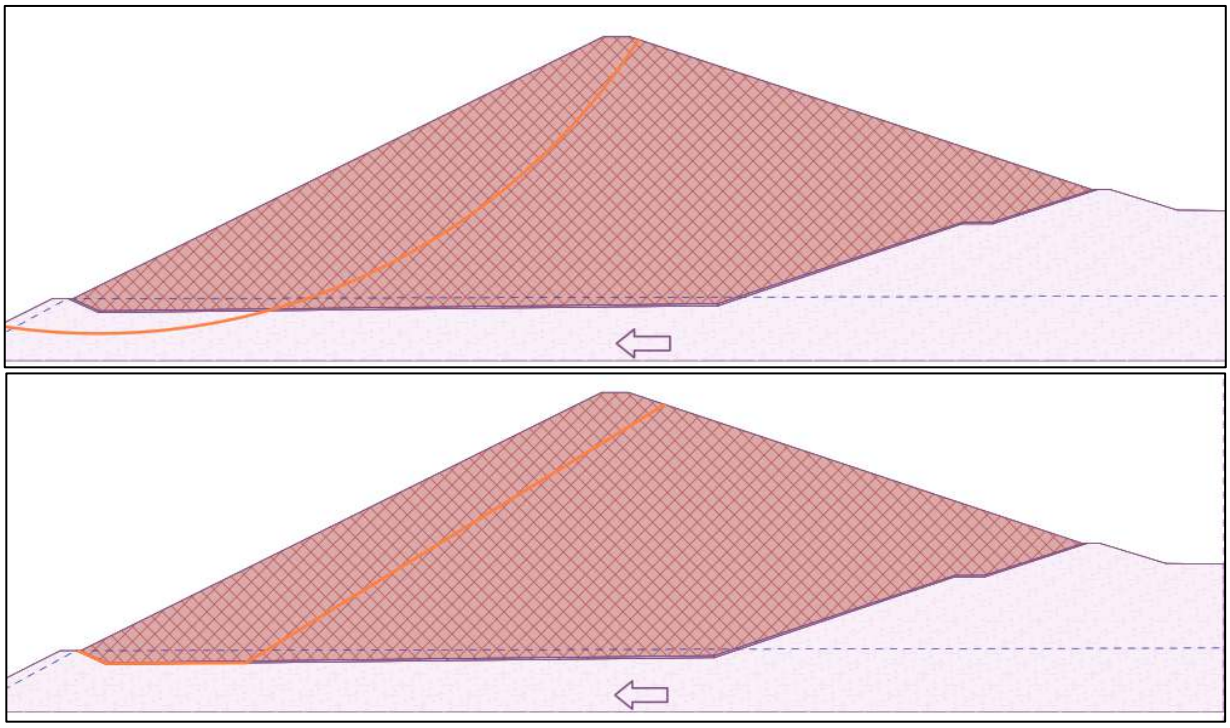


Figure 23: Circular and Block Failure Surfaces on 1:2 operating slope

Figure 24 shows the circular and block failure surfaces for the outer 1:3 (V:H) slope.

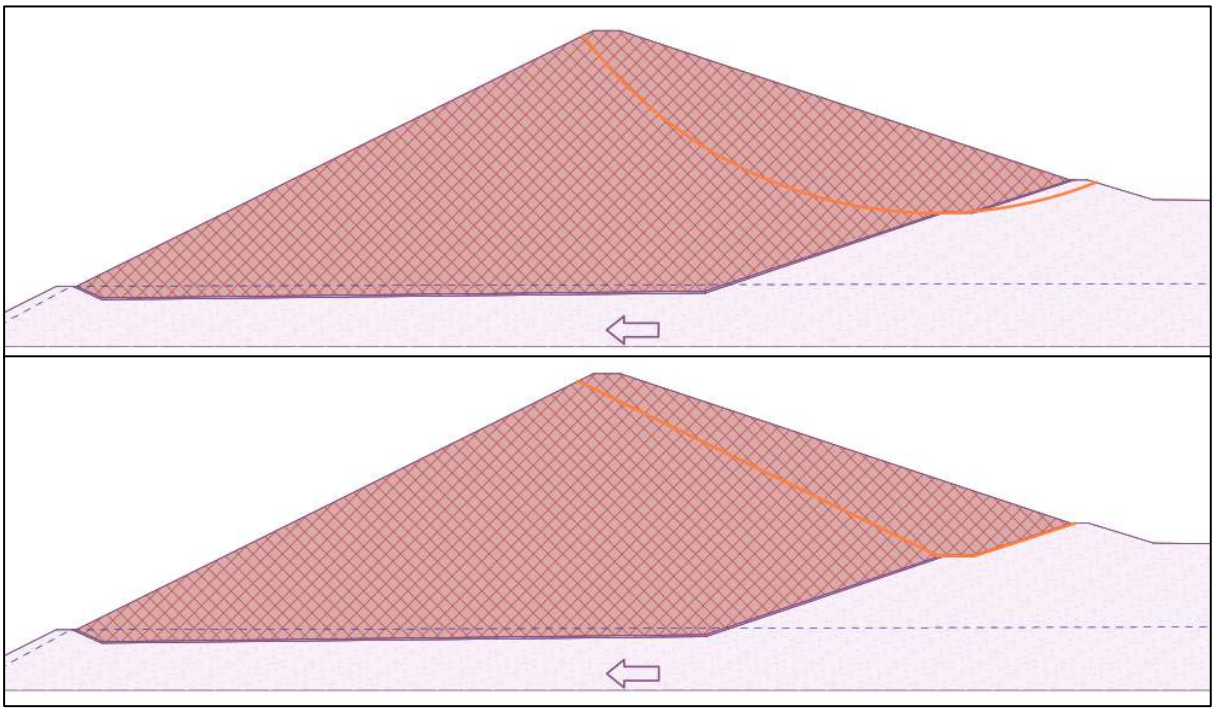


Figure 24: Circular and Block Failure Surfaces on 1:3 outer slope

A Factor of Safety (FoS) ≥ 1,5 is required for the permanent slope while minimum FoS ≥ 1,3 is required for temporary working slopes in a landfill operation. A minimum FoS ≥ 1,1 will be required under pseudo-static (seismic) conditions (Duncan et. al., 2014).

The analysis was run using the smooth geomembrane on the slopes and textured geomembrane on the base. Φ = 11º; c = 0 Kpa (slopes: GTX vs smooth GM - residual strength)
Φ = 21º; c = 0 Kpa (base: textured GM vs GCL - peak)
Note that adhesion has been ignored to be conservative.

Minimum factor of safety values (FoS) were determined for the above conditions. The results of these analyses are summarised in the Table below for the cases considered. The Geo5 Slope stability results are included in Appendix C.

Table 5: Summary of Stability Minimum Factor of Safety values

Condition	FoS Circular	FoS Block	FoS Threshold
Transient (internal operating slope at 1:2) Static, water at level of berm	1,61	1,54	**1,3**
Transient (internal operating slope at 1:2) Seismic (0,1g), water at berm level	1,29	1,14	**1,1**
Permanent (final outer slope at 1:3) Static, water at level of berm	2,57	2,27	**1,5**
Permanent (final outer slope at 1:3) Seismic (0,1g) water at berm level	1,95	1,72	**1,1**

4.12.9 Conclusions

Based on the stability analyses carried with the various assumed shear strength parameters, the stability FoS values of the inner operating face at a slope of 1:2 (V:H) are significantly higher than the minimum threshold values for both transient and seismic conditions for circular and block failures surfaces.

Similarly, the stability FoS values of the outer permanent face at a slope of 1:3 (V:H) are also significantly higher than the minimum threshold values for both permanent and seismic conditions for circular and block failures surfaces.

The stability assessment must be reviewed for each cell development once actual interface shear strength properties have been confirmed through laboratory testing.

4.13 Pumps and Ancillaries

The design philosophy behind the landfill, leachate tanks and contaminated water pond is that any contaminated water and or leachate can be pumped from the cells into either the tanks or the pond. Leachate is then discharged into the sewer line in a controlled manner for treatment. This is being done with the agreement of the Local Authority that is responsible for sewage treatment in the area. Contaminated water can be used for dust control over the landfill or can be discharged to the environment provided that the water quality meets the general effluent discharge criteria. A pump system has therefore been installed on the crest of the contaminated water pond embankment. The pump system has a floating suction located in the contaminated water pond, so that it can pump either into a tanker or the sewer line.

Subsoil seepage from beneath the landfill is pumped to the environment provided it is clean, or to the leachate tank if it contains leachate leakage. The process flow diagram (PFD) is shown in Figure 25 and on Drawing P077E-011.

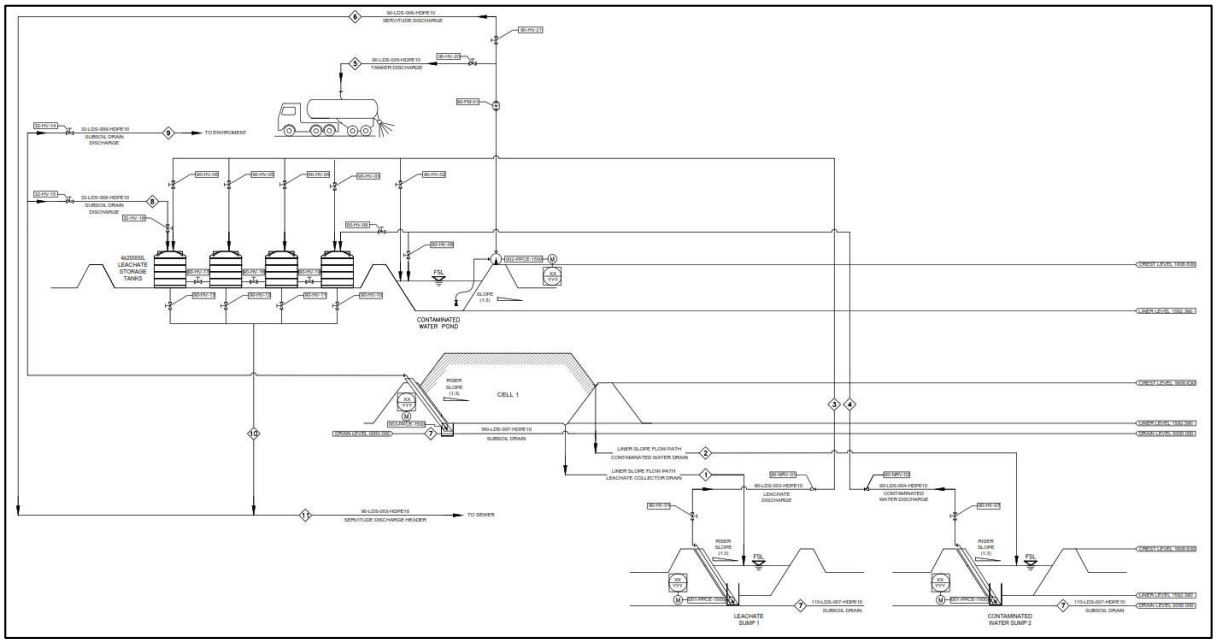


Figure 25: Contaminated Water and Leachate Process Flow Diagram

The design principles for sizing the pump system are as follows:

- The leachate and contaminated water submersible pumps in Cell 1 and Cell 2 should be identical as they would be each be required to pump both leachate and contaminated water.
- The Cell 2 submersible pump should be able to empty the maximum stored water within a reasonably short time after an extreme rainfall event (1 in 50-year 24-hour duration storm) (5 500 m^3); and
- The contaminated water dam pump should be able to empty the dam of 5 500 m^3 within a reasonably short time (a few days).
- The subsoil extraction pump should be a nominal borehole type pump capable of pumping the static head to surface, but at a low flow rate so that the pump does not stop/start too frequently.

Using the aforementioned criteria, it has been determined that for both the leachate and contaminated water extraction pumps, a submersible borehole type pump with capacity of 48 m^3/hr is suitable. The pump will be able to empty the Cell 2 ponded water from the 1 in 50-year 24-hour duration storm in 5 days.

For the contaminated water dam pump, a self-priming centrifugal pump with a capacity of 48 m^3/hr is suitable. It should be mounted on the crest of the dam wall and connected to a goose neck for filling of water tankers. Because of the possibility of some sediment accumulating in the base of the contaminated water pond, it is recommended that the pump should have a floating suction with a foot valve and strainer. The pump will be able to empty the contaminated water dam of 5 500 m^3 in less than 5 days.

The subsoil extraction pump is a submersible borehole pump with a capacity of 1,8 m^3/hr.

The pump specifications and design sheets are included in Appendix C.

4.14 Roads

The site access road from the entrance facility to the the landfill cell will be a gravel road with layerworks as shown in Figure 26.

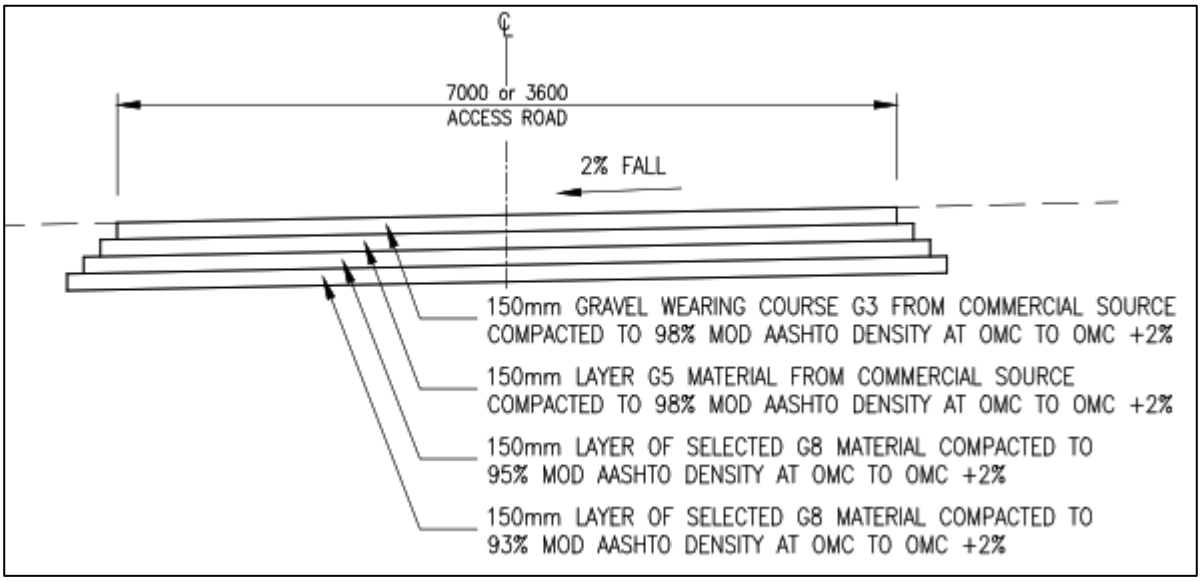


Figure 26: Access road layerworks

4.15 Construction Quality Assurance

The main risk to the performance of a geosynthetic liner system is mechanical/physical damage, during and after installation. For this reason, it is imperative that the liner is supplied and installed by a competent and reputable contractor, and in accordance with a strict quality assurance programme. In particular, extreme care must be taken when placing the leachate collection layer over the installed HDPE geomembrane so as not to damage the liner. Strict supervision is required.

The CQA Plan is a detailed and systematic programme for checking all elements of the design, particularly relating to the liner system. It covers specification and properties of the materials to be used, construction and installation techniques, testing methods and requirements, validation requirements. The CQA Plan would include, inter-alia, the following:

- General information (site details, project description, geology, critical design aspects, etc);
- Definitions used in the CQA Plan;
- Responsibilities of the various parties involved (Owner, Engineer, CQA Monitor, Contractor, Installer);
- Manufacturer’s Quality Control;
- Specifications;
- Testing requirements and frequencies;
- Defects and repairs;
- Reporting requirements; and
- Drawings.

A CQA Plan has been included in Appendix D that will be developed further to suit construction specific requirements for each cell development.

4.16 Landfill Gas Management

On account of the organic content of the general waste it is highly likely that the landfill will produce landfill gas. The biogas will be managed using a combination of horizontal and vertical wells with gas being actively extracted to a processing plant. For the initial layers, a number of horizontal wells will be established during operation and connected to the gas extraction system comprising of a main ring pipe around the cells connected to the processing plant.

Once the top of the landfill is approximately 10 m from the liners a series of pin well nests will be installed. These pin wells consist of steel pipes with holes drilled into them which will be driven into the waste with a collection pipe connected on top. Landfill activities will later on proceed on top off the pin wells.

Each phase will have its own management system and will be designed specifically for that Phase.

When the final capping is applied to the landfill at various stages of completion, appropriate capping structures would be constructed to enable passive venting to ensure that a build-up of gas does not occur in the landfill areas.

Notwithstanding the above, the gas management system at the site will incorporate a gas monitoring system, including the following:

- Monitoring of landfill gas concentrations on a regular basis on the landfill during operation and after closure; and
- Regular monitoring of safe practices to avoid hazardous concentrations of gases at temporary or permanent working areas of the site.

5 CLOSURE AND END-USE

The objectives of the end-use design of the Tonkmeter Area E landfill are as follows:

- To create an aesthetically acceptable landform with gentle slopes (not exceeding 1:3) that, where possible, blends in with the surrounding terrain.
- To maximise the landfill airspace available for waste disposal and hence the site life.

5.1 Final landform and end-use

At this stage, the proposed final shape of the Tonkmeter Area E landfill would be determined according to the surrounding terrain, and to maximise the airspace from the available footprint. It would also be designed to meet drainage and end-use requirements. It is recommended that the end-use of the landfill be considered as restricted open space, on account of the waste disposed on it. Other forms of development could also be considered. The end-use of the site should, however, be discussed with all stakeholders to ensure that the rehabilitated site is acceptable to them.

Based on the surrounding topography and land use, the maximum height of the landfills would be about 25 m above the original natural ground level. The upper surfaces of the landfill must have general slopes of at least 1:50 to promote rapid drainage from the landfill surface.

The proposed final landform for Tonkmeter Area E landfill is shown on Drawing No P077E-304, and in Figure 27 below.

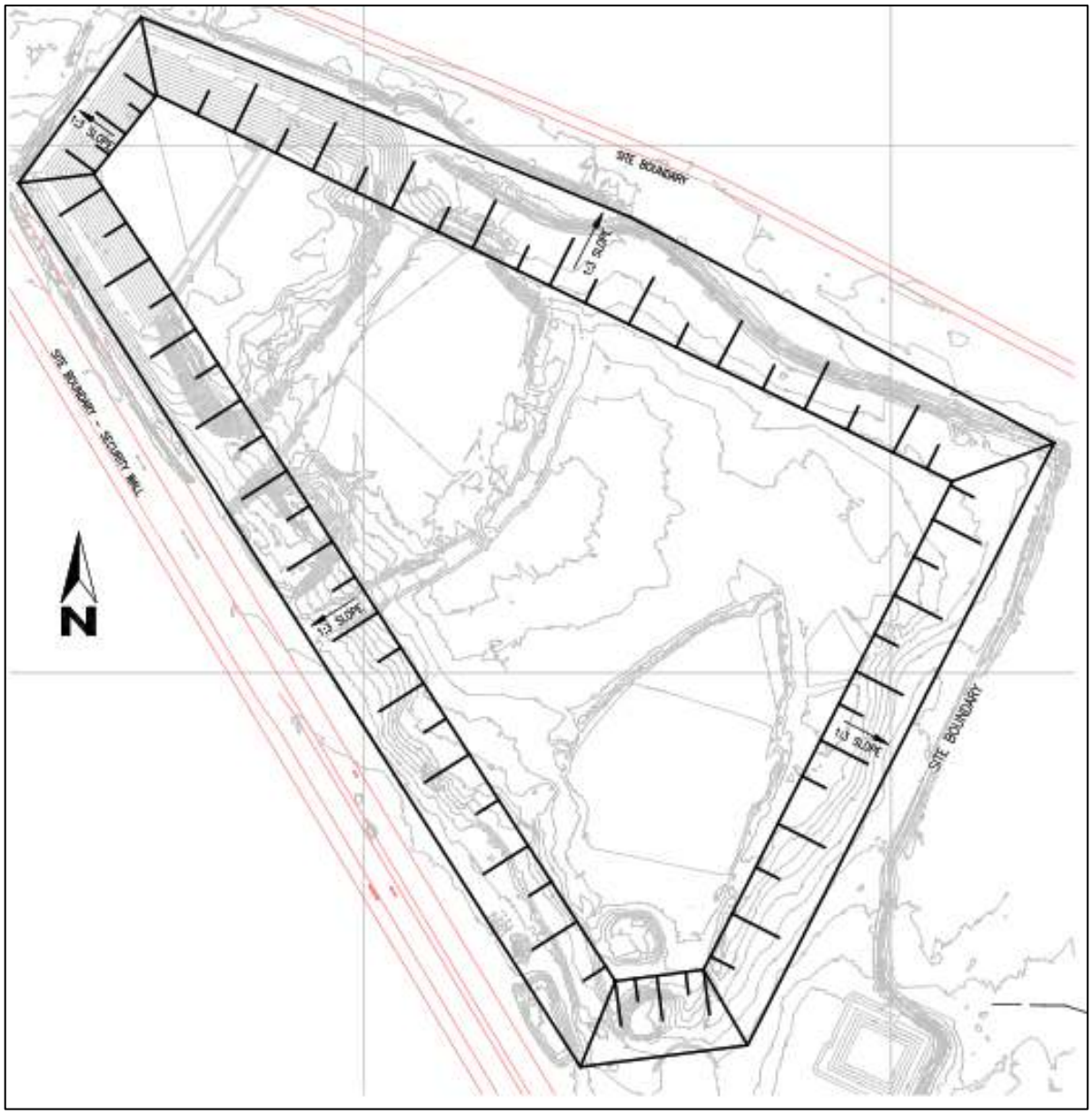


Figure 27: Tonkmeter Area E proposed final landform plan

5.2 Closure and rehabilitation

As the different sections of the landfill are completed to final height, they are to be appropriately shaped, graded and capped as indicated on Drawing P077E-304.

The general objective of the capping design is to provide a cost-effective, environmentally acceptable means of capping and rehabilitating the landfill, so that it will not present a threat to the environment and adjacent land users in the long term. In particular the capping system needs to:

- Provide long-term minimization of migration of liquids through the capped portion of the facility.
- Enable the venting of any gas from the landfill that may build up beneath the capping system towards the gas extraction vents.

- Function with minimum maintenance.
- Promote maximum runoff of precipitation and minimize erosion or abrasion of the cover soil.
- Accommodate settlement and subsidence so that the cover's integrity is maintained.
- Ensure the stability of the soil cover.

The capping design should generally based on the requirements of both the South African “Minimum Requirements for Waste Disposal by Landfill”, and the “Technical Advisory Practice Note: Capping Closure of Waste Management Facilities and Pollution Point Sources (DEA, 2016)”, as well as current best practice.

Based on the requirement that a non-infiltration cap would be required on the flatter plateau areas designed for eliminating infiltration (< 15 ℓ/ha/d) and side slopes designed for optimised stormwater runoff, the preliminary final capping systems would be as shown on Drawing No P077E-304, and in Figure 28 and Figure 29 below.

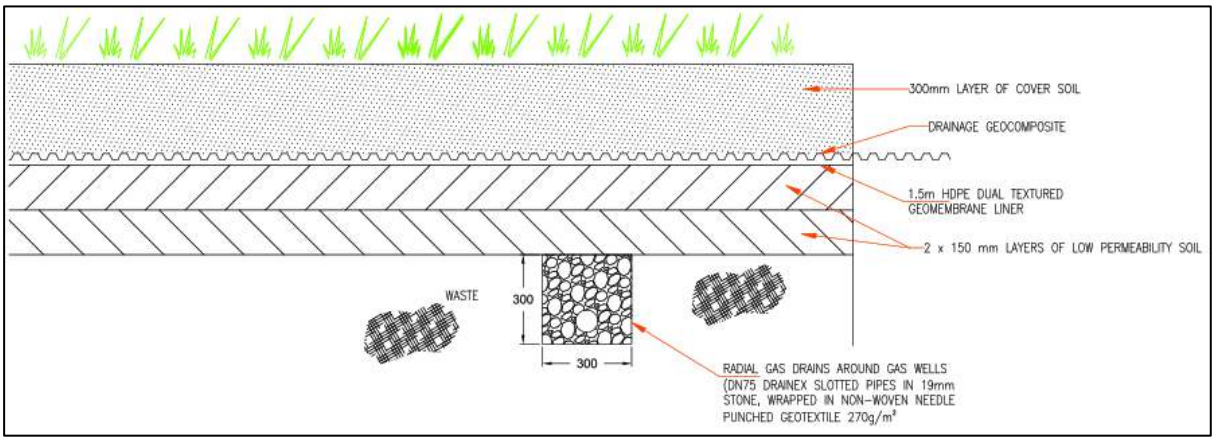


Figure 28: Final capping for top plateau

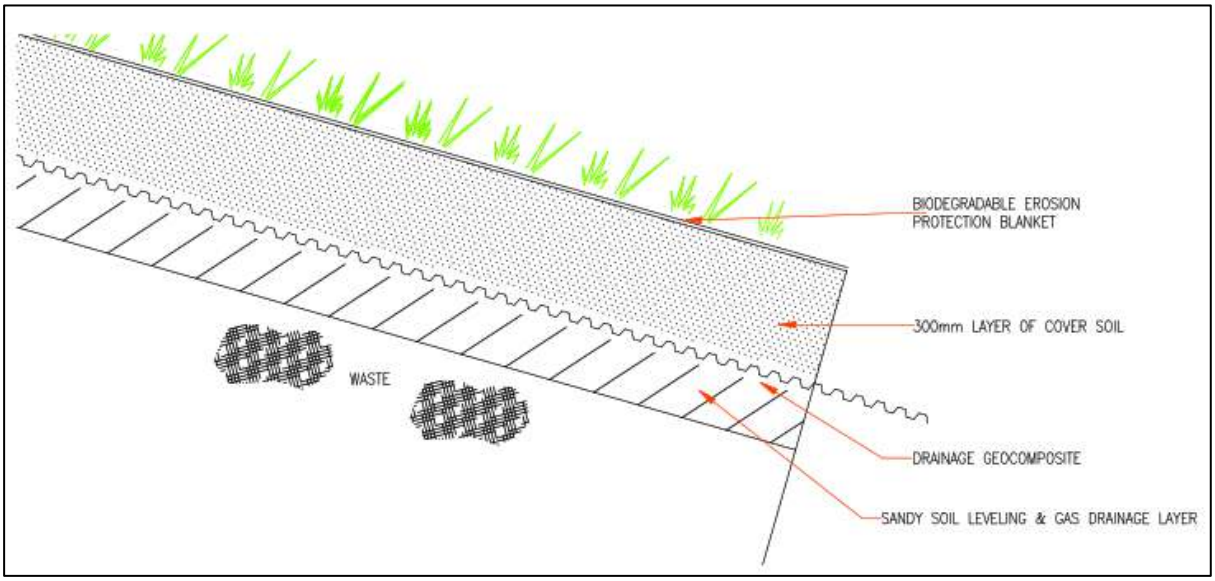


Figure 29: Final capping for side slopes

Vegetation of completed areas should commence as soon as possible after capping. Indigenous shrubs could be planted around the site for screening purposes. Over the rest of the site, grass is to be established using indigenous grass types.

Provided the vegetation is always maintained during operation, there should be no need for later rehabilitation. After closure, ongoing maintenance of the landfill capping and vegetation will be required. This could include rehabilitation of erosion and filling in of vegetation that died.

It is noted that prior to closure and capping of any portion of the Tonkmeter Area E Landfill, a formal detailed closure deign will be submitted to the regulatory authorities for approval.

6 CONCLUSIONS AND RECOMMENDATIONS

In formulating the detail design for the Tonkmeter Landfill Area E, every effort has been made to meet the objectives of landfill design, i.e. to provide a cost effective, environmentally sound and socially acceptable facility. In addition, the barrier requirements of the National Norms and Standards for Waste Disposal by Landfill (GN R636) have been followed as best as possible to provide adequate environmental protection for the disposal of all Type 2 wastes. The stability of the landfill when developed to a height of 25 m above ground level has been assessed to meet the minimum factors of safety against slope failure.

It is recommended that this landfill design report be submitted to the regulatory authorities for approval to enable landfilling of all Type 2 wastes and up to a height of 25 m above natural ground level.

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APPENDIX A

COUNCIL FOR GEOSCIENCE LETTERS OF APPROVAL

Council for Geoscience

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Your Reference: 2011/02
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Fax: (086) 676 2091
No. of Pages: 3

12 August 2011

Ekurhuleni Metropolitan Municipality
P O Box 13
Kempton Park
1620

ATTENTION: Mr. Pieter Grobler

By Fax: *0866228264*

Dear Sir

WASTE DISPOSAL SITE: REMAINDER OF PORTION 81, RIETFONTEIN 128-IR

The firm, Dr J P Venter Consulting Services (JPV) submitted their report: "Dolomite stability investigation of a part of the Portion 81 of the farm Rietfontein 128-IR for the development of waste disposal facilities, dated July 2011, on behalf of the client, to this office for comment on 29 July 2011. This office acts as Agent to your Municipality in auditing the geoscientific work and emanating recommendations. The report is in support of the Waste Disposal Facility proposed for the site.

The site is located south of the N17, west of Tonk Meter Road and east of Kwa-Thema in the southern part of Springs. The area investigated covers approximately 108ha. The site is highly disturbed with old mine slimes dumps that have been removed and a large rock dump is also present on the southern part of the site.

The available geology map shows that the site is underlain by Vryheid and Dwyka Formations of the Karoo Supergroup, which are also underlain by dolomite of the Chuniespoort Group, Transvaal Supergroup. Some boreholes in this area were drilled into the dolomite and have water levels in the excess of 55m below surface. A total of 34 percussion boreholes (from previous and recent investigation) have been drilled as part the stability assessment of the site.

Having reviewed the report;

a) JPV has classified the site as **IRC 1.** Most boreholes intersect residual Karoo material, mudrocks and shale of the Vryheid Formation, diamectite of the Dwyka Group with dolomite or dolerite at depth. Some boreholes, however, did not intersect any residual dolomitic material or dolomite rock.

This office is in broad agreement with the risk classification.

b) JPV indicated in Section 2 of the report that this investigation was done to determine the dolomite stability of the site, make recommendation on whether the site is suitable for use as waste disposal site and make recommendations on precautionary measures.

c) JPV has assigned a Dolomite Area Designation of **D2.** This is supported.

Therefore, this office confirms support of the proposed Waste Disposal Facility on the remainder of Portion 81 of the farm Rietfontein 128-IR, conditional to the following;

a) A Dolomite Risk Management Plan must be compiled for the site since it is underlain by dolomite.

b) In terms of draft **SANS 1936-1:2010**, waste disposal sites are classified as **C5** and are allowed to be developed on an **IRC 1** site.

c) Due to the homogeneous geological conditions encountered across the site, this Office does not regard the FPI as necessary.

d) The professional team involved shall carefully consider the appropriate water precautionary measures and then ensure and finally certify that these have been implemented. The design of a waste disposal site should ensure the following:

- ✓ No polluted water or liquids leak into the underlying soils or rocks.
- ✓ All stormwater be diverted away from the site to natural water courses.
- ✓ All water precipitated from undeveloped parts of the site be diverted away from the waste body and released into the natural water courses.
- ✓ The recommended standard lining system should be installed during the development of the site.
- ✓ If any leakage is detected steps must be taken to rectify the situation.

e) A Dolomite Risk Management Plan must be implemented for the site by the relevant Municipality. The responsible municipality must be made aware of the risks involved in building on dolomite, and be informed about how to be vigilant and act pro-actively by applying sound water management principles.

This letter reflects the Council for Geoscience's view and approach to development on dolomite at this time, as reflected by the above date. These comments may not be viewed as open-ended. If a property changes ownership or land-use changes are made, the comment may in part or wholly no longer apply. This Office should be informed of such changes and the Competent Person responsible for the dolomite stability investigation should be given the opportunity to indicate the influence such changes could have on the overall stability.

If you have any further queries, please do not hesitate to contact this office.

Yours faithfully,

G J HEATH
ENGINEERING GEOSCIENCE UNIT

CC: **Dr J P Venter Consulting Services**
P O Box 7271
Centurion
0046

By Fax: *012 663 2662*

Attention: **Dr. J P Venter**

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Council for Geoscience

Our Reference: F3849.2
Parts of Portion 81, Rietfontein 128 IR
Your Reference: 2013/03
Enquiries: T Oosthuizen
Tel: 012 841 1160
Fax: 086 615 6682
No. of Pages: 5

23 May 2013

Ekurhuleni Metropolitan Municipality
P O Box 13
Kempton Park
1620

ATTENTION: Mr. F Meyer

By email: *Francois.Meyer@ekurhuleni.gov.za*

Dear Sir,

WASTE DISPOSAL FACILITIES: PARTS OF PORTION 81, RIETFONTEIN 128-IR

The firm, Dr J P Venter Consulting Services (JPV) submitted their report: "Dolomite stability investigation of parts of Portion 81 of the farm Rietfontein 128-IR for the development of waste disposal facilities (Addendum report)", dated May 2013, on behalf of their client, Vereeniging Refractories (Pty) Ltd, to this office for comment on 13 May 2013. This office acts as an agent to state authorities in reviewing dolomite stability investigations on their behalf. The report and discussions with this office on 10 May 2013 and 17 May 2013 forms the subject of our audit.

Background:

1) The entire site covers a surface area of approximately 108 hectares. The site has been subdivided into Zones A to G, as shown on Figure 1 in JPV report dated May 2013.

2) A report for waste disposal on Areas D, E and F was prepared by JPV (Report No. 2011/02) in July 2011 and submitted to this office for comment. A favourable letter of comment, dated 12 August 2011 (ref. F3849) was issued from this office on this report.

3) JPV indicates that two small areas adjoining Area E were not included in the initial report i.e.:
   - A servitude area situated along the north-eastern boundary of Area E (the original gravity survey covered this area), and
   - A small area in the southern part of Area E.

   No boreholes were drilled on either of these areas and it was decided to drill an additional hole on the northern area and to assess the small area in the south with a gravity survey and investigative drilling.

4) Furthermore, an investigation for the types of development was also carried out on Area G.

5) JPV indicates that whereas no poor conditions were encountered during the 2011 study, some localized poorish conditions were encountered at depth during this investigation of Area G and it was decided to carry out some more detailed investigations on Area G. Further confirmatory drilling was also conducted in the western part of Zone D to determine whether these deep localised poorish conditions extend to Area D and to assess whether it could have an effect on waste disposal in the area. To increase the confidence in this assessment, additional holes were also drilled on the eastern part of Area D and on Area E.

JPV's Investigation procedures:

6) One additional borehole was drilled on the north-eastern servitude area of Zone E.

7) An additional gravity survey was carried out on the southern part of Area E followed by the drilling of an additional borehole.

8) The poorish conditions encountered in borehole 33/33 in Zone G were further investigated by the drilling of 4 additional boreholes 10 m away at four opposite extents from this borehole and the investigation of the conditions in the 5 holes by using a borehole video camera. Additional drilling was also conducted in other areas within this zone.

9) 3 additional boreholes were drilled in the western side of Area D to obtain a more dense drilling coverage in this area. 2 additional boreholes were drilled on the eastern side of Area D and 1 on Area E to gain a higher level of confidence that no adverse conditions are present to the east.

The following results and conclusions are noted from the report:

10) The additional investigations in Area E (servitude and southern areas) revealed the same hazard evaluations as for the remainder of Area E, i.e. Inherent hazard Class (IHC) 1. JPV assigned a Dolomite Area Designation of D2 to this area.

11) The following is indicated regarding the poorish zones encountered in Area G:
   - 15 boreholes were drilled on the 64 hectare area and poorish conditions were encountered at depth in two boreholes to the south and in one hole to the west of Area D.
   - A small cavity (38,2 m to 41,1 m)was encountered in borehole 15/13, whereas poorish conditions were encountered in borehole 19/17 from 48,5 m to 50,2 m below the Dwyka Group. In borehole 33/33 a cavernous zone was encountered from 37,9 m to 41 m, before the hole was stopped due to difficult drilling conditions.
   - In order to investigate these poorish zones, a video camera of about 60 mm diameter on a stiff cable of about 52 m length was sourced from Sight Lines. This camera could tilt and rotate to view the whole circumference of the hole.
   - Four boreholes were drilled to the west of Area D to obtain a more dense coverage. No conditions similar to those in borehole 33/33 were found in the additional boreholes drilled in Area G.

12) The additional drilling (3 boreholes) on the western part of D indicated no of soft zones or cavities in this area.

13) A depth to dolomite plan was drawn for the larger area using all the available boreholes. The contour plan and some sections drawn shows that the dolomite is deeper in the west (and the southern part of Area G) and it was thought that the "soft zones" in the chert may be related to depositional conditions in the deeper valleys scoured during glaciation. JPV therefore drilled some additional boreholes on the shallower dolomite on Areas D en E.
   - Three additional boreholes were drilled on Area E and no poor conditions were encountered. Air loss was encountered in BH40/12A to the south but was due to the loose backfill of bricks, ash and mudrock.

14) In conclusion, JPV indicated that the investigations show that the classification of Areas D, E and F remains the same as in the JPV (2011) report, i.e. IHC 1 with a D2 designation.

15) JPV emphasized that the precautionary measures, as stipulated in the 2011 are implemented. A dolomite risk management plan should be compiled and submitted to the Ekurhuleni Metropolitan Municipality.

<u>The Council for Geoscience would like to comment as follows:</u>

a) Having reviewed the report, we are of the opinion that the site has been assessed in detail and JPV has made every attempt to ensure that possible poor conditions are identified as well as properly investigated. The motivation / explanation in the JPV report for the 'poor conditions' encountered at depth in Area G is supported.

b) JPV has assigned an IHC 1 to the entire area. We are broadly in agreement with the hazard assessment of the site, although we are of the opinion that the areas where dolomite bedrock was encountered at a depth shallower than 20 m from surface, the classification should rather

be IHC 2. These areas can be determined from the depth to dolomite bedrock contour map (Drawing 2013/03/02) in the JPV 2013 report.

We realize that this was not specified in the letter issued on the 2011 report (12 August 2011), as the CGS has since adopted an approach to classify areas where dolomite is covered by say less than 20 m of non-dolomitic materials, as IHC 2.

c) During a discussion with Dr. K Venter from JPV on 17 May 2013, the proposed land use in terms of the SANS 1936-1:2012 document was clarified. According to the SANS 1936-1:2012 document, the proposed waste disposal facility now classifies as an IN 5 type development (C5 in the draft SANS 1936-1:2010). The SANS 1936-1:2012 requires a dolomite area designation of D3 for IHC 1 to 6 land.

JPV assigned a dolomite area designation of D2 to the entire site. This classification was supported in 2011, but has subsequently changed due to the publication of the SANS 1936-1 document in late 2012. Therefore, we would like to indicate that a dolomite area designation of D3 should be implemented for the entire site.

d) This office confirms that the level of investigation for the proposed waste disposal facility is considered adequate. For any other development types (like in Area G), additional drilling might be required as per the SANS 1936 document's requirements.

Therefore, this office confirms support of the proposed Waste Disposal Facility on parts of Portion 81 of the farm Rietfontein 128-IR, conditional to the points above and the following:

e) A Dolomite Risk Management Plan must be compiled and implemented for the site since it is underlain by dolomite.

f) The professional team involved. JPV shall carefully consider the appropriate water precautionary measures and then ensure and finally certify that these have been implemented. The design of a waste disposal site should ensure the following:
   - ✓ No polluted water or liquids leak into the underlying soils or rocks.
   - ✓ All stormwater be diverted away from the site to natural water courses.
   - ✓ All water precipitated from undeveloped parts of the site be diverted away from the waste body and released into the natural water courses.
   - ✓ The recommended standard lining system should be installed during the development of the site.
   - ✓ If any leakage is detected steps must be taken to rectify the situation.

g) The precautionary measures as stipulated in the SANS 1936-3:2012 document for waste disposal facilities should be implemented.

This letter reflects the Council for Geoscience's view and approach to development on dolomite at this time, as reflected by the above date. These comments may not be viewed as open-ended. If a property changes ownership or land-use changes are made, the comment may in part or wholly no longer apply. This Office should be informed of such changes and the Competent Person responsible for the dolomite stability investigation should be given the opportunity to indicate the influence such changes could have on the overall stability.

If you have any further queries, please do not hesitate to contact this office.

Yours faithfully,

A C OOSTHUIZEN
Engineering Geologist
for Dr S FOYA
Engineering Geoscience Unit Manager

CC: **Dr J P Venter Consulting Services**
P O Box 7271
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By Fax: *012 663 2662*

Attention: **Dr. J P Venter**

APPENDIX B

INTRASOLUTIONS - TONKMETER WASTE DISPOSAL FACILITY: AREA E: DOLOMITE STABILITY REVIEW

Consulting Engineering Geologists

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ATTENTION: MS SAN OOSTHUIZEN

Your reference	Our reference	Date
	IR2222	**10 JULY 2026**

TONKMETER WASTE DISPOSAL FACILITY: AREA E: DOLOMITE STABILITY REVIEW

SUMMARY

This document presents a review of dolomite stability conditions at the Tonkmeter Waste Disposal Facility, Area E in the context of present Facility Development proposals which entails increasing the height of landfill cells from 1 meter above ground to 25 meters above ground and associated infrastructure. The Facility as a whole is a Class GLB+ landfill designed to provide a reliable alternative to constrained municipal facilities in the area. Area E is 47,3 hectares in extent.

The Council for Geoscience supported the development of the larger site as a Waste Treatment Facility in comments dated 23 May 2013, referenced F3849-2 Rem Porton 81, Rietfontein 128-IR. This support is understood to remain in place provided certain conditions are met.

Area E is located on Karoo age deposits, which blanket dolomite rock at intermediate to great depth. In the past clays were quarried from this site.

The shallow groundwater levels reported in the 2011 and 2013 boreholes are confirmed by the 2013 geohydrological study to represent a perched aquifer. The groundwater level of the deeper aquifer is 20-40m deep at an elevation of 1570-1590m AMSL.

Based on available data and current best practice, Area E represents a **low to medium** susceptibility of sinkhole and subsidence formation with respect to ingress of water and a **low** susceptibility of sinkhole and subsidence formation with respect to anthropogenic groundwater level drawdown, i.e. Inherent Hazar Class 1/2//1. The Design Hazard Class is 2. In light of the land use, the Dolomite Area Designation is D3.

TONKMETER WASTE DISPOSAL FACILITY: AREA E: DOLOMITE STABILITY REVIEW

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BOREHOLE PROFILES APPENDIX 1

1. INTRODUCTION

This document presents a review of dolomite stability conditions at the Tonkmeter Waste Disposal Facility, Area E in the context of present Facility Development proposals to increase the height of landfill cells from 1 meter above ground to 25 meters above ground.

2. TERMS OF REFERENCE AND SCOPE OF WORK

IntraSolutions is appointed by EcoPartners. The terms of reference and scope of the work were set out in Proposal Number IR2222P and IntraSolutions was instructed to proceed with the report accordingly.

3. AVAILABLE INFORMATION

Information sources include:

3.1. Geological Information

- **Geological Map, 1: 250 000 Scale Series**: published by the Geological Survey of South Africa (Council for Geoscience):
- **The Geology of South Africa**: by MR Johnson, CR Anhaeusser, RJ Thomas, 2006.

3.2. Industry Standards and Publications

- South African National Standard **SANS 1936-1/2/3/4: 2012**, Edition 1 (ISBN 978-0-626-27840-3).
- Environmental Earth Sciences, Springer-Verlag: "**A Performance Based Approach to Dolomite Risk Management**" by D Buttrick, N Trollip, R Watermeyer, N Pieterse, A Gerber, Volume 64, Issue 4, p1127 to p1138, dated 2011.
- **Department of Public Work's Consultants Manual** entitled: "Department of Public Works: Appropriate development of infrastructure on dolomite: Manual for consultants: PW344", dated September 2010.
- Journal of the South African Institution of Civil Engineering: "**Proposed Method for Dolomite Land Hazard and Risk Assessment in South Africa**" by Buttrick, Van Schalkwyk, Kleywegt and Watermeyer, Volume 43, Number 2, dated 2001.

3.3. Geohydrological Data and Reports

Groundwater information is made available by the Department of Water Affairs through the National Groundwater Information System (NGIS) which offers read-only access to data. In addition, the report "Geohydrology Guideline Development: Implementation of Dolomite Guideline – Phase 1, Activities 19 & 28: Desktop development of a Dolomite Hydrogeological Compartment Map and explanation booklet (Report), dated November 2009, Project Number: 14/14/5/2, Authors: Martin Holland: Water Geosciences Consulting, Frans Wiegmans: Golder Associates", was consulted.

3.4. Existing Reports

- Dr J P Venter Consulting Services, Report Nr. 2011/02, dated July 2011, titled "Dolomite Stability Investigation of Part of Portion 81 of the Farm Rietfontein 128-IR for the Development of Waste Disposal Facilities".

- Dr J P Venter Consulting Services and J S Stiff (JD Geotechnical Services CC) Report Nr. 2013/03, dated May 2013, titled: “Dolomite Stability Investigation of Parts of Portion 81 of the Farm Rietfontein 128-IR for the Development of Waste Disposal Facilities Addendum Report”.

4. DESCRIPTION OF THE STUDY AREA

4.1. Locality

The Facility is located on the corner of Tonk Meter Road and Coaton Avenue in Springs, Gauteng, refer to Figure 1.

4.2. Site Description

The Facility is currently a Class GLB+ landfill designed to divert waste and provide a reliable alternative to constrained municipal facilities in the area. Area E is 47,3 hectares in extent. Refer to Figure 2.

The 2011 report indicates that old mine slimes dumps were removed from the southern part of the mined areas on Area E. Area E was mined by Corobrik for the manufacturing of bricks. The plastic and fire clays were mined, stockpiled and used for the manufacturing of refractory bricks by Verref.

The northern side of Area E is relatively undisturbed apart from some shallow excavations, probably for sand or construction materials. The natural slopes on the larger original farm portion have been disturbed. The natural drainage used to be at a shallow slope in a northeasterly direction towards the Klein Blesbok Spruit which drains towards the east north of the N17 freeway. Orthophoto maps show northeasterly slopes of about 1 percent across areas D and E.

5. PROCEDURES USED IN THIS STUDY

The procedure in the investigation and assessment of geotechnical conditions on the site follow:

5.1. Assimilation of Available Data

This investigation is initiated by gathering, assimilating and reviewing existing, available geological, geotechnical and geohydrological data pertaining to the site.

As dolomite hazard assessment is an empirical science that is constantly changing, these succinct background notes are provided to contextualise the procedures used in these investigations, in time and content. The objective is to provide an understanding of the process currently followed in classifying the site and the determination of appropriate risk management requirements, should it be required at some future date.

5.2. Gravity Data

A gravity survey of a dolomitic terrain can be used to help determine dolomite bedrock configuration (bedrock topography) and allows various deductions to be made about tectonic processes.

Use is made of existing gravity data, as compiled in the 2013 report (and referenced in Section 3.4.).

5.3. Rotary Percussion Boreholes

Existing boreholes were extracted from the reports provided (13). The borehole logs are presented in Appendix 1.

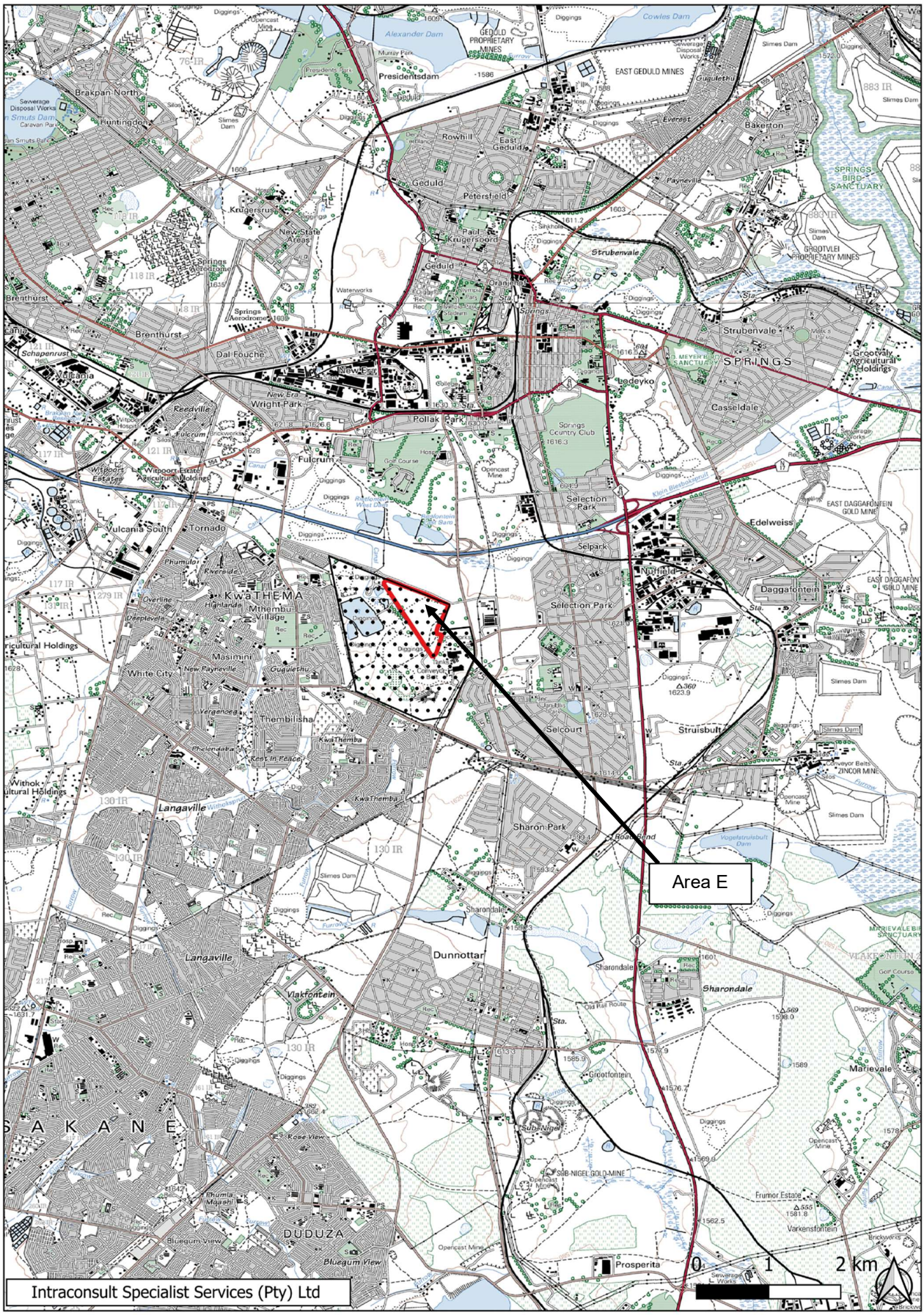
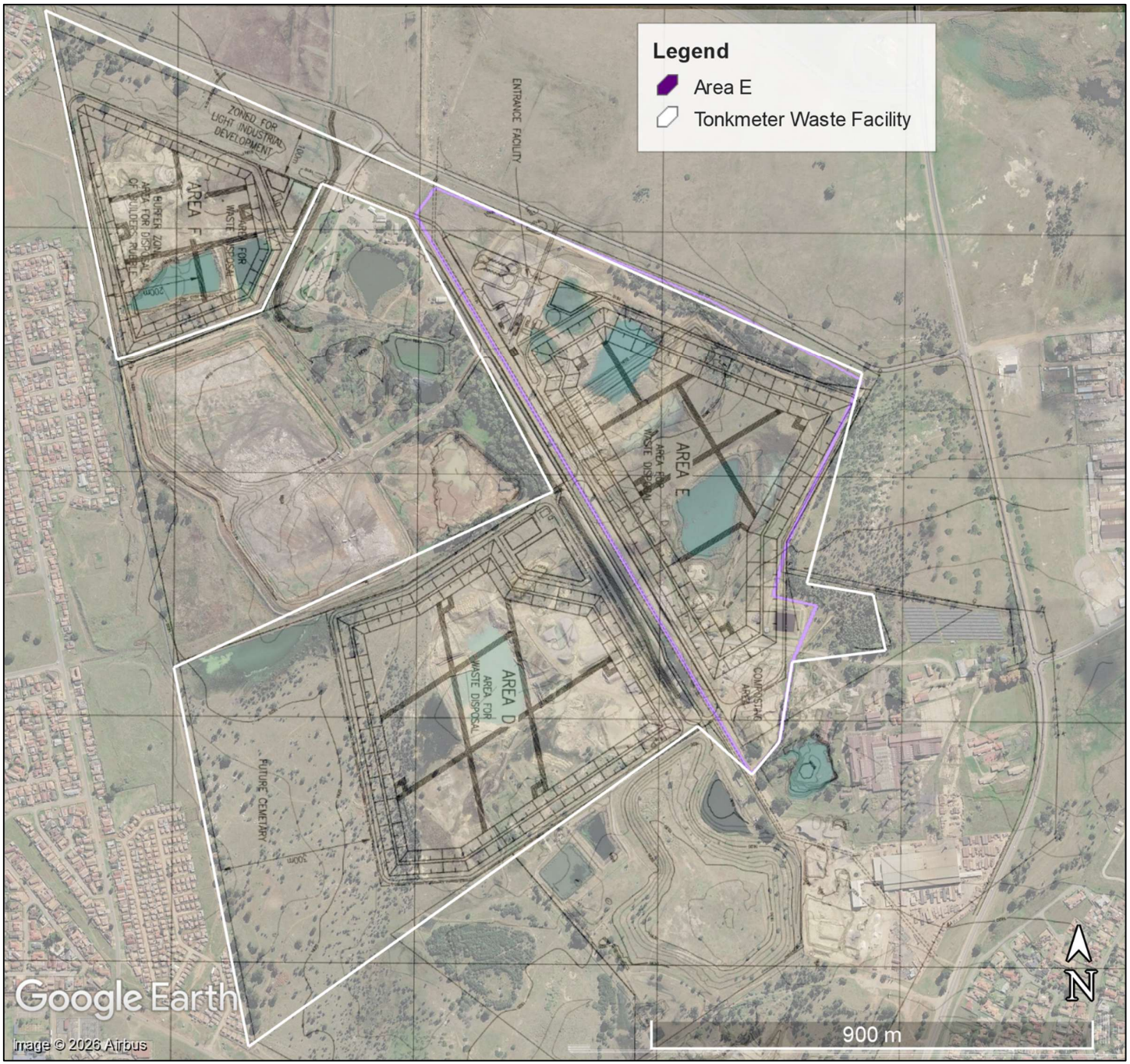


Figure 1: Locality (Regional)

Figure 2: Facility Layout



5.4. Hazard Characterisation Procedure

National Standards (SANS 1936, 2012) provide defined terminology (with previous term used) are given below:

5.4.1. Terminology

(i) Hazard

Source of potential harm. Hazard is the function of magnitude (of the events), area, and frequency.

(ii) Inherent Susceptibility (Inherent Risk)

A reflection of the geological susceptibility of a karst area to an event (sinkhole or subsidence). Inherent Susceptibility is expressed in terms of three broad categories, namely low, medium and high, typically, but not exclusively, denoting anticipated number of events per area over time.

(iii) Inherent Hazard Class (Inherent Risk Class)

A site is characterised in terms of **eight standard inherent hazard classes,** denoting the likelihood of an event (sinkhole or subsidence) occurring (susceptibility) as well as its likely size (diameter). The larger the Inherent Hazard Class (IHC) number, the greater the likelihood of the event occurring and the larger its potential size should it occur, for each respective inherent susceptibility category (low, medium, high).

(iv) Subsidence (Doline)

Shallow, enclosed depression. In past South African literature subsidence, as defined above, is synonymous with doline. The term subsidence is substituted to prevent contradiction with international literature.

(v) Sinkhole

A feature that occurs suddenly and manifests itself as a hole in the ground. In international literature the term sinkhole is often synonymous with doline.

(vi) Dolomite Land

Land underlain by dolomite or limestone residuum or bedrock (or both), within the Malmani Subgroup and Campbell Rand Subgroup, typically at depths of no more than:

a) 60m in areas where no de-watering has taken place and the local authority has jurisdiction, is monitoring and has control over the groundwater levels in the areas under consideration; or
b) 100m in areas where de-watering has taken place or where the local authority has no jurisdiction or control over groundwater levels.

5.4.2. Assessment Procedure

The hazard characterisation procedure (previously referred to as "risk" characterisation procedure in South African literature) is in accordance with the peer reviewed paper "A performance based approach to dolomite risk management":

The available information, geophysical data, borehole data and geohydrological information gathered during the investigation has been pooled and reviewed permitting the formulation of a perspective concerning the characterisation of the stability of the site.

The predominant mobilising agencies considered in this investigation are major groundwater level fluctuations (>6m), ingress of water, ground vibrations and gravity. Use is made of a generalised list of evaluation factors to evaluate the hazard.

These factors are as follows:

- Receptacle development;
- Mobilising agencies, particularly ingress of water from leaking services;
- Potential sinkhole development space;
- Nature of the blanketing layer;
- Mobilisation potential of the blanketing layer;
- Bedrock morphology.

Receptacles or disseminated receptacles refer to any voids or cavities in the dolomite bedrock or in the overburden capable of receiving mobilised materials. *Receptacles are assumed to be present as no reliable geophysical tool exists to determine the location of these features. Consequently, the information gathered from boreholes, including penetration times, air loss, hammer action, etc, combined with geophysical and geological information are used to formulate an impression of the degree of voids.*

The potential sinkhole development space, where used, refers to the expected maximum size sinkhole that conservatively may be anticipated to be generated if sustained ingress of water were to occur. This factor is related to the depth of the receptacles or disseminated receptacles. The gravity survey, combined with borehole information strongly guides the appraisal of this factor.

The nature of the material covering the receptacles, be they above or in the bedrock, determines the susceptibility of the subsurface material to erosion by ingress of water. The presence of materials such as shales or intrusives, which can act as aquitards, serve to reduce the mobilisation potential and enhance the stability.

In the case of dramatic groundwater level fluctuations, the susceptibility of the soil material to mobilisation (i.e. consolidation settlement - subsidence formation, or ravelling and arch failure - sinkhole formation, due to pore pressure changes in soils), is strongly influenced by the position of the original groundwater level in the subsurface profile. In assessing the susceptibility of a subsurface profile to sinkholes and subsidences due to groundwater level drawdown, attention is given to the nature and extent of the material below the groundwater level.

For example, in the case of compressible dolomite residuum, as the groundwater recedes, pore pressures in the residual dolomite soils, typically characterized by high void ratios, gradually dissipate and the effective stress on the soil increases causing consolidation of the compressible material. A surface depression may occur gradually due to the load of the near-surface materials on the deeper lower density materials that settle into a denser state.

The general maximum magnitude of natural groundwater fluctuation in the Gauteng dolomites is in the order of 5m (Hobbs, 2004; Holland, 2007). Hence artificial groundwater level drawdown is generally defined as drawdown exceeding 6m. Experience shows that groundwater level drawdown (beyond seasonal variations) has the greatest negative impact on dolomite stability in areas of shallow groundwater levels (30 m or shallower) i.e. deeper (> 30 m) groundwater levels, pose less of a negative impact on stability, in the event of groundwater level drawdown or dewatering. However, where groundwater drawdown occurs in areas of deeper groundwater, the size of instability is typically anticipated to be large to very large.

In view of the factors discussed above the following characteristics have been extracted from the gathered information during the assessment process:

- borehole position relative to the gravity data.
- collar elevation.

- depth to dolomite bedrock*.
- depth to potential receptacles.
- depth to the present and original groundwater level.
- nature and thickness of blanketing layer i.e. material type, penetration times, etc.
- position of the bedrock with respect to the present and original groundwater level.
- thickness and nature of the soil materials above and below the present and original groundwater level.

*In the context of this report the concepts of thickness and depths for subsurface horizons are used as follows:

Depths		**Thicknesses (Alluvium / Colluvium / Residuum)**	
Range in m	**Appellation**	**Range in m**	**Appellation**
0-2/4	Near-surface	0-8/12	Thin
2/4-8/12	Shallow	8/12-16/24	Intermediate
8/12-16/24	Intermediate	16/24-36/44	Thick
16/24-36/44	Great Depth	36/44 and greater	Very Thick
More than 40	Very Great Depth	-	-

Inherent susceptibility is a reflection of the geological susceptibility of a karst area to an event (sinkhole or subsidence formation) and is expressed in three broad categories, namely low, medium and high. The following reference to incidences, gives a perspective of the magnitude of problems encountered in each of the of hazard zones in research areas.

It is important to note that these figures are largely derived from developments not effectively and appropriately designed or maintained.

Inherent Susceptibility	**Anticipated events per hectare over time (magnitude of problem)***
LOW	0 up to and including 0.1 events per hectare anticipated but occurrence of events cannot be excluded. Return Period is greater than 200 years.
MEDIUM	Greater than 0.1 and less than and equal to 1.0 events per hectare. Return period is between 200 and 20 years.
HIGH	Greater than 1.0 events anticipated per hectare. Return period is less than 20 years.
* that have occurred per hectare in a 20 year period in the "type" areas (statistics based on poor service design and maintenance)	

The study area is characterised in terms of eight standard Inherent Hazard Classes. These classes denote the chance of a sinkhole or subsidence occurring as well as its likely size (diameter).

The terminology used in terms of likely size of an event (sinkhole or subsidence) is defined as follows:

Maximum diameter of surface manifestation (in metres)	**Terminology**
< 2	Small-size
2-5	Medium-size
5-15	Large-size
> 15	Very large-size

The meaning/definition of each Inherent Hazard Class is as follows:

Inherent Hazard Class	Characterisation of Area
Class 1 Areas	Areas characterised as reflecting a **low** inherent susceptibility of **all sizes** of events occurring.
Class 2 Areas	Areas characterised as reflecting a **medium** inherent susceptibility of **small-size** events occurring.
Class 3 Areas	Areas characterised as reflecting a **medium** inherent susceptibility of **medium-size** events occurring.
Class 4 Areas	Areas characterised as reflecting a **medium** inherent susceptibility of **large-size** events occurring.
Class 5 Areas	Areas characterised as reflecting a **high** inherent susceptibility of **small-size** events occurring.
Class 6 Areas	Areas characterised as reflecting a **high** inherent susceptibility of **medium-size** events occurring.
Class 7 Areas	Areas characterised as reflecting a **high** inherent susceptibility of **large-size** events occurring.
Class 8 Areas	Areas characterised as reflecting a **high** inherent susceptibility of **very large-size** events occurring.

5.5. Design Hazard Class

While various combinations of Inherent Hazard Classes may be assigned to a particular zone, for example 4(1)//7, for development planning and design purposes a single Design Hazard Class is to be selected. This selection is dependent on factors such as groundwater monitoring and control and the like. In the stated example the Design Class may be 4 in an area where there is groundwater level control by the relevant authorities and 7 where no or only limited control is in place.

5.6. Dolomite Area Designation (SANS 1936 Part 2 (2012))

Dolomite Area Designations must be identified on sites located on or near dolomite land (land where dolomite is located at or near [less than 100m] ground surface).

The definitions of the Dolomite Area Designations, as defined in SANS 1936 Part 1 (2012), are as follows:

D Designation	Description
D1	No precautionary measures are required.
D2	General precautionary measures, in accordance with the requirements of SANS 1936-3, that are intended to prevent the concentrated ingress of water into the ground, are required.
D3	Precautionary measures in addition to those pertaining to the prevention of concentrated ingress of water into the ground, in accordance with the relevant requirements of SANS 1936-3, are required.
D4	Additional site-specific precautionary measures are required.*

*Section 4.3.4 states the following specific measures for D4 areas:

a) site characterization, analysis and design, specification of precautionary measures, supervision of implementation and formulation of dolomite risk management plan shall be undertaken by a Competence Level 4 geo-professional;

b) the foundation design and design of the structure, precautionary measures and dolomite risk management plan shall specifically address and effectively mitigate the dolomite risks *(should read hazard)* present on the site;

c) the site characterization, foundation design and design of the structure, precautionary measures and dolomite risk management requirements shall be reviewed and approved by an independent Competence Level 4 geo-professional and, where relevant, by a structural engineer with a similar level of competence; and

d) all aspects of the development proposal shall be reviewed and approved by the local authority who may request a further review by an authority-designated Competence Level 4 peer if required.

5.7. Permissible Land Use (SANS 1936-1, 2012)

According to SANS 1936 Part 1, Table 2 (2012), the development types suitable for the eight standard Inherent Hazard Classes are summarised as follows:

Inherent Hazard Class	Land Usage Type	Land Usage Permitted with Dolomite Area Designation and footprint investigation requirement in terms of Deemed-To-Satisfy
1	Commercial and miscellaneous non-residential usage	C1 (D3 + FPI), C2 (D3 + FPI), C3 (D2 + FPI), C4 (D2 + FPI), C5 (D2 + DLI), C6 (D2 + DLI), C7 (D2), C8 (D2)
	High rise dwelling units	RH1 (D4), RH2 (D2 + FPI), RH3 (D2 + FPI)
	Low rise dwelling units	RL1 (D2 + FPI), RL2 (D2 + FPI)
	Dwelling houses	RN1 (D2), RN2 (D2), RN3 (D2)
2	Commercial and miscellaneous non-residential usage	C1 (D3 + FPI), C2 (D3 + FPI), C3 (D3 + FPI), C4(D3 + FPI), C5 (D3 + DLI), C6 (D3 + DLI), C7 (D3+ FPI), C8 (D3)
	High rise dwelling units	RH1-RH2 (D4), RH3 (D3 + FPI)
	Low rise dwelling units	RL1 (D2 + FPI), RL2 (D2 + FPI)
	Dwelling houses	RN1 (D3), RN2 (D3), RN3 (D3)
3	Commercial and miscellaneous non-residential usage	C1 (D3 + FPI), C2 (D3 + FPI), C3 (D3 + FPI), C4 (D4), C5 (D3 + DLI), C6 (D3 + DLI), C7 (D3 + FPI), C8 (D3)
	High rise dwelling units	RH1-RH2 (D4), RH3 (D3 + FPI)
	Low rise dwelling units	RL1 (D4), RL2 (D3 + FPI)
	Dwelling houses	RN1 (D4), RN2 (D3), RN3 (D3)
4	Commercial and miscellaneous non-residential usage	C1 (D3 + FPI), C2 (D3 + FPI), C3 (D3 + FPI),), C4 (D4), C5 (D3 + DLI), C6 (D3 + DLI), C7 (D3 + FPI), C8 (D3)
	High rise dwelling units	RH1-RH2 (D4), RH3 (D3 + FPI)
	Low rise dwelling units	RL1 (D4), RL2 (D3 + FPI)
	Dwelling houses	RN1 (D4), RN2 (D3), RN3 (D3)
5	Commercial and miscellaneous non-residential usage	C1 (D3 + FPI), C2 (D3 + FPI), C3 (D3 + FPI),), C4 (D4), C5 (D3 + DLI), C6 (D3 + DLI), C7 (D3 + FPI), C8 (D3)
	High rise dwelling units	RH1-RH2 (D4), RH3 (D3 + FPI)
	Low rise dwelling units	RL1-2 (D4), RL2 (D3 + FPI)
	Dwelling houses	RN1-RN2 (D4), RN3 (D3 + FPI)
6	Commercial and miscellaneous non-residential usage	C1 (D4), C2 (D3 + FPI), C3 (D3 + FPI),C4-C5 (D4), C6 (D3 + DLI) , C7 (D3 + FPI), C8 (D3)
	High rise dwelling units	RH1-RH3 (D4)
	Low rise dwelling units	RL1-RL2 (D4)
	Dwelling houses	RN1-RN3 (D4)
7	Commercial and miscellaneous non-residential usage	C1-C5 (D4), C6 (D3 + DLI), C7 (D4), C8 (D3)
	High rise dwelling units	RH1-RH3 (D4)
	Low rise dwelling units	RL1-RL2 (D4)
	Dwelling houses	RN1-RN3 (D4)
8	Commercial and miscellaneous non-residential usage	C1-C8 (D4)
	High rise dwelling units	RH1-RH3 (D4)
	Low rise dwelling units	RL1-RL2 (D4)
	Dwelling houses	RN1-RN3 (D4)

Land Use Class Definitions	
Commercial and miscellaneous non-residential usage	
C1	Places of detention, police stations, and institutional homes for the handicapped or aged
C2	Hospitals, hostels, hotels
C3	Commercial developments ≤ 3 storeys, including railway stations, shops, wholesale stores, offices, places of worship, theatrical, indoor sports or public assembly venues, other institutional land uses, such as universities, schools, colleges, libraries, exhibition halls and museums, light (dry) industrial developments, dry manufacturing, commercial uses such as warehousing, packaging, electrical sub-stations, filling stations

Land Use Class Definitions	
C4	Commercial developments > 3 storeys, including railway stations, shops, wholesale stores, offices, places of worship, theatrical, indoor sports or public assembly venues, other institutional land uses, such as universities, schools, colleges, libraries, exhibition halls and museums, light (dry) industrial developments, dry manufacturing, commercial uses such as warehousing, packaging, electrical sub-stations
C5	Fuel depots, processing plants or any other areas for the storage of liquids, waste sites
C6	Outdoor storage facilities, stock yards, container depots
C7	Parking garages
C8	Parking areas
High rise dwelling units	
RH1	> 10 storeys
RH2	> 3 storeys, no higher than 10 storeys with a population of ≤ 1 500 people per hectare
RH3	> 3 storeys with a residential coverage ratio of ≤ 0,4, no higher than 10 storeys, and a population of ≤ 800 people per hectare
Low rise dwelling units	
RL1	≤ 3 storeys with 80 to 120 units per hectare and a population not exceeding 600 people per hectare
RL2	≤ 3 storeys with up to 80 units per hectare and a population not exceeding 400 people per hectare
Dwelling houses	
RN1	Up to 60 dwelling houses per hectare with stands larger than 150 m^2, and a population of ≤ 300 people per hectare
RN2	Up to 25 dwelling houses per hectare with stands no smaller than 300 m^2, and a population of ≤ 200 people per hectare
RN3	Up to 10 dwelling houses per hectare with 1 000 to 4 000 m^2 stands, and a population of ≤ 60 people per hectare

According to SANS 1936 (2012) Part 1, Table 3, the infrastructure and social services suitable for the eight standard Inherent Hazard Classes are summarised as follows:

Infrastructure and social facilities		**Inherent hazard class determined in accordance with the requirements of SANS 1936-2**							
Designation	**Description**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		Dolomite area designation							
IN1	Trunk roads (national and regional roads which facilitate intercity travel) and primary distributor roads (major arterial roads forming the primary network for an urban area as a whole), railway lines, power lines, runways, bulk pipelines, including water, sewer, fuel and gas lines, pump stations	D2	D3					D4	
IN2	Reservoirs and public swimming pools, water care works, attenuation and retention ponds for stormwater management and artificial lakes	D2	D3				D4		
IN3	Cemeteries	D3				D4			
IN4	Dams, slimes dams	D3	D4						
IN5	Solid waste disposal facilities	D3						D4	

5.8. Monitoring Designations (SANS 1936-4, 2012)

According to SANS 1936 Part 4 (2012) Monitoring Designations must be identified and delineated according to the Inherent Susceptibility characterisation of the site and knowledge of problems which could impact on the infrastructure on site. The generic Monitoring Activities considered appropriate for dolomite land are as follows:

Annotation	Activity*	Reaction	Purpose
A	**Visual inspections of ground, structures and above ground infrastructure** (e.g. buildings, taps, gardens, private and public open space etc.) and **dry services, sleeves, ducts, manholes and facility chambers for water ingress**: • Examine buildings for cracks. • Visual checks for outside dripping taps and pressure valves. • Visual checks for damp or moss grown areas. • Visual check for over-wetting of gardens. • Visual check for blocked drainage ports in garden walls. • Visual check for cracks in the ground.	Any evidence of cracking or ground settlement should immediately be reported and investigated. Any evidence of blockages should be reported and cleared immediately, or cracks repaired once settlement is halted. Any water ingress in dry services, sleeves, ducts, manholes and facility chambers shall be reported and point of entry repaired and investigated.	Monitor, control and prevention of concentrated ingress of water
B	**Visual inspection of stormwater systems**: • Visual checks for debris in open storm water channels at, for example, the start of the rainy season and after heavy storms. • Visual checks for water flowing out of stormwater manholes at the start of the rainy season and after heavy storms. • Search for ground cracks and cracks in lined and unlined channels.		
C	**Testing of wet-services for leaks**: • Regular air and water tests on wet-services. • In waste water and stormwater pipes with a diameter greater than 100 mm wet-services to be inspected by camera.	Any leaks to be reported and repaired immediately.	
D	**Monitoring of structures and ground levels:** In many instances visual inspections may not be sufficient: It may be necessary to undertake precision ground-surface levelling, particularly in areas that have been rehabilitated after a ground movement event. Such levelling must be undertaken by a surveyor, recorded and stored in the databank and appropriate actions taken when excessive settlement continues.	Any evidence of movement must be reported and investigated and appropriate repair work undertaken.	Monitor the effects of concentrated ingress of water or groundwater level drawdown

Annotation	Activity*	Reaction	Purpose
E	**Monitoring of the groundwater level:** This activity not only entails the measuring and recording of the groundwater level, but also the analysis and understanding of groundwater level trends over time and the control of excessive [beyond seasonal fluctuations] groundwater level drawdown: • Drill and equip strategic boreholes with the necessary equipment to measure the groundwater level. • Recorded groundwater level, monitoring well number, date and other relevant observations. • Analyse groundwater level trend over time on a regular basis. • Report artificial/excessive groundwater level drawdown to Municipality and the Department of Water Affairs. • Investigate cause of artificial/excessive groundwater level drawdown. • Arrest artificial/excessive groundwater level drawdown.	Evidence of lowering (exceeding 6 m) must be reported to Municipality and the Department of Water Affairs and appropriate action taken to ensure the groundwater level returns to within natural seasonal fluctuations.	Monitor, control and prevention of groundwater level drawdown
*If there is no evidence of a particular problem, this result should be recorded.			

Note that in SANS 1936 Part 4 (2012), pg. 15, the *visual inspection of dry services, sleeves, ducts, manholes and facility chambers* is included as a separate monitoring activity (as Annotation D), with the Annotations D and E as listed above, annotated as E and F. For simplicity sake this activity has been included in Annotation A above.

The Frequency with which each Activity is to be performed is selected from the following categories:

Annotation	Frequency
$()^{DAILY}$	Activities to be undertaken daily.
$()^{WEEKLY}$	Activities to be undertaken weekly.
$()^{1}$	Activities to be undertaken once a month.
$()^{3}$	Activities to be undertaken quarterly.
$()^{6}$	Activities to be undertaken bi-annually.
$()^{12}$	Activities to be undertaken annually.
$()^{24}$	Activities to be undertaken once every two years.
$()^{NA}$	NO ACTION REQUIRED CURRENTLY
$()^{tbd}$	TO BE DETERMINED

6. GEOLOGY

6.1. Regional Geological Setting

The lithological units (and their weathered derivatives), anticipated according to regional geological mapping information are as follows (refer to Figure 3):

Lithology	Lithostratigraphic Unit
Dolerite dykes and sills (Drakensburg Group age/150 Ma)	- (di)
Shale/sandstone/siltstone/mudstone etc. and soil derivatives	Ecca and Dwyka Groups, Karoo Supergroup (Pv, C-Pd, Pe)

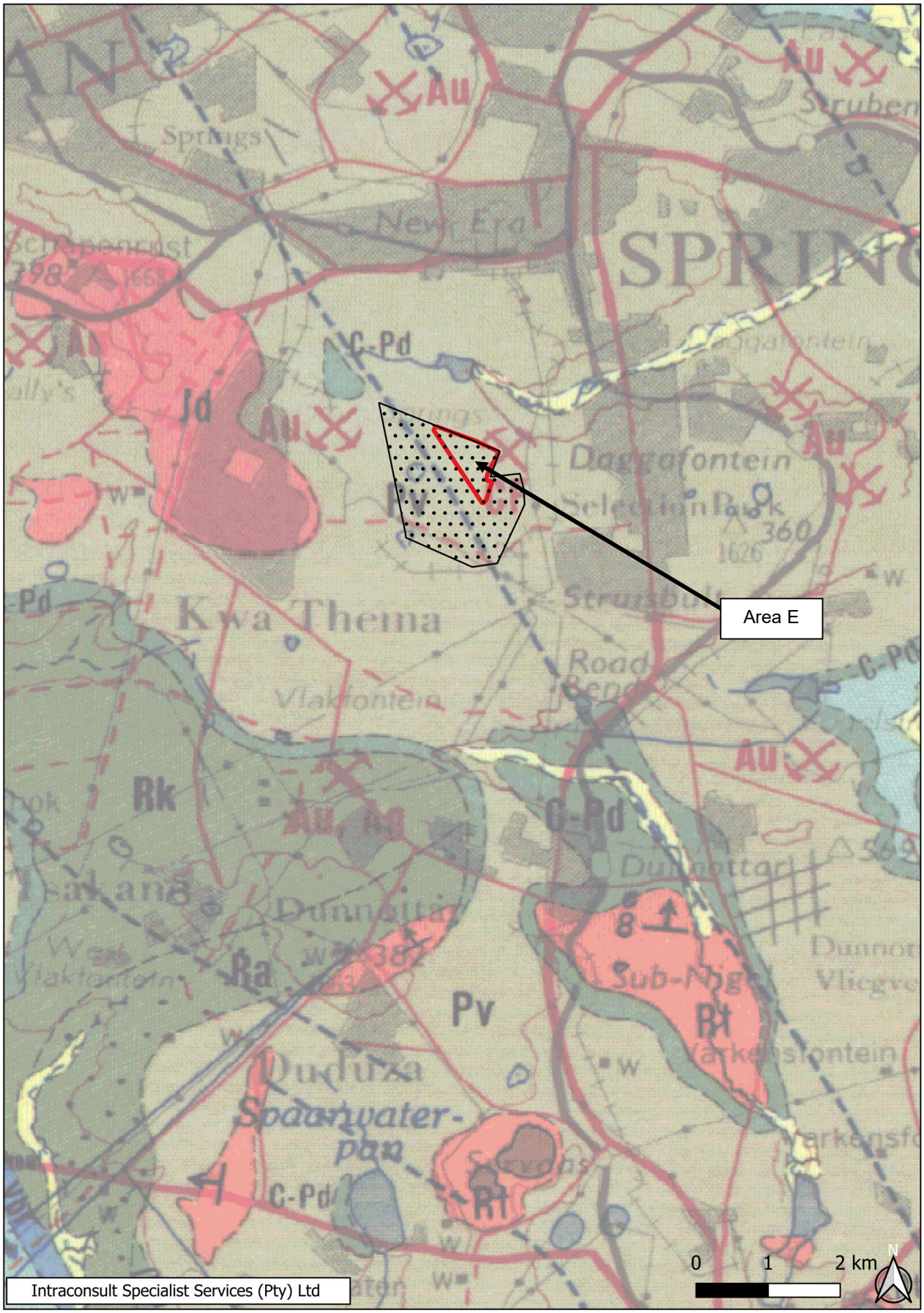


Figure 3: Excerpt of 1:250 000 scale Geological Map (Sheet 2628 East Rand)

Lithology	Lithostratigraphic Unit
Syenite dykes (Pilanesberg dyke swarm age/1120-1310 Ma)	- (sy)
Diabase dykes and sills (Bushveld Igneous Complex age/2061 Ma)	- (di)
Shale, quartzite and soil derivatives	Pretoria Group, Transvaal Supergroup (Vt, Vb, Vh & Vha)
Chert, chert gravel, silty clay and clayey silt, dolomite, dolomite residuum and associated chert breccia	Eccles Formation, Malmani Subgroup, Chuniespoort Group, Transvaal Supergroup (Vmd)
Dolomite, dolomite residuum	Lyttelton Formation, Malmani Subgroup, Chuniespoort Group, Transvaal Supergroup (Vmd)
Chert, chert gravel, silty clay and clayey silt, dolomite, dolomite residuum and associated chert breccia	Monte Christo Formation, Malmani Subgroup, Chuniespoort Group, Transvaal Supergroup (Vmd)
Dolomite, dolomite residuum	Oaktree Formation, , Malmani Subgroup, Chuniespoort Group, Transvaal Supergroup (Vmd)

Unconsolidated materials deemed to be various recent deposits (most likely 24 Ma [Miocene Epoch] and younger) are anticipated to mantle the hard rock geology (and residual product thereof). The material varies in thickness, sedimentological- and geotechnical properties:

Post-Gondwana Deposits	
Fill material	Variable composition
Pedocretes (cemented soils)	e.g., calcrete (cemented or replaced by carbonates), ferricrete (cemented or replaced by iron oxides), silcrete (cemented or replaced by silica), manganocrete (cemented or replaced by manganese oxides) OR Calcified- (less than 50% cement or replacement material), ferruginised-, silicified -OR manganised soil
Colluvium: Talus (Course Colluvium) or Hillwash	Soils of clayey and silty composition or gravels
Sandy Soils of Mixed Origin	Silty and clayey sands, sandy silts, sandy clays (Red brown, orange brown, or yellow in colour)

The site is located on dolomite land. Karst development has led to a highly variable dolomite and chert bedrock topography, with deep weathering having taken place along linear features such as fractures. Cavernous conditions can be expected both within the bedrock and the overburden. The process of dissolution has resulted in a vertically zoned succession of residual products, which in turn are generally overlain by geologically younger formations or soils.

Hard, unweathered dolomitic bedrock is overlain by slightly weathered jointed bedrock and thereafter, through a sudden, dramatic transition, to totally weathered and low strength, insoluble residual material consisting of mainly manganese oxides (wad), chert and iron oxides that reflect the original insoluble matrix structure.

6.2. Site Specific Conditions (2011/2013)

- The southern part of Area E was excavated by Corobrik for “clays”. Backfilled areas exist over large areas.
- Dolerite was intercepted in the borehole drilled in the Rand Water servitude near the inferred position of the dyke.
- Dwyka age sediments were identified in five of the nine holes drilled in 2011. Dwyka sediments were profiled at depths ranging from 17-28m.
- The depth to dolomite bedrock varies from 20-38m.
- In the boreholes in the quarry area dolomite rock was encountered is 16m and 18m below the backfill

level in boreholes 5414 and 5212 respectively. The dolomite is overlain by low permeability materials.

- The contact zones between the Dwyka Formation or the Vryheid Formation and the dolomite not highly leached. Low density and erodible materials were not intercepted.
- Sample and air loss was recorded in one of the 2013 boreholes (Borehole 40/12A).

Summaries of the horizons intercepted are captured in the respective reports and presented below:

BOREHOLE NR.	COLLAR ELEVATION m)	TRANSPORTED (T)	TRANSPORTED (T) RESIDUAL (R) PEDOCRETE (P)	KAROO (VRYHEID)	DOLERITE (WEATHERED)	DOLERITE (HARD ROCK)	DWYKA	NO SAMPLE	CAVITY	DOLOMITE BEDROCK (slightly leached (m - m)	DOLOMITE BEDROCK (hard rock)	DEPTH OF HOLE (m)	GROUNDWATER LEVEL (m)
4414			0-13 (Fill)	13-19 (Mr)						19-21	21-29	29	4,8
4911			0-18 (Fill)	18-30 (Mr)						30-33	33-41	41	5,5
5017			0-8 (Fill)	8-21 (Mr)			21-26			26-30	30-38	38	3,7
5212			0-9 (Fill)	9-16 (Mr)			16-18				18-21	21	1.0
5414			0-8,4 (Fill)	8,4-10 (Mr)			10-16				16-24	24	4,3
5507		1-2	0-1 (Fill); 2-6 (P)	6-32 (Mr)						32-33	33-40	40	2.0
5521/5422		0-1	1-4 (Fe)	4-22 (Mr); 22-25 (Mr, Carb); 25-34 (Mr)						34-38	38-47	47	14,8
5818		0-8	8-9 (Fe)	9-25 (Mr); 25-37 (Mr, Carb)						37-41	41-48	48	16,8
6024		0-2		2-17 (Mr)			17-20			20-24,5	24,5-33	33	1,2
F		3-5	0-3 (Fill)	5->19 (Mr)									6.0
7/24		0-4,5		4,5-12 (Mr)	26-41		12-21?			21-26	41-54	54	46.0
V10			0-9 (Fill)	9-13			13-20,5				20,5-27	27	Coll
40/12A			0-19 (Fill)				19-31,3 (N/S)	19-37			31,3-37 (N/S)	37	2,8
61/21		0-1		1-14,5			14,5-18			18-22,2	22,2-28	28	7,9

6.3. Past Sinkholes and Subsidences

Records suggest that no sinkholes or subsidences have occurred on or in close proximity to the site.

7. GEOHYDROLOGY

7.1. Quaternary Catchment Area

According to the country wide Quaternary Catchment data the site is located in Catchment C21E.

7.2. Groundwater Management Area (WMA)

According to the country wide Water Management Areas data the sites are located in the **Upper Vaal WMA.**

7.3. Dolomite Groundwater Management Area

According to the Water Geosciences Consulting Dolomite Hydrogeological Compartment Map[1] (2009) the site is located in the **East Rand Basin GMA.**

7.4. Dolomite Groundwater Levels Regionally

According to the Water Research Commission report of 2014, Number TT 553/14, Section 1.3.3 Groundwater Levels, the DWS operates and maintains 129 groundwater level monitoring stations in Region 10 (The Karst Belt). All of these target the dolomitic groundwater resource. The statistical characteristics for each dolomitic groundwater compartment or area are presented in Table 11.3.

[1] *The map represents the subdivision of the dolomitic surface outcrop of the Ghaap Plateau and the Gauteng/North West dolomites. The major delineation is represented by the dolomitic Groundwater Management Areas (GMA). The names of these GMA's typically correspond closely to the common known dolomitic compartments as established in the late 1980's by the DWS. However, the delineation does not necessarily represent a dolomite compartment or unit.*

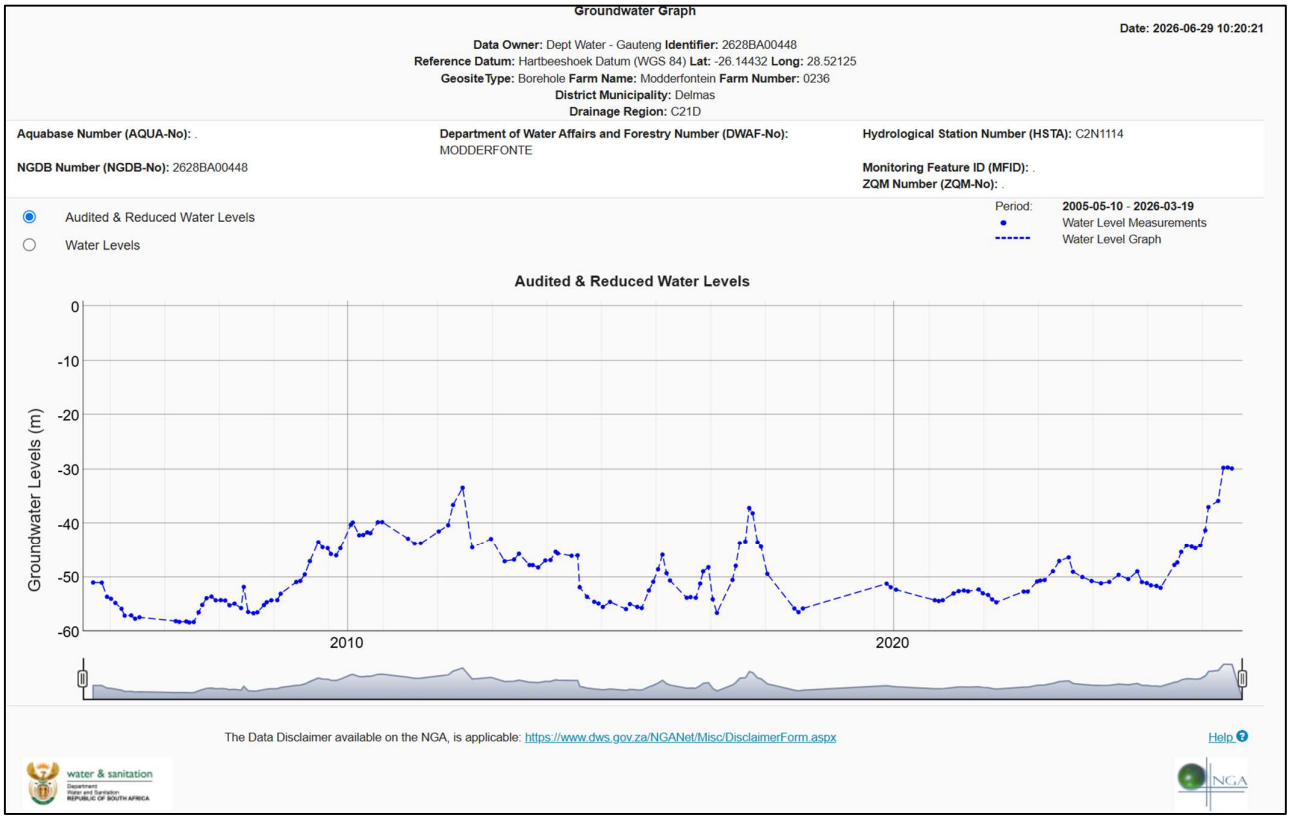
Table 11.3. Groundwater level information for dolomitic compartments.

Compartment	Groundwater level depth (m.b.g.s)			Groundwater level elevation (m.a.m.s.l.)		
	10 percentile	Mean	90 percentile	10 percentile	Mean	90 percentile
Rietvlei	9.66	16.44	36.74	1478.40	1483.50	1489.91
Witkoppies	1.73	37.30	71.77	1500.20	1505.62	1517.61
Elandsfontein	27.09	43.56	64.95	1544.44	1580.82	1603.73
Bapsfontein	49.12	79.20	79.94	1528.33	1559.57	1559.95
Varkfontein-Knoppiesfontein	6.03	9.65	18.51	1605.03	1610.35	1621.51
Delmas	5.53	17.29	44.97	1539.02	1560.45	1583.90
East Rand	50.13	51.81	53.49	1568.29	1569.29	1570.28

The monitoring stations identified in the East Rand Basing are indicated in Table 11.4 of report TT 553/14:

Compartment/ Area	Station	Depth to Groundwater Level (m.b.g.s)				Complete Record Late 1980s to 2007		Selected Record 1997 to 2007
		Min.	Mean	Median	Max.	Max Δh	Cumulative Δh	Cumulative Δh
Varkfontein-Knoppies-fontein	C2N891	6.70	8.87	9.06	10.75	4.05	-0.20	-2.18
	C2N893	2.29	4.84	4.57	8.47	6.18	+2.88	-1.38
	A2N0783*	16.20	18.93	18.96	20.76	4.56	-	-2.56
East Rand Basin	C2N1113*	42.52	52.08	53.23	54.27	11.75	-	4.32
	C2N1114*	51.03	55.92	56.79	58.36	7.33	-	-0.25

The groundwater levels in Station C2N1114 over time is indicated below:



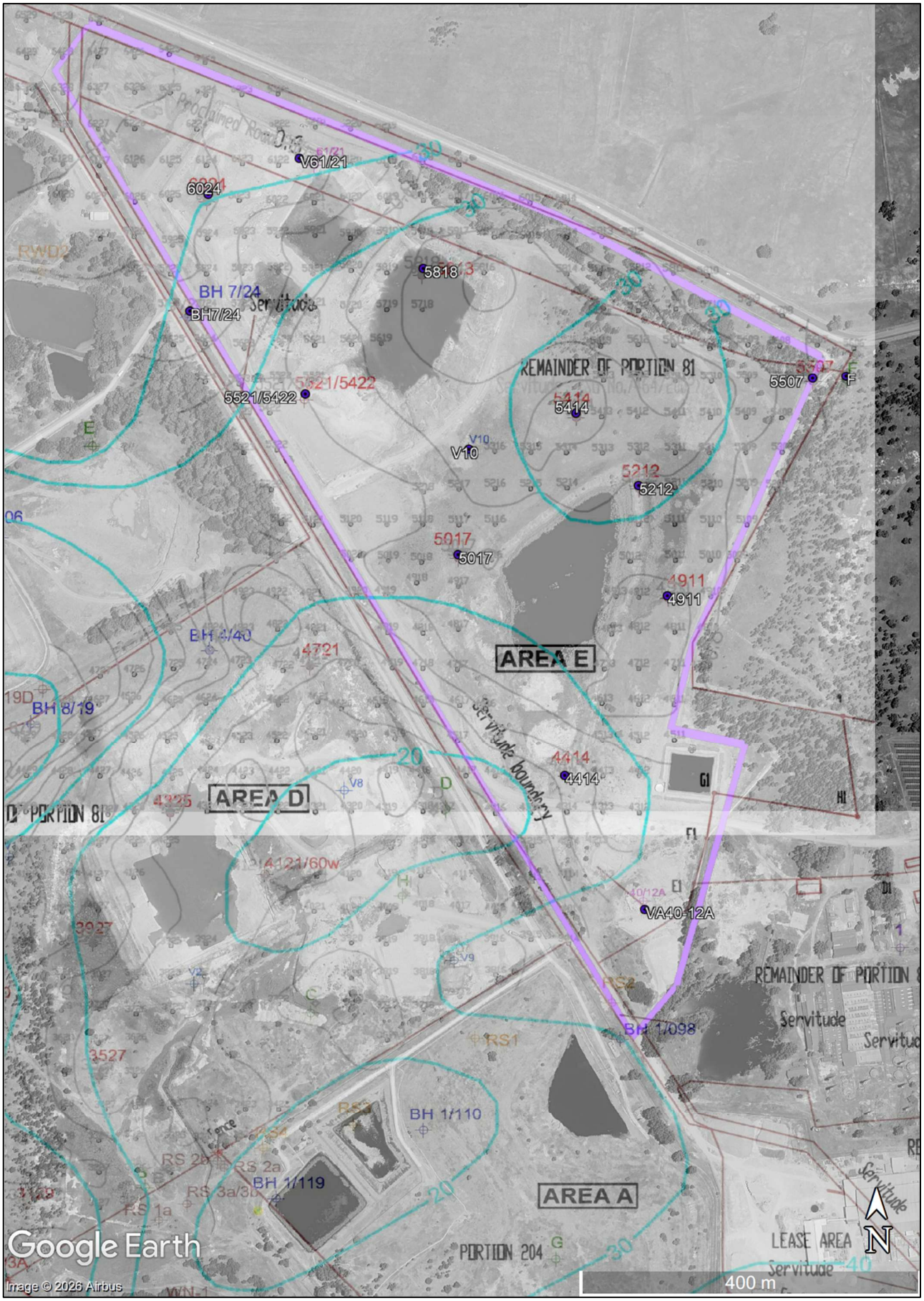


Figure 4: Residual Gravity and Borehole Positions (2013)

7.5. Groundwater Depths (Site Specific)

During the drilling process the driller notes the depth of groundwater strike/s. The driller also observes the hole after drilling (typically 24 hours after completion) for the presence of groundwater, using a dip meter tool. The depths (if reported) are typically reproduced on the borehole log. A summary of pertinent information follows:

No.	Borehole number	Depth of borehole (m)	Groundwater rest depth (m)
1	4414	29	4,8
2	4911	41	5,5
3	5017	38	3,7
4	5212	21	1,0
5	5414	24	4,3
6	5507	40	2,0
7	5521/5422	47	14,8
8	5818	48	16,8
9	6024	33	1,2
10	7/24	54	46
11	V10	27	collapsed
12	V40/12A	37	2,8
13	V61/21	28	7,9

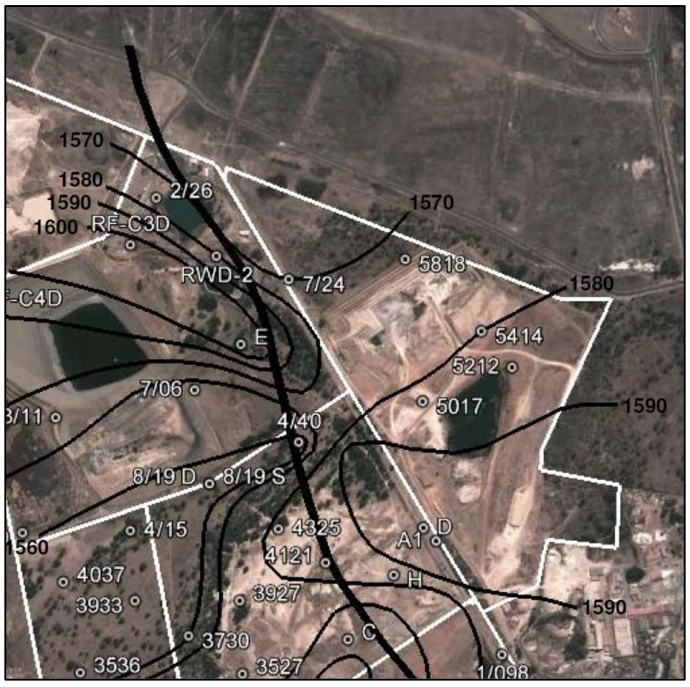
The 2011 reports states the following regarding groundwater:

- The depth to water level varies considerably across the site.
- In severely disturbed areas, due to the mining of clay and excavation of material for road building purposes (for example Areas E and F), ponding of rain and storm water occurs and elevated or perched piezometric levels are observed that do not reflect the natural static groundwater level.
- More representative and deeper static water levels are expected towards the end of the winter once surface water has dried up. Some boreholes in this area drilled into dolomite have water levels in excess of 55 m below ground level.
- Representative static water levels across the area are expected to indicate a north-westerly ground water flow direction towards the Blesbokspruit.
- To what degree, if at all, the dyke intrusion influences groundwater movement is not yet clear.

The following report provides information with respect to the deeper groundwater aquifer:

Titled: “An Assessment Of The Geohydrological Conditions At The Proposed Tonk Meter Road Landfill Site Springs, Gauteng Province”, by by R Meyer R Meyer, dated February 2013, Report No. 01/2013.

The report indicates that the groundwater level elevations range from 1570m AMSL in the north to 1590m AMSL in the south of Area E.



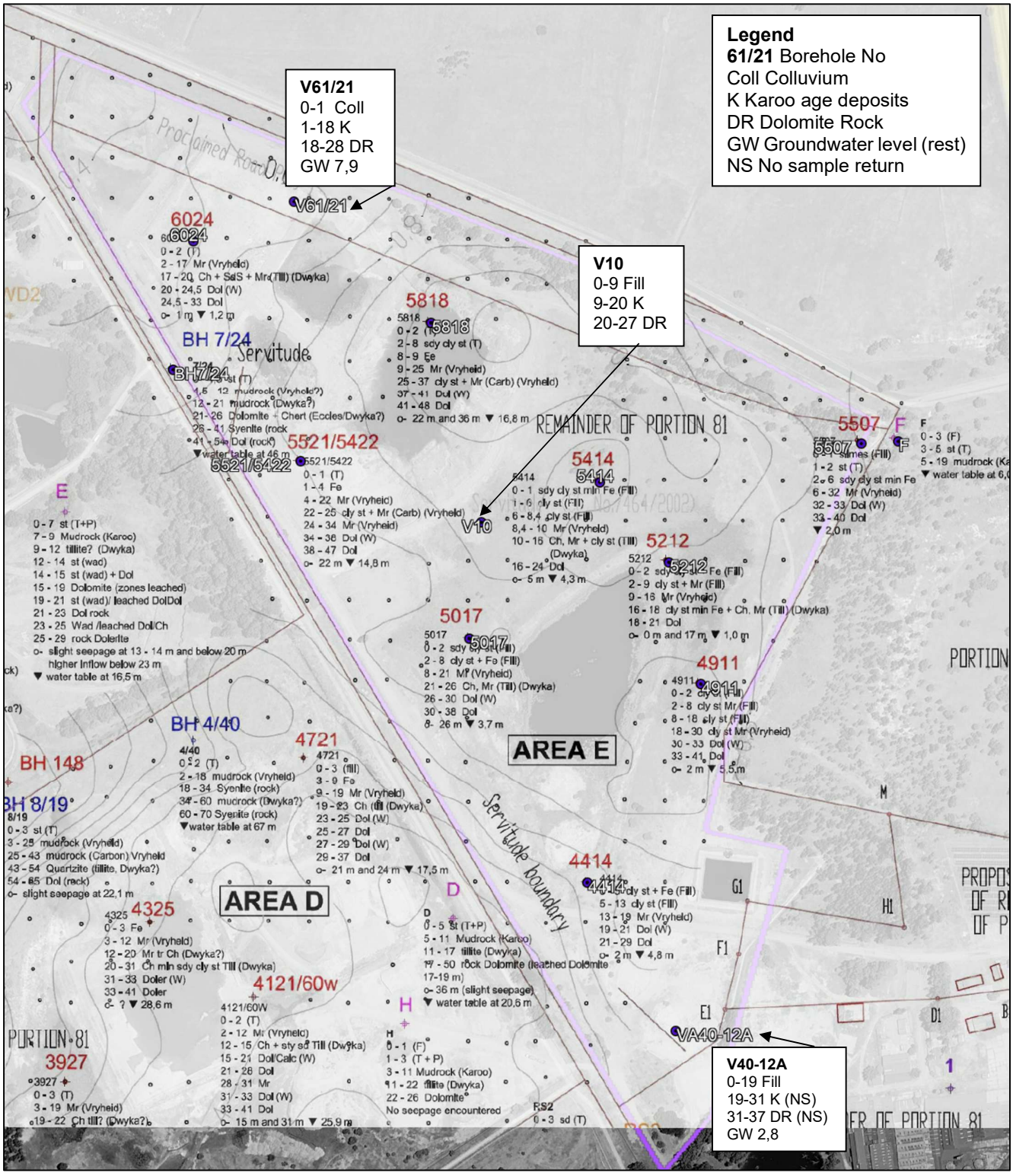


Figure 5: Borehole Summaries

8. DOLOMITE STABILITY CHARACTERISATION

8.1. Dolomite Hazard Assessment (2011)

According to the 2011 study:

- The Rand Water pipeline forms the southwestern boundary.
- The southern part of the area has been excavated by Corobrik for “clays”. Rehabilitation has started on the southern part of the excavated area which resulted in backfill being present over

large areas. Most of the southern holes i.e. 4414, 4911, 5017, 5212, and 5414, have therefore been drilled through backfill.

- Some temporary earth dams are also present on the backfilled area and the area south of the northern excavated face is filled with water.
- Some trenches and old deep partly backfilled excavations are present near the southeastern boundary while some shallow water filled excavations are also present on the northern area. High unnatural water tables are therefore present and this resulted in difficult drilling conditions and contamination in some of the holes.
- Corobrik orthophoto plans indicate a natural ground elevation of about 1609m AMSL parallel to the northeastern boundary and an elevation of 1615m AMSL at the northeastern corner of Area A (the Ekurhuleni waste disposal site).
- A magnetic anomaly shows a dyke to be present west of the RW pipeline and a number of intersections of dolerite sheets or sills were encountered in some of the boreholes. In Area E only BH 7/24 intersected dolerite. According to the magnetic surveys this location should be close to the dyke. No dolerite was found in any of the other boreholes drilled in Area E. If dolerite is present on Area E it must therefore be at deeper levels.
- Dwyka age sediments were profiled in 5 of the 9 holes (BH F was only drilled to 19 m in Vryheid Formation mudrocks). The Dwyka material was found at depths ranging from 1728m (from collar elevation) and the thicknesses of the Dwyka vary between 2-9m.
- The depth to dolomite correlates reasonably well with the gravity data. Dolomite is shallower on the northern area and deeper near the present northern quarry face. It is shallower in the central part and southern corner which have been mined and deeper on the eastern boundary and central southwestern boundary.
- Dolomite residuum is absent (as for Area D) but a slightly leached dolomite rock with penetration times about of 1 to 2 min/m is also present near the contact with the Dwyka or Vryheid Formation sediments
- The whole area is overlain by Karoo mudrocks.
- The upper part of the Karoo mudrock profile can be seen in the northern quarry face. It consists of sandy ferruginised material near surface with nodular ferricrete lower down. This changes into ferruginised residual mudrock followed by mainly structureless reworked slightly ferruginised clayey silt (completely weathered mudrock). All the above is referred to as “overburden” by the quarry operators. This is followed by a soft to very soft rock shale (“yellow clay”), in turn followed by a purplish shale (plastic clay) with the fire clays (a grey mudrock) further down. The fire clay is below the present water level in the quarry and is about at the depth to which the area was mined (about 15 – 22 m). Borehole 5521/5422 indicates that some carbonaceous shale often underlies these sediments.
- The floor levels of the future waste cells have not been determined and it is highly unlikely that they will ever reach the dolomite level but it will be a requirement to leave at least a 3m well consolidated layer above the dolomite followed by the base preparation layer, the under drainage and monitoring layer, the upper lining layers and the leachate collection layer.
- The Inherent Hazard Class is IHC 1, the Dolomite Area Designation is D2.

8.2. **Supplementary Dolomite Stability Investigation (2013)**

The report states the following salient points-

Servitude area

- Three boreholes are situated close to the southern boundary of the servitude (boreholes 60/24, 58/18 and 55/07).
- Dolomite rock was encountered at 20m, 37m and 32m in the three boreholes respectively.
- The dolomite is overlain by Vryheid Formation rocks which include some carbonaceous material in hole 58/18. Dwyka Group (tillite) is present above the dolomite in 60/24.
- Slightly weathered dolomite rock is present from 18,0 to 22,2 m followed by unweathered rock down to 28,0 m (EOH). No air or sample loss occurred in the hole.

Southern area

- Borehole 40/12A was drilled in the southern area where the existing gravity survey was extended. Old quarry plans show that the area was previously excavated.
- The drilling results confirm that the area was excavated in the past and then backfilled. Backfilled material consisting of brick, ash and mudrock, in a matrix of silty clay, is present down to 19,0m and no samples could be recovered from below the fill/rock contact.
- The material from 31,3-37 m is rock with penetration times varying from 3:19 – 3:44 min:sec/m indicating hard rock.
- The Inherent Hazard Class is IHC 1, the Dolomite Area Designation is D2.

8.3. Dolomite Hazard Assessment Review (2026)

The investigation and hazard assessment procedures are presented in Sections 5 of this report.

A summary of subsurface conditions intercepted in boreholes drilled in Area E follows:

Borehole Number	Transported material (m-m)	Pre or Post Malmani Subgroup (m-m)	Chert Residuum		Dolomite Residuum (m-m)	Weathered dolomite rock (m-m)	Dolomite Bedrock m	Ground water rest level
			Fines subordinate (m-m)	Fines predominant (m-m)				
4414	0-13 Fill	13-19 K	-	-	-	19-21	21-29	4,8
4911	0-18 Fill	18-30 K	-	-	-	30-33	33-41	5,5
5017	0-8 Fill	8-26 K	-	-	-	26-30	30-38	3,7
5212	0-9 Fill	9-18 K	-	-	-	-	18-21	1,0
5414	0-8 Fill	8-16 K	-	-	-	-	16-24	4,3
5507	1-20-1 Fill, 2-6 Pedo	6-32 K	-	-	-	**32-33**	**33-40**	**2,0**
5521/5422	0-1,1-4 Pedo	4-34 K	-	-	-	34-38	38-47	14,8
5818	0-88-9 Pedo	9-37 K	-	-	-	37-41	41-48	16,8
6024	0-2	2-20 K	-	-	-	20-24	24-33	1,2
F	0-3 Pedo, 3-5	5-19 K	-	-	-			6,0
7/24	0-4	4-21 K 26-41 Dolerite	-	-	-	21-26	41-45	45,0
V10	0-9 Fill	9-20 K	-	-	-	-	20-27	
V40/12A	0-19 Fill	19-31 NSR	-	-	-	-	31-37 NSR	2,8
V61/21	0-1	1-18 K	-	-	-	18-22	22-28	7,9

It appears that Boreholes 5212 and 5414 were drilled in a gravity high anomaly. To the east, west and south of this anomaly, the depth to and thickness of the Karoo material increases.

i. Blanketing Layer

The blanketing layer is of **great thickness**, consisting of relatively thick horizons of Karoo deposits. The blanketing layer is anticipated to be characterised by **poor internal drainage characteristics**. Prolonged ingress of water is unlikely to lead to erosion and sinkhole or subsidence formation.

ii. Potential Development Space

In general, the depth to dolomite bedrock, and the potential receptacles determine the Potential Development Space (PDS). The PDS is characterised as, typically, **large**.

iii. Hazard Classification (with Ingress of Water as a Triggering Mechanism)

Considering the anticipated low permeability of materials above dolomite bedrock, the area exhibits a low susceptibility of sinkhole and subsidence formation. In areas where Karoo cover thins a medium susceptibility of sinkhole and subsidence formation is anticipated.

i. Hazard Classification (with Groundwater Drawdown as a Triggering Mechanism)

The dolomite groundwater level is recorded at 51m deep in 2005 in the nearest Department of Water and Sanitation dolomite aquifer monitoring station. The current groundwater level is 29m deep. The mean groundwater level elevation is 1469m AMSL, as determined by the Water Research Commission, refer to Section 7.4).

The shallow groundwater levels reported in the 2011 and 2013 boreholes may represent perched groundwater. The deeper groundwater level recorded in Borehole 7/24 (45m deep) may represent the dolomite groundwater level. The groundwater level elevation as determined by the R Meyer report is 1570m to 1590m AMSL across Area E, some 20-40m deep.

Anticipating the possibility of artificial groundwater level drawdown, drawdown would typically occur within dolomite rock. The susceptibility to sinkhole and subsidence formation is considered to be low.

ii. Impact of Lowering of the Groundwater Level/Base Level of Erosion on the Action of Ingress

In the event that the groundwater level is drawn down significantly (6m or more), the hazard classification remains unchanged.

Area E is broadly characterised in terms of a single primary Inherent Hazard Class area, described as follows:

Inherent Hazard Class	Description
1/2//1	An area reflecting a **low to medium** susceptibility of sinkhole and subsidence formation with respect to ingress of water and a **low** susceptibility of all-size sinkhole and subsidence formation with respect to anthropogenic groundwater level drawdown.

9. CONCLUSIONS

9.1. Geology

Area E is located on Karoo age deposits, which blanket dolomite rock at intermediate to great depth. In the past clays were quarried from this site. These areas have apparently been backfilled.

9.2. Geohydrology

The shallow groundwater levels reported in the 2011 and 2013 boreholes may be perched groundwater. The deeper groundwater level recorded in Borehole 7/24 (45m deep) may represent the dolomite groundwater level.

The dolomite groundwater level was 51m deep in 2005 in the nearest Department of Water and Sanitation dolomite aquifer monitoring station. The current groundwater level is 29m deep. The mean groundwater level elevation is 1469m AMSL according to the Water Research Commission (2014).

Acknowledging the significant difference between the current groundwater level depth at Monitoring Station C2N1114 on the farm Modderfontein and Borehole 7/24 (drilled in 2012), monitoring of the site specific groundwater level (of the dolomite aquifer) is recommended.

9.3. Dolomite Hazard Assessment

Based on available data and current best practice, Area E represents a **low to medium** susceptibility of sinkhole and subsidence formation with respect to ingress of water and a **low** susceptibility of all-size sinkhole and subsidence formation with respect to anthropogenic groundwater level drawdown, i.e. Inherent Hazar Class 1/2//1.

9.4. Design Hazard Class

The Design Hazard Class is 2. In light of the land use, the Dolomite Area Designation is D3.

10. RECOMMENDATIONS

10.1. Permissible Land Use SANS 1936 Part 1 (2012)

Development normally results in a disturbance of the meta-stable conditions in the dolomite environment. Consequently, factors such as the basic design of services, construction and service installation procedures, and ongoing infrastructure maintenance programmes are key elements in the overall strategy to reduce the probability of generating ground movement events. Risk management consists of key elements such as **implementation of precautionary measures** to prevent the concentrated ingress of water into the ground, **monitoring actions** to detect problems which could lead to ground movement and **maintenance** of infrastructure.

According to SANS 1936 (2012) Part 1, Table 3, landfill is permissible:

Infrastructure and social facilities		**Inherent hazard class determined in accordance with the requirements of SANS 1936-2**							
Designa-tion	**Description**	**1**	**2**	**3**	**4**	**5**	**6**	**7**	**8**
		Dolomite area designation							
IN1	Trunk roads (national and regional roads which facilitate intercity travel) and primary distributor roads (major arterial roads forming the primary network for an urban area as a whole), railway lines, power lines, runways, bulk pipelines, including water, sewer, fuel and gas lines, pump stations	D2	D3					D4	
IN2	Reservoirs and public swimming pools, water care works, attenuation and retention ponds for stormwater management and artificial lakes	D2	D3				D4		
IN3	Cemeteries	D3				D4			
IN4	Dams, slimes dams	D3	D4						
IN5	***Solid waste disposal facilities***	***D3***						D4	

The following conditions were set for support of the development by the Council for Geoscience in May 203:

1) A Dolomite Risk Management Plan must be compiled and implemented for the site since it is underlain by dolomite.

2) The professional team involved shall carefully consider the appropriate water precautionary measures and then ensure and finally certify that these have been implemented.

3) The design of a waste disposal site should ensure the following:
   - 3-1 No polluted water or liquids leak into the underlying soils or rooks.
   - 3-2 All stormwater be diverted away from the site to natural water courses.
   - 3-3 All water precipitated from undeveloped parts of the site be diverted away from the waste body and released into the natural water courses.
   - 3-4 The recommended standard lining system should be installed during the development of the site.
   - 3-5 If any leakage is detected steps must be taken to rectify the situation.

4) The precautionary measures as stipulated in SANS 1936-3:2012 document for waste disposal facilities should be implemented.

10.2. Precautionary Measures (SANS1936-3, 2012)

The site is subject to dolomite precautionary measures and designated as D3.

SANS 1936 (2012) Part 3 indicates the following with respect to waste disposal facilities:

"9.2 Additional precautionary measures on dolomite area designation D3 sites

9.2.1 *Swimming pools and liquid-retaining structures shall be designed so that any leakage that might occur during their lifetime can readily be detected or remedied.*

9.2.2 *Other liquid-retaining structures, including public swimming pools, shall not be placed within 50m of residential buildings and 30 m of non-residential structures.*

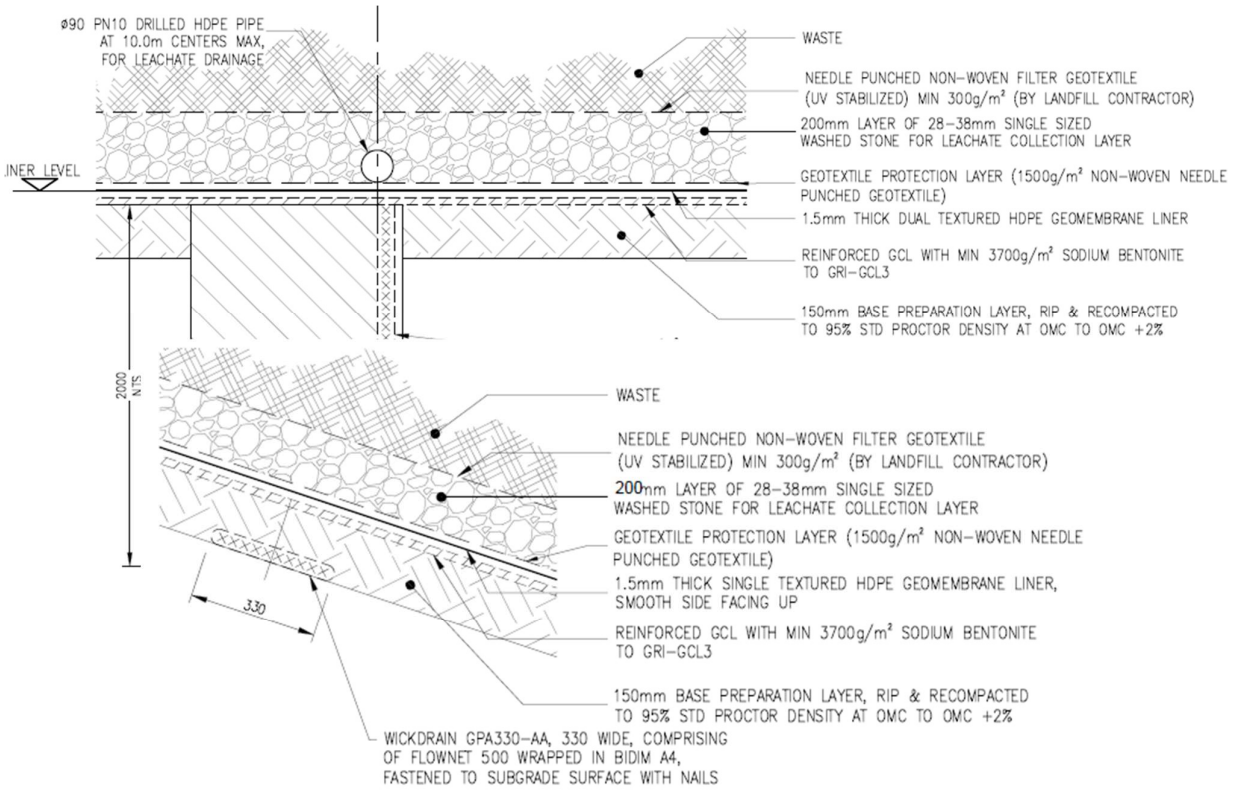
NOTE ***An impervious lining might be necessary*** *to prevent leakage in attenuation and retention ponds for stormwater management, artificial lakes, slimes dams and waste disposal facilities"*

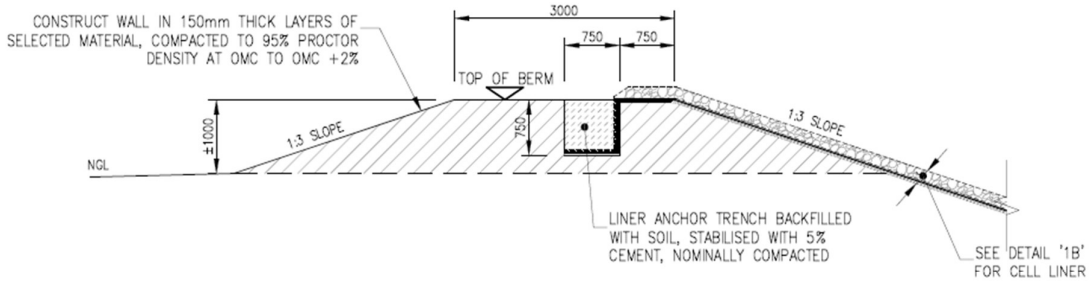
In addition, the standard calls for stormwater management, as a point of departure. This has been addressed and should be reassessed after the first rains of the rainy season as part of dolomite risk management.

The site-specific depth to and elevation of the dolomite groundwater level should be established and monitored. In the event that the dolomite groundwater level is recorded above dolomite bedrock, the need for further investigations should be considered.

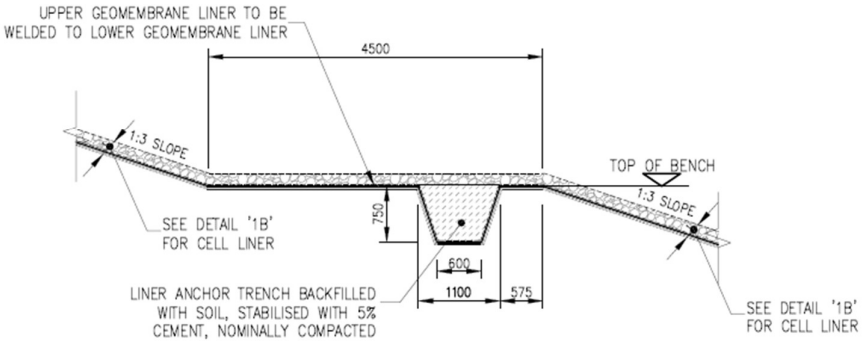
10.3. Design Considerations

The cell base liner design should comply with SANS 1936 (2012) Part 3, Section 9.2.1.

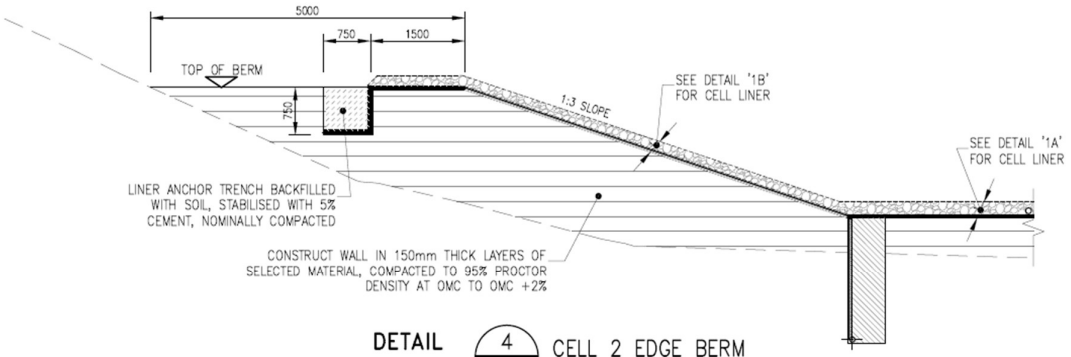




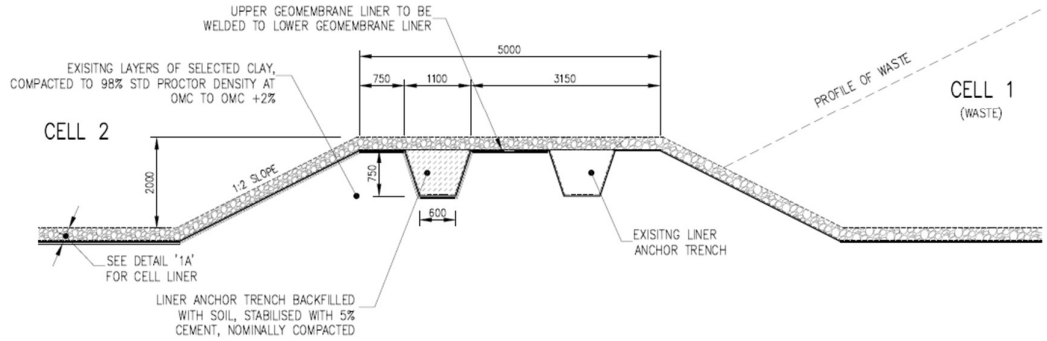
DETAIL 2 — PERIMETER BERM
SCALE 1:50



DETAIL 3 — BENCH
SCALE 1:50



DETAIL 4 — CELL 2 EDGE BERM
SCALE 1:50



DETAIL 5 — CELL DIVISION BERM
SCALE 1:50



Figure 6: Area E, Cells 1 and 2 in relation to boreholes drilled

10.4. Dolomite Risk Management

It is recommended that the owner of the Facility ensure that monitoring from a dolomite risk management perspective takes place.

The results of the visual inspection should be captured in a report.

In the event that any ground cracks develop, or stormwater ponds and does not effectively flow into the larger stormwater system, this should be appropriately addressed. The inspection should include date, time, person responsible and photographs, even if no problems are identified/observed.

10.5. Future Extensions

Refer to Figure 7. According to existing reports, the backfill area within Area E is delineated. The nature of the backfill operation and extent to which material was compacted is not known to this author. Development of waste cells in the future in this area should consider the risks, including differential movement and infiltration of concentrated water to the deeper dolomite aquifer and potential for subsurface erosion. Prior to approval of any expansion over areas of backfill, it should be determined if leaks from the cell could negatively impact dolomite stability and the like.

Areas considered for storage of water (relate to landfill operations) are to meet the requirements of SANS 1936 Part 2 (2012) and may require footprint specific dolomite stability assessment.

11. GENERAL

11.1. Verification Process

The findings in this report are based upon our interpretation of existing data. While every effort has been made, within the limits of the project budget, time and present-day insight, to determine overall ground conditions on this site, poorer sub-areas may have been missed. For this reason, it is recommended that a competent specialist is always invited to inspect open works on this site in order to further confirm the findings described in this report.

11.2. Third Party Reviews

If this report is reviewed by third parties, it should be noted that dolomite hazard assessment and management is an empirical science and consequently such parties may formulate opinions concerning the findings, conclusions and recommendations of this investigation at variance with those contained in this report. This fact should be borne in mind when costs are incurred on this project prior to receipt of the comments of these parties. IntraSolutions, its members and personnel cannot be held responsible for these divergent views or consequential damages.

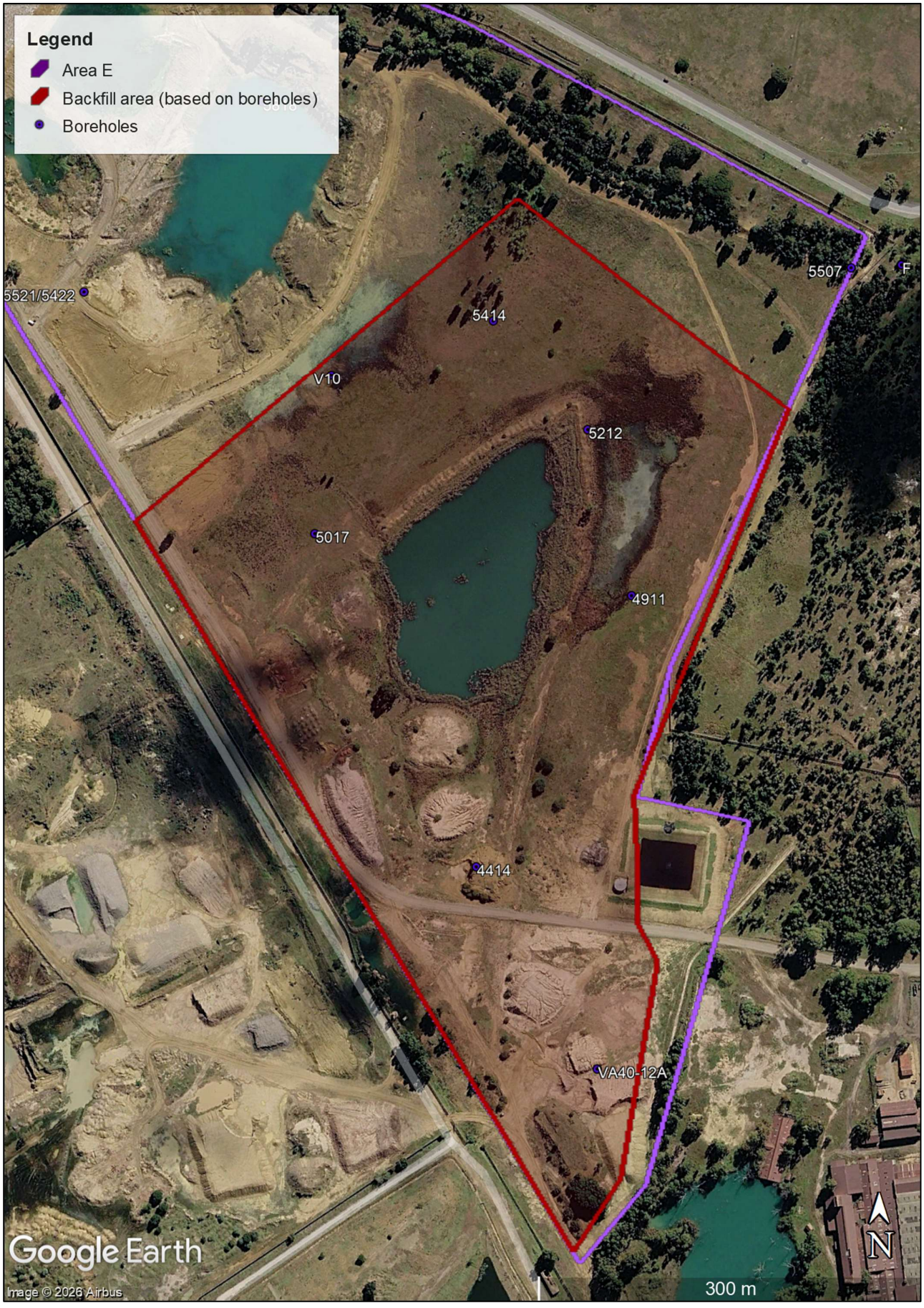


Figure 7: Backfill area (southern portion of Area E)

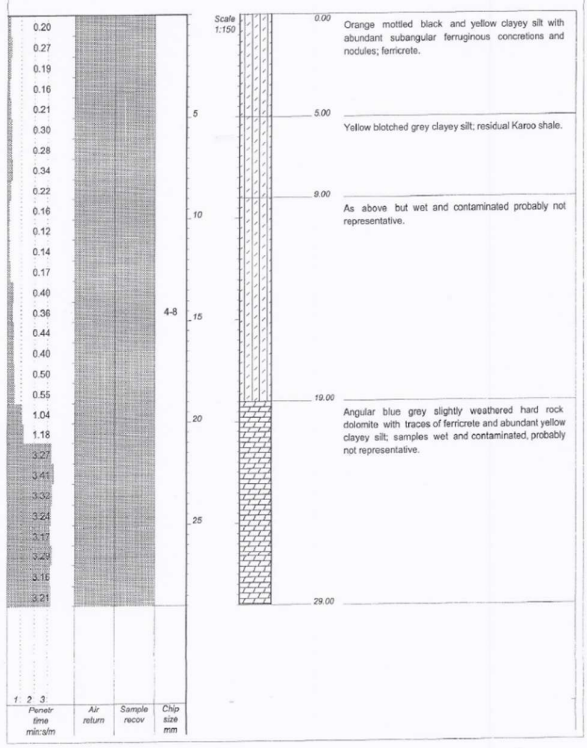
APPENDICES

BOREHOLE LOGS APPENDIX 1

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 4414
Sheet 1 of 2

JOB NUMBER: 2011/02



0.00 Orange mottled black and yellow clayey silt with abundant subangular ferruginous concretions and nodules; ferricrete.

5.00 Yellow blotched grey clayey silt; residual Karoo shale.

9.00 As above but wet and contaminated probably not representative.

19.00 Angular blue grey slightly weathered hard rock dolomite with traces of ferricrete and abundant yellow clayey silt; samples wet and contaminated, probably not representative.

29.00

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 4414
Sheet 2 of 2

JOB NUMBER: 2011/02

NOTES

1) Driller records hard rock dolomite from 21 m with soft rock from 19 m.

1 2 3
Penetr time min:s/m | Air return | Sample recov | Chip size mm

CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho

TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET

INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-15

DATE : 23/06/11 22:13
TEXT : ..C:\DOTFILES\M22HOL~1.TXT

ELEVATION :
X-COORD : -2909792.0175
Y-COORD : -57 388.3760

HOLE No: BH 4414

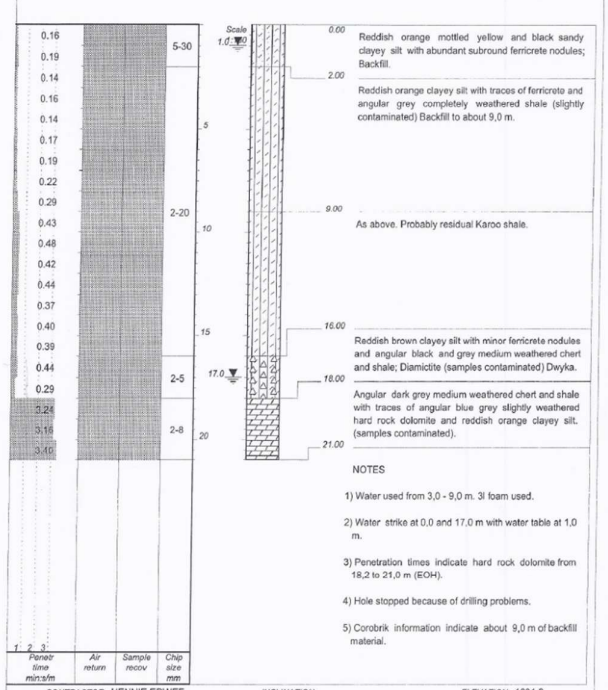
D039 JD GEOTECHNICAL SERVICES CC

dotPLOT 6005 PBp

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 5212
Sheet 1 of 1

JOB NUMBER: 2011/02



0.00 Reddish orange mottled yellow and black sandy clayey silt with abundant subround ferricrete nodules; Backfill.

2.00 Reddish orange clayey silt with traces of ferricrete and angular grey completely weathered shale (slightly contaminated) Backfill to about 9,0 m.

9.00 As above. Probably residual Karoo shale.

16.00 Reddish brown clayey silt with minor ferricrete nodules and angular black and grey medium weathered chert and shale; Diamictite (samples contaminated) Dwyka.

18.00 Angular dark grey medium weathered chert and shale with traces of angular blue grey slightly weathered hard rock dolomite and reddish orange clayey silt. (samples contaminated).

21.00

NOTES

1) Water used from 3,0 - 9,0 m. 3l foam used.

2) Water strike at 0,0 and 17,0 m with water table at 1,0 m.

3) Penetration times indicate hard rock dolomite from 18,2 to 21,0 m (EOH).

4) Hole stopped because of drilling problems.

5) Corobrik information indicate about 9,0 m of backfill material.

CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho

TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET

INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-16

DATE : 21/07/11 15:32
TEXT : ..C:\DOTFILES\22HOLE~2.TXT

ELEVATION : 1601.8
X-COORD : -2909452.2307
Y-COORD : -57 302.8742

HOLE No: BH 5212

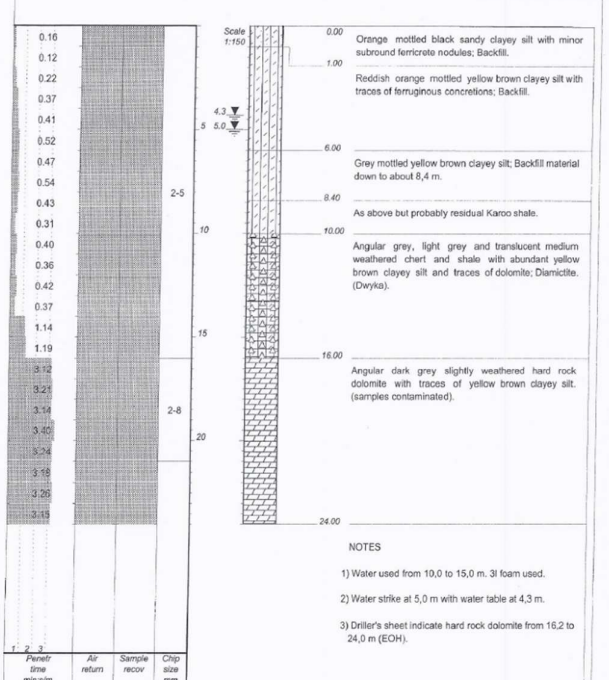
D039 JD GEOTECHNICAL SERVICES CC

dotPLOT 6005 PBp

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 5414
Sheet 1 of 1

JOB NUMBER: 2011/02



0.00 Orange mottled black sandy clayey silt with minor subround ferricrete nodules; Backfill.

1.00 Reddish orange mottled yellow brown clayey silt with traces of ferruginous concretions; Backfill.

6.00 Grey mottled yellow brown clayey silt; Backfill material down to about 8,4 m.

8.40 As above but probably residual Karoo shale.

10.00 Angular grey, light grey and translucent medium weathered chert and shale with abundant yellow brown clayey silt and traces of dolomite; Diamictite. (Dwyka).

16.00 Angular dark grey slightly weathered hard rock dolomite with traces of yellow brown clayey silt. (samples contaminated).

24.00

NOTES

1) Water used from 10,0 to 15,0 m. 3l foam used.

2) Water strike at 5,0 m with water table at 4,3 m.

3) Driller's sheet indicate hard rock dolomite from 16,2 to 24,0 m (EOH).

CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho

TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET

INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-20

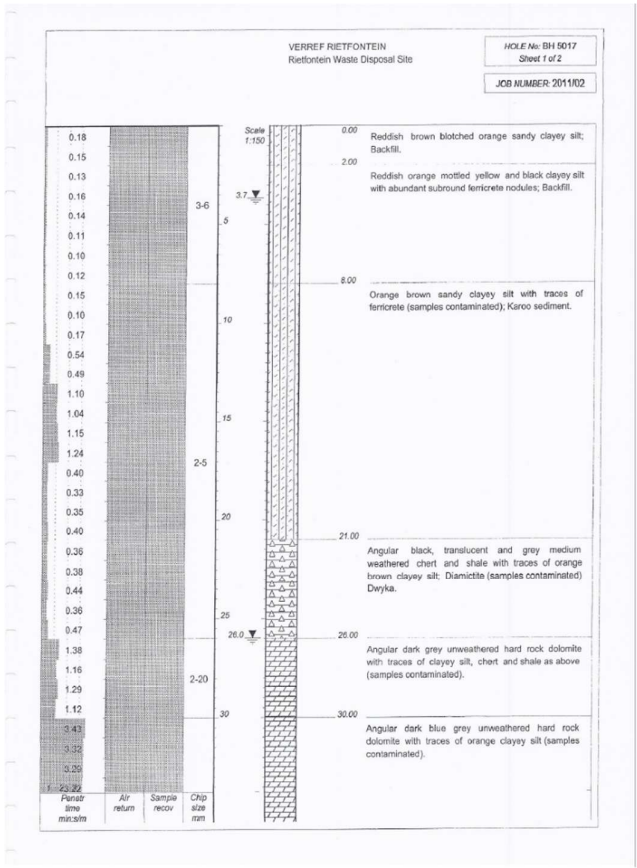
DATE : 21/07/11 15:33
TEXT : ..C:\DOTFILES\22HOLE~2.TXT

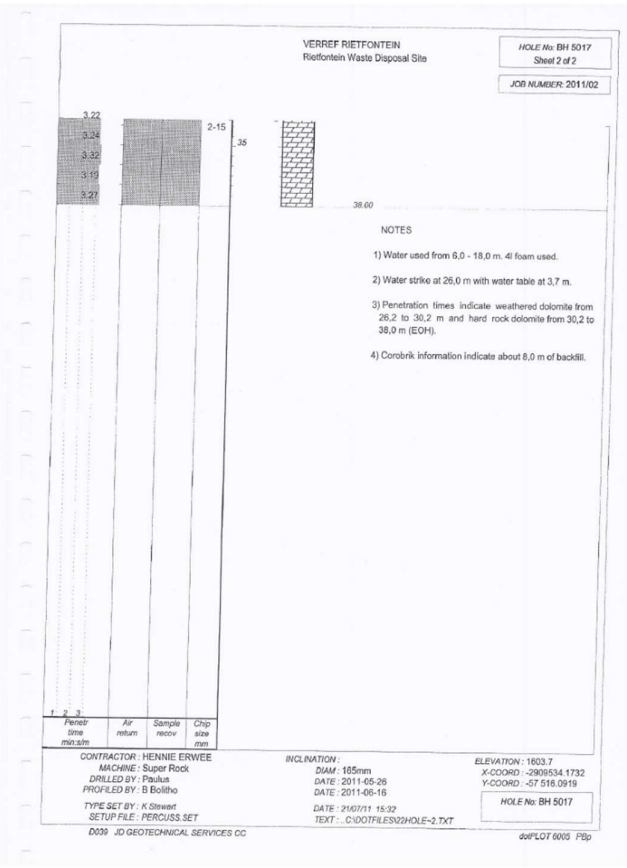
ELEVATION : 1601.5
X-COORD : -2909367.9691
Y-COORD : -57 377.0560

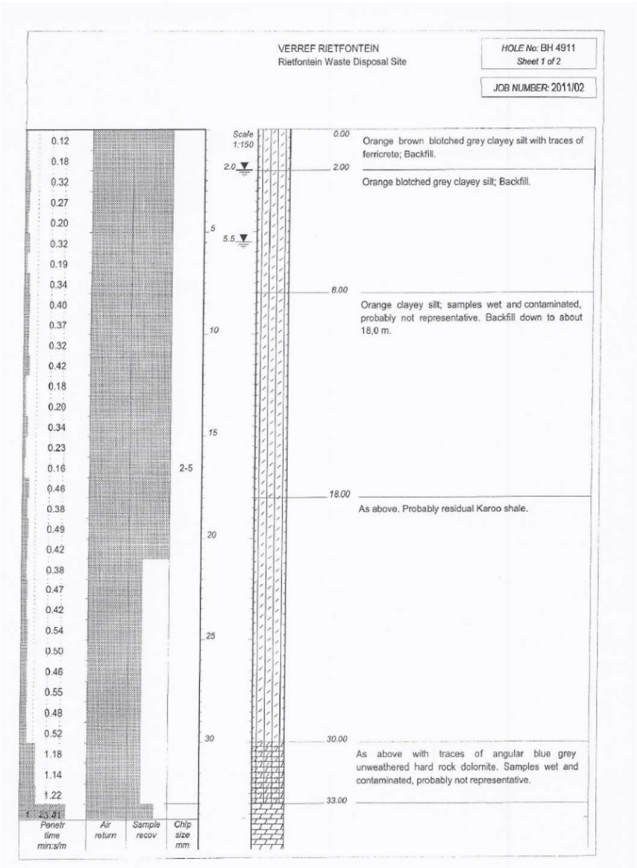
HOLE No: BH 5414

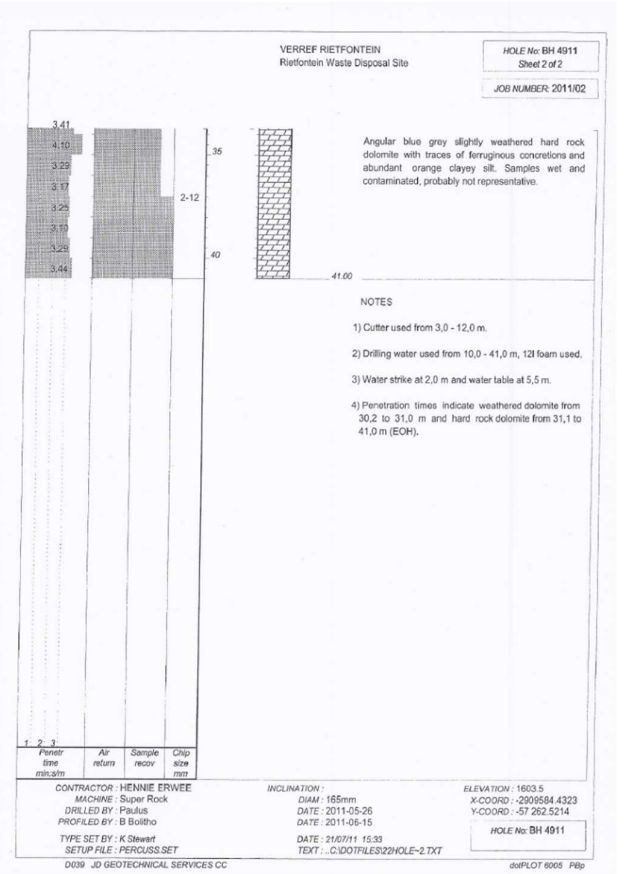
D039 JD GEOTECHNICAL SERVICES CC

dotPLOT 6005 PBp


VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 5017
Sheet 1 of 2
JOB NUMBER: 2011/02
Scale 1:150
0.00 Reddish brown blotched orange sandy clayey silt; Backfill.
2.00 Reddish orange mottled yellow and black clayey silt with abundant subround ferricrete nodules; Backfill.
3.7
8.00 Orange brown sandy clayey silt with traces of ferricrete (samples contaminated); Karoo sediment.
21.00 Angular black, translucent and grey medium weathered chert and shale with traces of orange brown clayey silt; Diamictite (samples contaminated) Dwyka.
26.0
26.00 Angular dark grey unweathered hard rock dolomite with traces of clayey silt, chert and shale as above (samples contaminated).
30.00 Angular dark blue grey unweathered hard rock dolomite with traces of orange clayey silt (samples contaminated).
3-6
2-5
2-20
Penetr time min:s/m
Air return
Sample recov
Chip size mm


VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 5017
Sheet 2 of 2
JOB NUMBER: 2011/02
2-15
38.00
NOTES
1) Water used from 6,0 - 18,0 m. 4l foam used.
2) Water strike at 26,0 m with water table at 3,7 m.
3) Penetration times indicate weathered dolomite from 26,2 to 30,2 m and hard rock dolomite from 30,2 to 38,0 m (EOH).
4) Corobrik information indicate about 8,0 m of backfill.
CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho
TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET
INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-16
DATE : 21/07/11 15:32
TEXT : ..C:\DOTFILES\22HOLE~2.TXT
ELEVATION : 1603.7
X-COORD : -2909534.1732
Y-COORD : -57 516.0919
HOLE No: BH 5017
D039 JD GEOTECHNICAL SERVICES CC
dotPLOT 6005 PBp

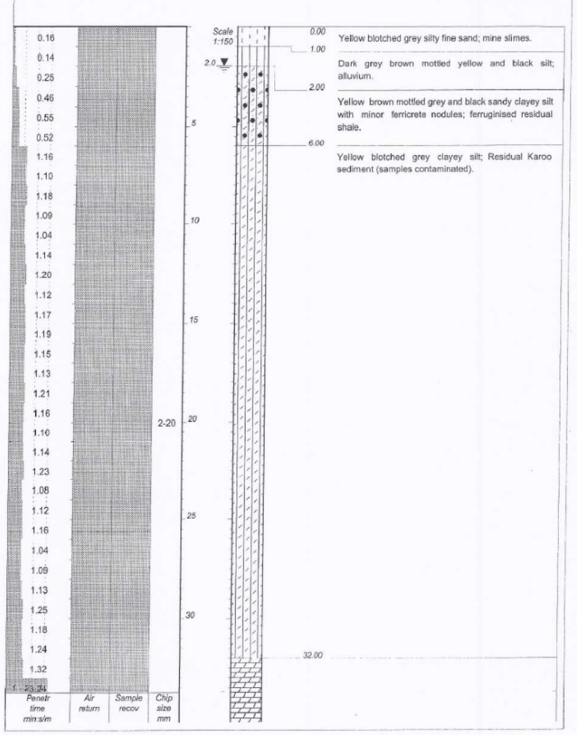

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 4911
Sheet 1 of 2
JOB NUMBER: 2011/02
Scale 1:150
0.00 Orange brown blotched grey clayey silt with traces of ferricrete; Backfill.
2.0
2.00 Orange blotched grey clayey silt; Backfill.
5.5
8.00 Orange clayey silt; samples wet and contaminated, probably not representative. Backfill down to about 18,0 m.
18.00 As above. Probably residual Karoo shale.
30.00 As above with traces of angular blue grey unweathered hard rock dolomite. Samples wet and contaminated, probably not representative.
33.00
2-5
Penetr time min:s/m
Air return
Sample recov
Chip size mm


VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 4911
Sheet 2 of 2
JOB NUMBER: 2011/02
2-12
41.00
Angular blue grey slightly weathered hard rock dolomite with traces of ferruginous concretions and abundant orange clayey silt. Samples wet and contaminated, probably not representative.
NOTES
1) Cutter used from 3,0 - 12,0 m.
2) Drilling water used from 10,0 - 41,0 m, 12l foam used.
3) Water strike at 2,0 m and water table at 5,5 m.
4) Penetration times indicate weathered dolomite from 30,2 to 31,0 m and hard rock dolomite from 31,1 to 41,0 m (EOH).
CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho
TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET
INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-15
DATE : 21/07/11 15:33
TEXT : ..C:\DOTFILES\22HOLE~2.TXT
ELEVATION : 1603.5
X-COORD : -2909584.4323
Y-COORD : -57 262.5214
HOLE No: BH 4911
D039 JD GEOTECHNICAL SERVICES CC
dotPLOT 6005 PBp

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 5507
Sheet 1 of 2

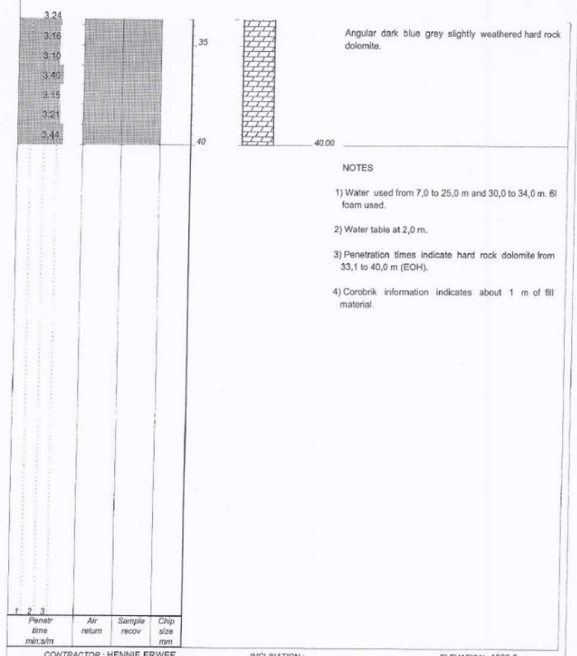
JOB NUMBER: 2011/02



VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 5507
Sheet 2 of 2

JOB NUMBER: 2011/02



CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho
TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET

INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-16
DATE : 21/07/11 15:33
TEXT : ..C:\DOTFILES\22HOLE~2.TXT

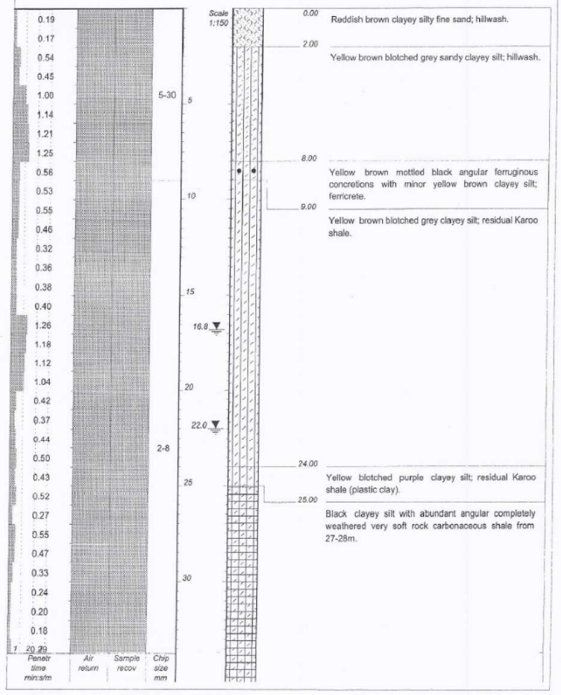
ELEVATION : 1606.0
X-COORD : -2909325.5842
Y-COORD : -57 097.3706
HOLE No: BH 5507

D039 JD GEOTECHNICAL SERVICES CC
dotPLOT 6005 PBp

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 5818
Sheet 1 of 2

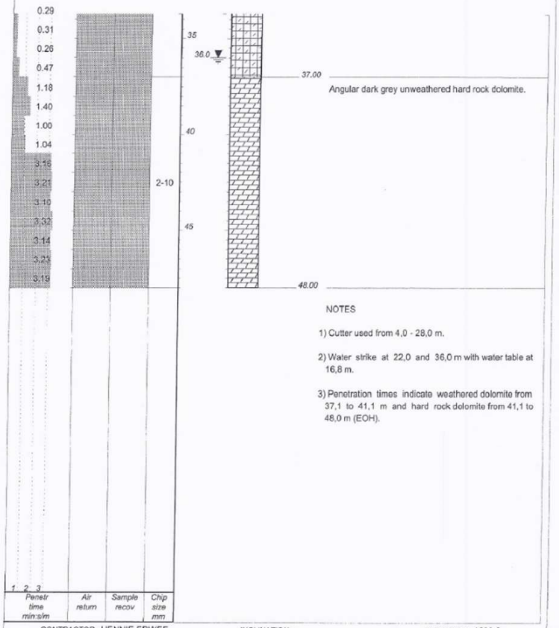
JOB NUMBER: 2011/02



VERREF RIETFONTEIN
Rietfontein Waste Disposal Site

HOLE No: BH 5818
Sheet 2 of 2

JOB NUMBER: 2011/02

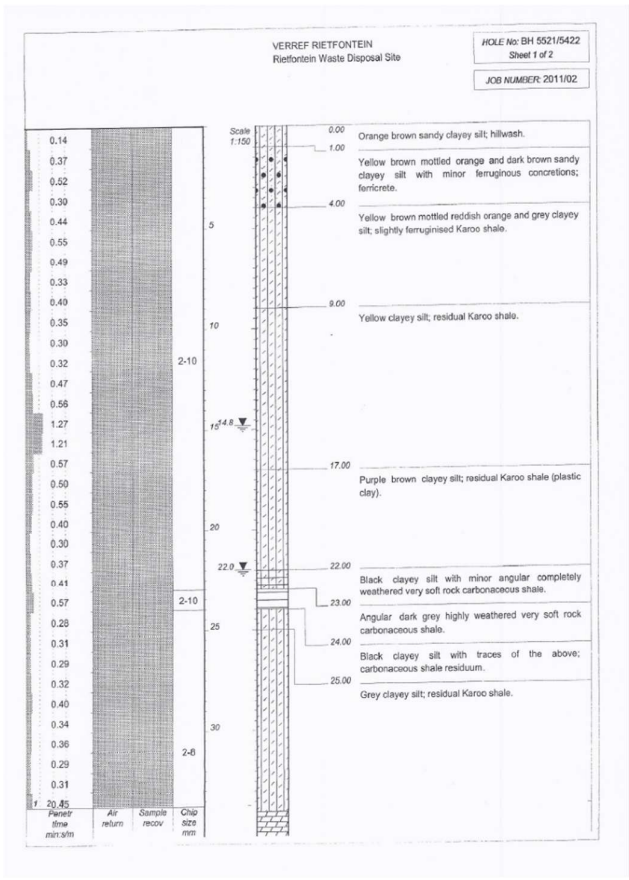


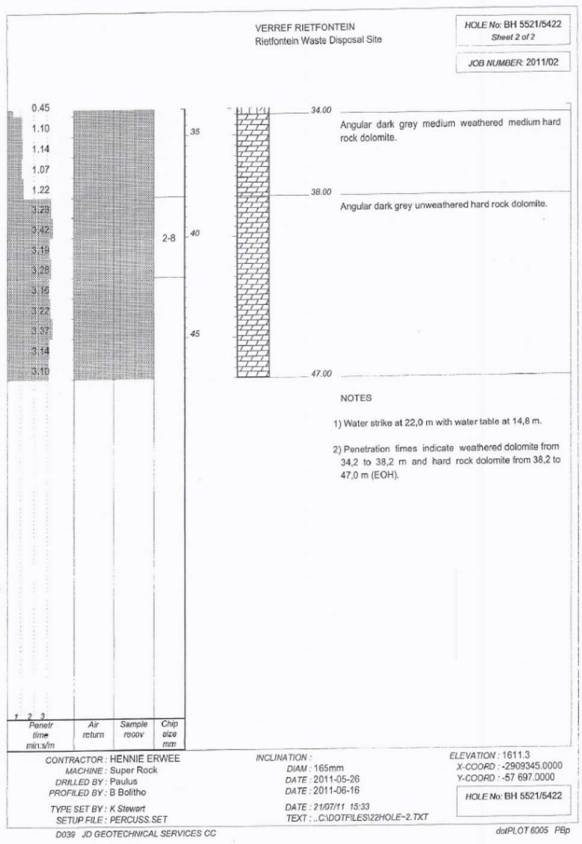
CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho
TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET

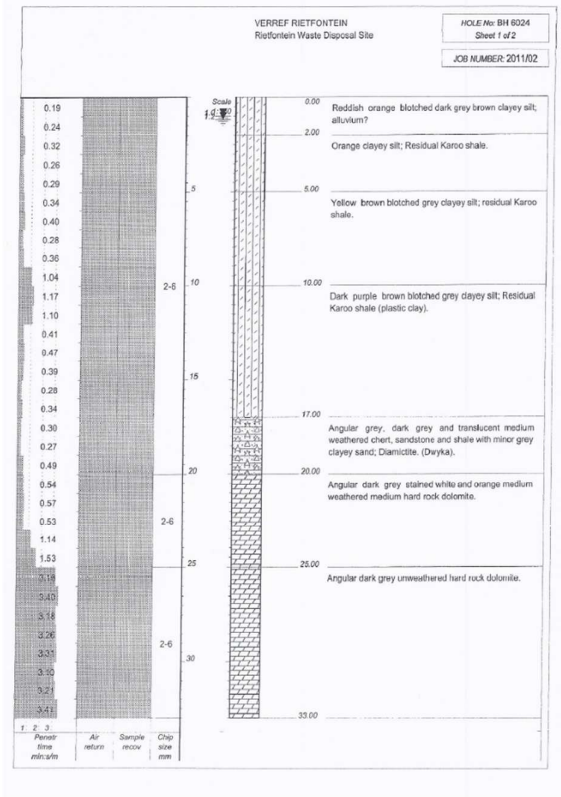
INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-16
DATE : 21/07/11 15:33
TEXT : ..C:\DOTFILES\22HOLE~2.TXT

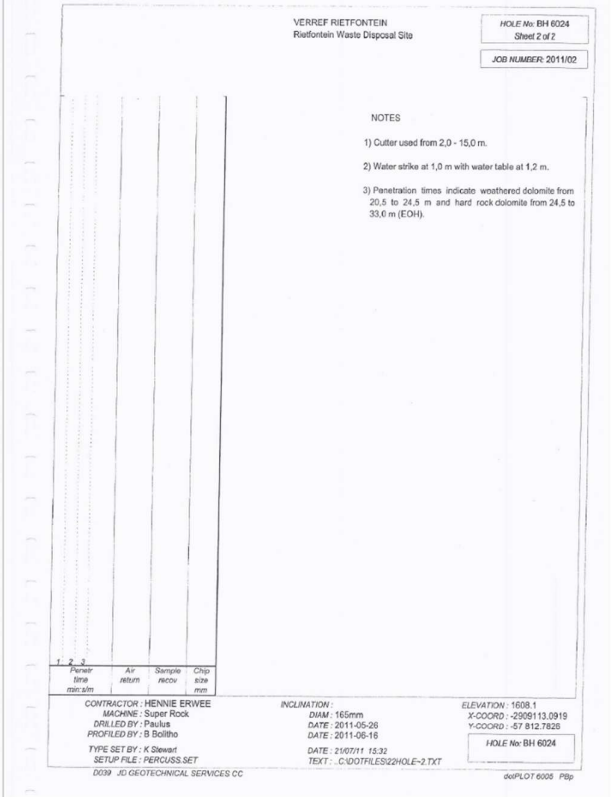
ELEVATION : 1609.6
X-COORD : -2909199.6336
Y-COORD : -57 559.2589
HOLE No: BH 5818

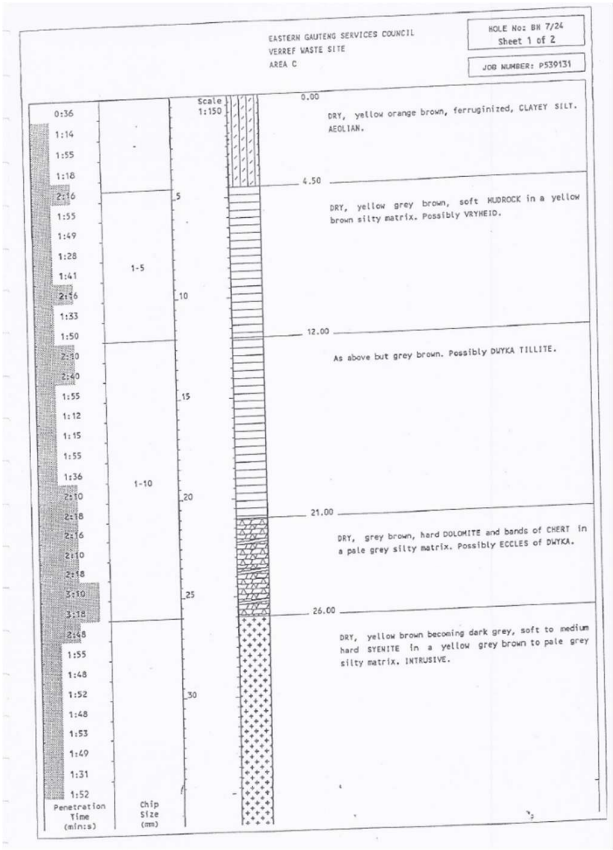
D039 JD GEOTECHNICAL SERVICES CC
dotPLOT 6005 PBp

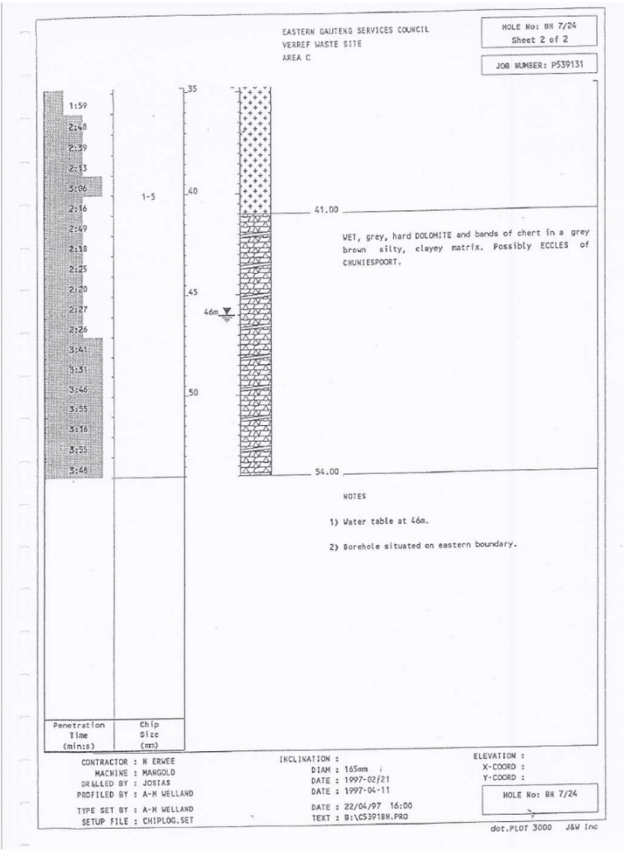

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 5521/5422
Sheet 1 of 2
JOB NUMBER: 2011/02
Scale 1:150
0.00 Orange brown sandy clayey silt; hillwash.
1.00 Yellow brown mottled orange and dark brown sandy clayey silt with minor ferruginous concretions; ferricrete.
4.00 Yellow brown mottled reddish orange and grey clayey silt; slightly ferruginised Karoo shale.
9.00 Yellow clayey silt; residual Karoo shale.
14.8
17.00 Purple brown clayey silt; residual Karoo shale (plastic clay).
22.0
22.00 Black clayey silt with minor angular completely weathered very soft rock carbonaceous shale.
23.00 Angular dark grey highly weathered very soft rock carbonaceous shale.
24.00 Black clayey silt with traces of the above; carbonaceous shale residuum.
25.00 Grey clayey silt; residual Karoo shale.
2-10
2-10
2-6
Penetr time min:s/m
Air return
Sample recov
Chip size mm

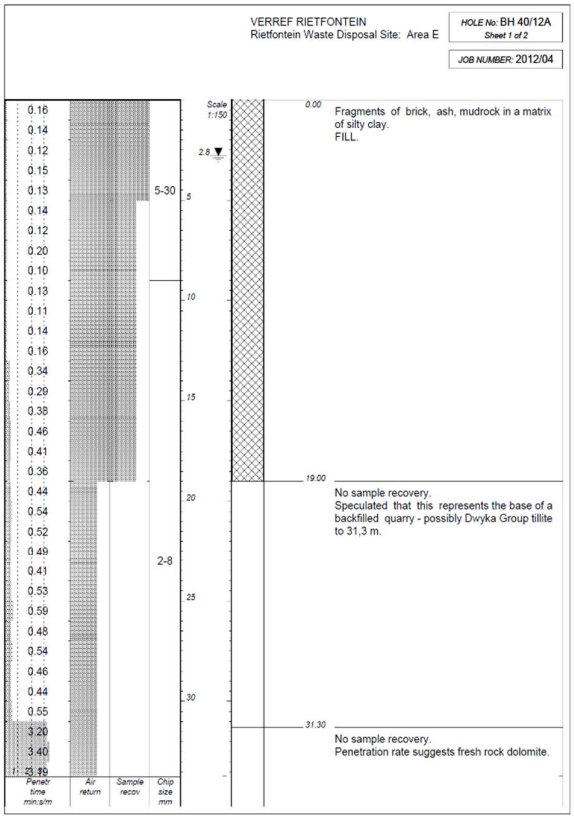

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 5521/5422
Sheet 2 of 2
JOB NUMBER: 2011/02
34.00 Angular dark grey medium weathered medium hard rock dolomite.
38.00 Angular dark grey unweathered hard rock dolomite.
47.00
2-8
NOTES
1) Water strike at 22,0 m with water table at 14,8 m.
2) Penetration times indicate weathered dolomite from 34,2 to 38,2 m and hard rock dolomite from 38,2 to 47,0 m (EOH).
CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho
TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET
INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-16
DATE : 21/07/11 15:33
TEXT : ..C:\DOTFILES\22HOLE~2.TXT
ELEVATION : 1611.3
X-COORD : -2909345.0000
Y-COORD : -57 697.0000
HOLE No: BH 5521/5422
D039 JD GEOTECHNICAL SERVICES CC
dotPLOT 6005 PBp

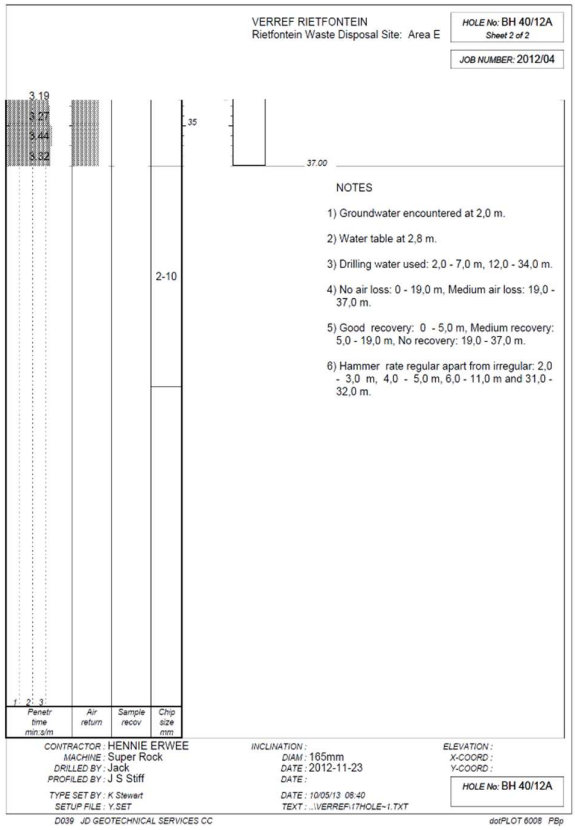

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 6024
Sheet 1 of 2
JOB NUMBER: 2011/02
Scale 1:150
0.00 Reddish orange blotched dark grey brown clayey silt; alluvium?
2.00 Orange clayey silt; Residual Karoo shale.
5.00 Yellow brown blotched grey clayey silt; residual Karoo shale.
10.00 Dark purple brown blotched grey clayey silt; Residual Karoo shale (plastic clay).
17.00 Angular grey, dark grey and translucent medium weathered chert, sandstone and shale with minor grey clayey sand; Diamictite. (Dwyka).
20.00 Angular dark grey stained white and orange medium weathered medium hard rock dolomite.
25.00 Angular dark grey unweathered hard rock dolomite.
33.00
2-6
2-6
2-6
Penetr time min:s/m
Air return
Sample recov
Chip size mm


VERREF RIETFONTEIN
Rietfontein Waste Disposal Site
HOLE No: BH 6024
Sheet 2 of 2
JOB NUMBER: 2011/02
NOTES
1) Cutter used from 2,0 - 15,0 m.
2) Water strike at 1,0 m with water table at 1,2 m.
3) Penetration times indicate weathered dolomite from 20,5 to 24,5 m and hard rock dolomite from 24,5 to 33,0 m (EOH).
CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Paulus
PROFILED BY : B Bolitho
TYPE SET BY : K Stewart
SETUP FILE : PERCUSS.SET
INCLINATION :
DIAM : 165mm
DATE : 2011-05-26
DATE : 2011-06-16
DATE : 21/07/11 15:32
TEXT : ..C:\DOTFILES\22HOLE~2.TXT
ELEVATION : 1608.1
X-COORD : -2909113.0919
Y-COORD : -57 812.7826
HOLE No: BH 6024
D039 JD GEOTECHNICAL SERVICES CC
dotPLOT 6006 PBp


EASTERN GAUTENG SERVICES COUNCIL
VERREF WASTE SITE
AREA C
HOLE No: BH 7/24
Sheet 1 of 2
JOB NUMBER: P539131
Scale 1:150
0.00
DRY, yellow orange brown, ferruginized, CLAYEY SILT. AEOLIAN.
4.50
DRY, yellow grey brown, soft MUDROCK in a yellow brown silty matrix. Possibly VRYHEID.
12.00
As above but grey brown. Possibly DWYKA TILLITE.
21.00
DRY, grey brown, hard DOLOMITE and bands of CHERT in a pale grey silty matrix. Possibly ECCLES of DWYKA.
26.00
DRY, yellow brown becoming dark grey, soft to medium hard SYENITE in a yellow grey brown to pale grey silty matrix. INTRUSIVE.
1-5
1-10
Penetration Time (mins)
Chip Size (mm)


EASTERN GAUTENG SERVICES COUNCIL
VERREF WASTE SITE
AREA C
HOLE No: BH 7/24
Sheet 2 of 2
JOB NUMBER: P539131
41.00
WET, grey, hard DOLOMITE and bands of chert in a grey brown silty, clayey matrix. Possibly ECCLES of CHUNIESPOORT.
46m
54.00
NOTES
1) Water table at 46m.
2) Borehole situated on eastern boundary.
1-5
Penetration Time (mins)
Chip Size (mm)
CONTRACTOR : H ERWEE
MACHINE : MANGOLD
DRILLED BY : JOHAN
PROFILED BY : A-H WELLAND
TYPE SET BY : A-H WELLAND
SETUP FILE : CHIPLOG.SET
INCLINATION :
DIAM : 165mm
DATE : 1997-02/21
DATE : 1997-04-11
DATE : 22/04/97 16:00
TEXT : B:\C53918H.PRO
ELEVATION :
X-COORD :
Y-COORD :
HOLE No: BH 7/24
dot.PLOT 3000 J&W Inc


VERREF RIETFONTEIN
Rietfontein Waste Disposal Site: Area E
HOLE No: BH 40/12A
Sheet 1 of 2
JOB NUMBER: 2012/04
Scale 1:150
0.00
Fragments of brick, ash, mudrock in a matrix of silty clay.
FILL.
2.8
19.00
No sample recovery.
Speculated that this represents the base of a backfilled quarry - possibly Dwyka Group tillite to 31,3 m.
31.30
No sample recovery.
Penetration rate suggests fresh rock dolomite.
5-30
2-8
Penetr time min:s/m
Air return
Sample recov
Chip size mm


VERREF RIETFONTEIN
Rietfontein Waste Disposal Site: Area E
HOLE No: BH 40/12A
Sheet 2 of 2
JOB NUMBER: 2012/04
37.00
NOTES
1) Groundwater encountered at 2,0 m.
2) Water table at 2,8 m.
3) Drilling water used: 2,0 - 7,0 m, 12,0 - 34,0 m.
4) No air loss: 0 - 19,0 m, Medium air loss: 19,0 - 37,0 m.
5) Good recovery: 0 - 5,0 m, Medium recovery: 5,0 - 19,0 m, No recovery: 19,0 - 37,0 m.
6) Hammer rate regular apart from irregular: 2,0 - 3,0 m, 4,0 - 5,0 m, 6,0 - 11,0 m and 31,0 - 32,0 m.
2-10
Penetr time min:s/m
Air return
Sample recov
Chip size mm
CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Jack
PROFILED BY : J S Stiff
TYPE SET BY : K Stewart
SETUP FILE : Y.SET
INCLINATION :
DIAM : 165mm
DATE : 2012-11-23
DATE :
DATE : 10/05/13 08:40
TEXT : ..\VERREF\17HOLE~1.TXT
ELEVATION :
X-COORD :
Y-COORD :
HOLE No: BH 40/12A
D039 JD GEOTECHNICAL SERVICES CC
dotPLOT 6008 PBp

H. WEBER & ASSOCIATES INC.
CONSULTING GEOTECHNICAL ENGINEERS

PERCUSSION BOREHOLE LOG

Client: RAND SCRAP IRON & METAL COMPANY

Borehole Number: F

Project: VLAKFONTEIN (GIFGAT) SOLID WASTE SITE.

Project Number:

Depth Metres	Legend	Description	Penetration Rate (Minutes) 2 4 6 8	Air/ Sample Return
		Yellow, becoming grey, silty fine SAND. Mine slimes.		
3,0		Yellow brown, clayey SILT with ferruginised concretions. Hillwash.		
5,0		Yellowish orange, mottled yellow, CLAY-SILT Residual Karoo shale.		
▽6,0		As above, but brown, mottled yellow.		
7,0		Light brown, blotched grey, CLAY-SILT. Residual Karoo shale.		
13,0		Grey, blotched yellow, CLAY-SILT. Karoo shale.		
15,0		Pale pink, blotched dark brown, CLAY-SILT Karoo shale.		
19,0		Hole stopped - not requested. Not cased.		

Contractor: JACK BENNETT	Ground Level: /
Machine and Hammer: ROCK GIANT/150mm	Logged By: B BOLITHO
Date Drilled: 3-9-94	Date Logged: 3-9-94

Remarks
Air/Sample Return: G - Good, M — Moderate, P — Poor, N — No

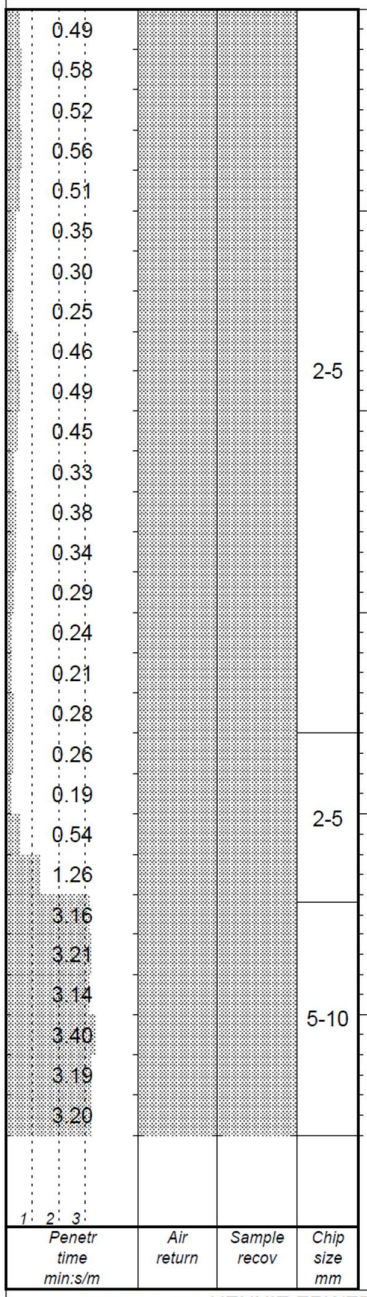
Compression 17 Bar Pressure
(1700 kPa)

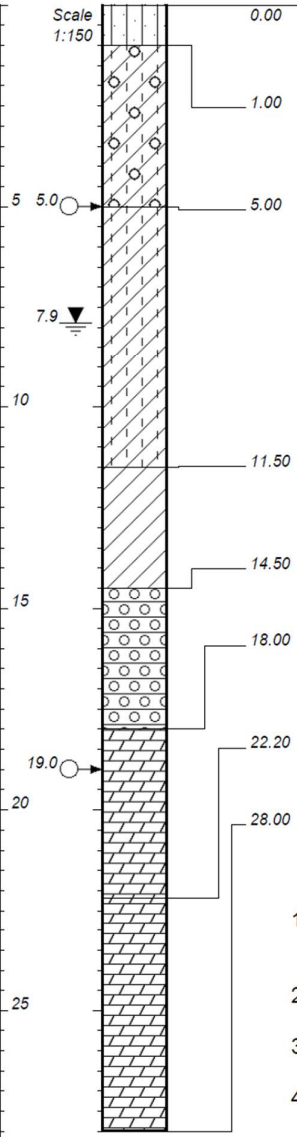
ater level at 6,0m on 5-9-94 and 15-9-94

VERREF RIETFONTEIN
Rietfontein Waste Disposal Site: Area G

HOLE No: BH 61/21
Sheet 1 of 1

JOB NUMBER: 2012/04





0.00	Moist, yellow orange, sandy SILT. Topsoil and Aeolian. Transported.
1.00	Moist, orange brown and black, gravelly silty CLAY with ferricrete fragments. Ferrugenised residual Ecca mudrock.
5.00	Moist, yellow and light grey, silty CLAY. Completely to highly weathered Ecca Group shale.
11.50	Moist, purple and yellow brown CLAY. Completely weathered Ecca Group mudrock (plastic clay).
14.50	Grey, medium weathered rock diamictite. Dwyka Group tillite.
18.00	Grey and light grey, fresh dolomite fragments with discolouration on some joint surfaces. Slightly weathered possibly fractured dolomite.
22.20	Dark blue grey fresh rock DOLOMITE. Malmani Subgroup.
28.00	

NOTES

1) Minor water strike (encountered) at 5,0 m. Strong water strike (encountered) at 19,0 m.

2) Water table at 7,9 m.

3) No drilling water used.

4) No air or sample loss.

5) Hammer rate regular apart from irregular: 19,0 - 20,0 m, 22,0 - 23,0 m, Cutter used: 5,0 m - 14,0 m.

CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Jack
PROFILED BY : J S Stiff
TYPE SET BY : K Stewart
SETUP FILE : Y.SET

INCLINATION :
DIAM : 165mm
DATE : 2012-11-23
DATE :
DATE : 10/05/13 08:40
TEXT : ..\VERREF\17HOLE~1.TXT

ELEVATION :
X-COORD :
Y-COORD :

HOLE No: BH 61/21

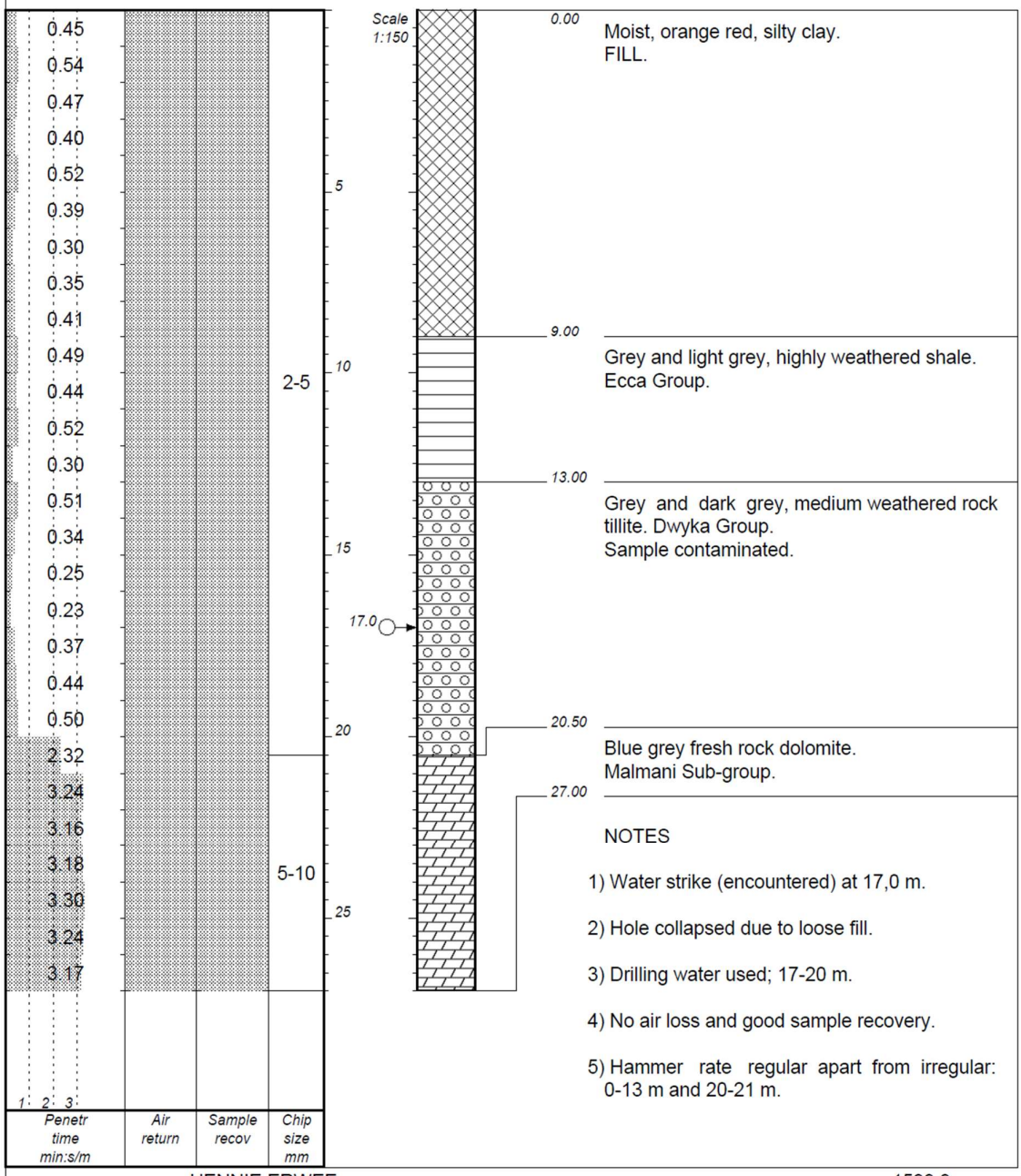
D039 JD GEOTECHNICAL SERVICES CC

dotPLOT 6008 PBp

VERREF RIETFONTEIN
Rietfontein - Area E

HOLE No: BH V10
Sheet 1 of 1

JOB NUMBER: 2013/03



CONTRACTOR : HENNIE ERWEE
MACHINE : Super Rock
DRILLED BY : Jack
PROFILED BY : JS Stiff
TYPE SET BY : K Stewart
SETUP FILE : Y.SET

INCLINATION :
DIAM : 165mm
DATE : 2013-03-19
DATE : 2013-04-30
DATE : 10/05/13 08:31
TEXT : ..\VERREF\V1-V10~1.TXT

ELEVATION : 1599.9
X-COORD : 2909410.1
Y-COORD : 57503.7

HOLE No: BH V10

D039 JD GEOTECHNICAL SERVICES CC

dotPLOT 6008 PBp

APPENDIX C

STABILITY ANALYSIS RESULTS

Slope stability analysis

Input data

Project

Task : Tonkmeter Landfill Cell 1
Description : Cell 1 interim stability
Customer : Tonkmeter Resource Facility
Author : Peter Legg
Date : 2026/09/04
Project number : P077b

Settings

(input for current task)

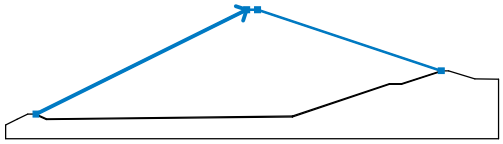
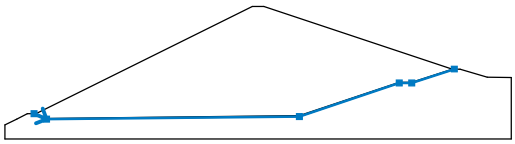
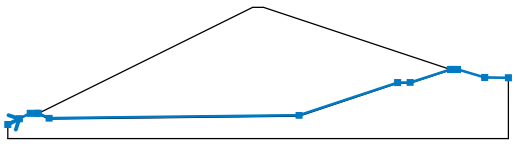
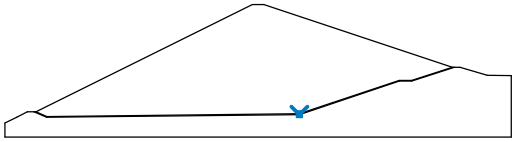
Stability analysis

Verification methodology : Safety factors (ASD)
Earthquake analysis : Standard

Safety factors			
Transient design situation			
Safety factor :	SF_s =	1,30	[–]

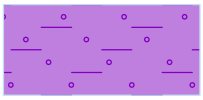
Safety factors			
Seismic design situation			
Safety factor :	SF_s =	1,10	[–]

Interface

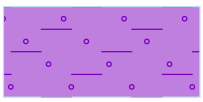
No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		11,01	1,93	88,00	40,00	92,00	40,00
		159,12	17,75				
2		10,36	1,93	14,80	-0,08	104,69	0,90
		140,16	12,80	144,65	12,80	159,72	17,75
3		0,00	-2,09	4,16	0,00	8,01	1,93
		10,36	1,93	11,01	1,93	14,87	0,18
		104,69	1,21	140,16	13,03	144,66	13,05
		159,12	17,75	159,72	17,75	161,76	17,75
		171,43	14,81	180,00	14,72		
4		104,69	1,21	104,69	0,90		

[GEO5 - Slope Stability (32 bit) | version 5.2022.57.0 | hardware key 30127 / 2 | Peter Legg Consulting | Copyright © 2024 Fine spol. s r.o. All Rights Reserved | www.finesoftware.eu]

Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	liner slope		11,00	0,00	18,00
2	waste		30,00	15,00	11,00
3	liner base		21,00	14,00	18,00
4	residual clay		29,00	14,00	18,00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	liner slope		18,00		
2	waste		15,00		
3	liner base		18,00		
4	residual clay		18,00		

Soil parameters

liner slope

Unit weight : γ = 18,00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 11,00 °
Cohesion of soil : c_{ef} = 0,00 kPa
Saturated unit weight : γ_{sat} = 18,00 kN/m^3

waste

Unit weight : γ = 11,00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 30,00 °
Cohesion of soil : c_{ef} = 15,00 kPa
Saturated unit weight : γ_{sat} = 15,00 kN/m^3

liner base

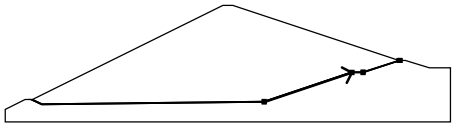
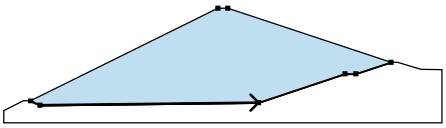
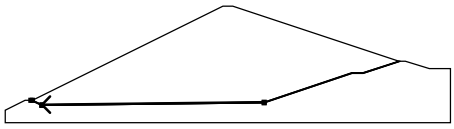
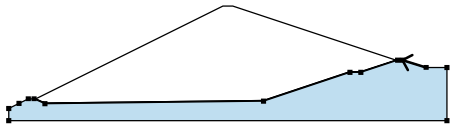
Unit weight : γ = 18,00 kN/m^3
Stress-state : effective

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Angle of internal friction : φ_{ef} = 21,00 °
Cohesion of soil : c_{ef} = 14,00 kPa
Saturated unit weight : γ_{sat} = 18,00 kN/m^3

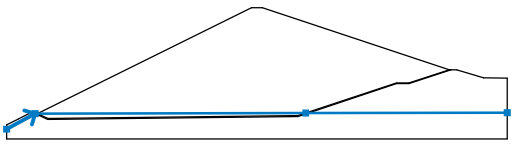
residual clay
Unit weight : γ = 18,00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 29,00 °
Cohesion of soil : c_{ef} = 14,00 kPa
Saturated unit weight : γ_{sat} = 18,00 kN/m^3

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m]				Assigned soil
		x	z	x	z	
1		104,69	0,90	140,16	12,80	liner slope
		144,65	12,80	159,72	17,75	
		159,12	17,75	144,66	13,05	
		140,16	13,03	104,69	1,21	
2		14,87	0,18	104,69	1,21	waste
		140,16	13,03	144,66	13,05	
		159,12	17,75	92,00	40,00	
		88,00	40,00	11,01	1,93	
3		104,69	1,21	14,87	0,18	liner base
		11,01	1,93	10,36	1,93	
		14,80	-0,08	104,69	0,90	
4		171,43	14,81	161,76	17,75	residual clay
		159,72	17,75	144,65	12,80	
		140,16	12,80	104,69	0,90	
		14,80	-0,08	10,36	1,93	
		8,01	1,93	4,16	0,00	
		0,00	-2,09	0,00	-7,09	
		180,00	-7,09	180,00	14,72	

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m]					
		x	z	x	z	x	z
1		0,00	-3,61	10,26	1,93	107,55	2,16
		180,00	2,33				

Tensile crack

Tensile crack not input.

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Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation : transient

Results (Stage of construction 1)

Analysis 1 (stage 1)

Circular slip surface

Slip surface parameters							
Center :	x =	14,68	[m]	Angles :	α_1 =	-8,91	[°]
	z =	90,34	[m]		α_2 =	57,13	[°]
Radius :	R =	93,46	[m]				
Analysis of the slip surface without optimization.							

Slope stability verification (all methods)

Bishop : FS = 1,64 > 1,30 **ACCEPTABLE**
Fellenius / Petterson : FS = 1,55 > 1,30 **ACCEPTABLE**
Spencer : FS = 1,64 > 1,30 **ACCEPTABLE**
Janbu : FS = 1,64 > 1,30 **ACCEPTABLE**
Morgenstern-Price : FS = 1,64 > 1,30 **ACCEPTABLE**

Analysis 2 (stage 1)

Polygonal slip surface

Coordinates of slip surface points [m]									
x	z	x	z	x	z	x	z	x	z
10,69	1,93	10,71	1,92	14,83	0,06	35,47	0,14	97,31	38,24
Analysis of the slip surface without optimization.									

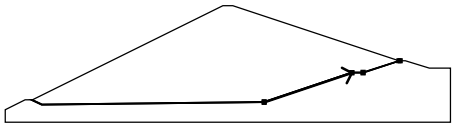
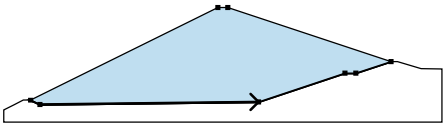
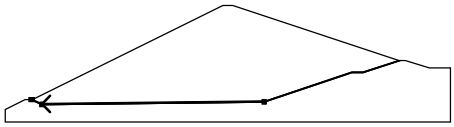
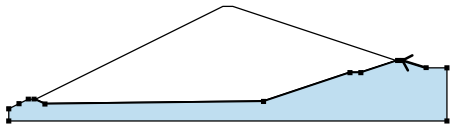
Slope stability verification (all methods)

Sarma : FS = 1,68 > 1,30 **ACCEPTABLE**
Spencer : FS = 1,68 > 1,30 **ACCEPTABLE**
Janbu : FS = 1,66 > 1,30 **ACCEPTABLE**
Morgenstern-Price : FS = 1,66 > 1,30 **ACCEPTABLE**

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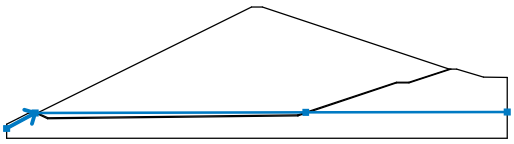
Input data (Stage of construction 2)

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m] x	z	x	z	Assigned soil
1		104,69	0,90	140,16	12,80	liner slope
		144,65	12,80	159,72	17,75	
		159,12	17,75	144,66	13,05	
		140,16	13,03	104,69	1,21	
2		14,87	0,18	104,69	1,21	waste
		140,16	13,03	144,66	13,05	
		159,12	17,75	92,00	40,00	
		88,00	40,00	11,01	1,93	
3		104,69	1,21	14,87	0,18	liner base
		11,01	1,93	10,36	1,93	
		14,80	-0,08	104,69	0,90	
4		171,43	14,81	161,76	17,75	residual clay
		159,72	17,75	144,65	12,80	
		140,16	12,80	104,69	0,90	
		14,80	-0,08	10,36	1,93	
		8,01	1,93	4,16	0,00	
		0,00	-2,09	0,00	-7,09	
		180,00	-7,09	180,00	14,72	

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m] x	z	x	z	x	z
1		0,00	-3,61	10,26	1,93	107,55	2,16
		180,00	2,33				

Tensile crack

Tensile crack not input.

Earthquake

Horizontal seismic coefficient : K_h = 0,1000
Vertical seismic coefficient : K_v = 0,0000

Settings of the stage of construction

Design situation : seismic

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Results (Stage of construction 2)

Analysis 1 (stage 2)

Circular slip surface

Slip surface parameters							
Center :	x =	15,66	[m]	Angles :	α_1 =	-9,88	[°]
	z =	86,82	[m]		α_2 =	58,57	[°]
Radius :	R =	90,15	[m]				
Analysis of the slip surface without optimization.							

Slope stability verification (all methods)

Bishop : FS = 1,29 > 1,10 **ACCEPTABLE**
Fellenius / Petterson : FS = 1,21 > 1,10 **ACCEPTABLE**
Spencer : FS = 1,29 > 1,10 **ACCEPTABLE**
Janbu : FS = 1,29 > 1,10 **ACCEPTABLE**
Morgenstern-Price : FS = 1,29 > 1,10 **ACCEPTABLE**

Analysis 2 (stage 2)

Polygonal slip surface

Coordinates of slip surface points [m]									
x	z	x	z	x	z	x	z	x	z
10,69	1,93	10,71	1,92	14,83	0,06	35,47	0,14	97,31	38,24
Analysis of the slip surface without optimization.									

Slope stability verification (all methods)

Sarma : FS = 1,33 > 1,10 **ACCEPTABLE**
Spencer : FS = 1,32 > 1,10 **ACCEPTABLE**
Janbu : FS = 1,31 > 1,10 **ACCEPTABLE**
Morgenstern-Price : FS = 1,31 > 1,10 **ACCEPTABLE**

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Slope stability analysis

Input data

Project

Task : Tonkmeter Landfill Cell 1
Description : Cell 1 Outer slope stability
Customer : Tonkmeter Resource Facility
Author : Peter Legg
Date : 2026/09/04
Project number : P077b

Settings

(input for current task)

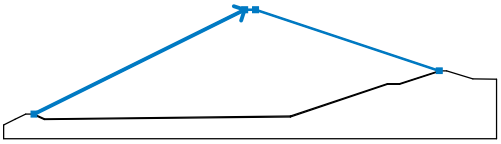
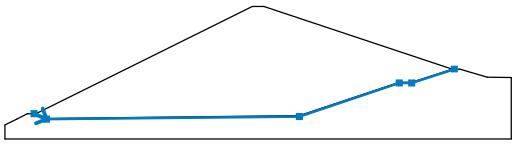
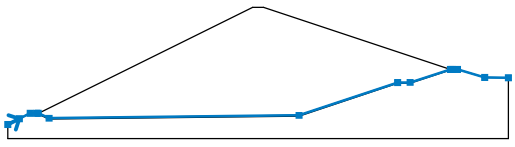
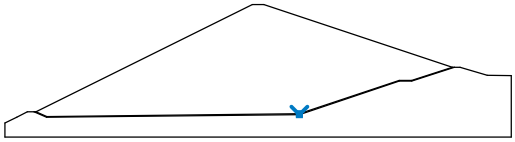
Stability analysis

Verification methodology : Safety factors (ASD)
Earthquake analysis : Standard

Safety factors			
Permanent design situation			
Safety factor :	SF_s =	1,50	[–]

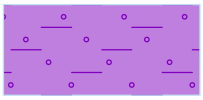
Safety factors			
Seismic design situation			
Safety factor :	SF_s =	1,10	[–]

Interface

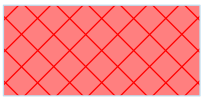
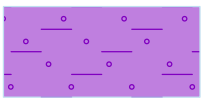
No.	Interface location	Coordinates of interface points [m]					
		x	z	x	z	x	z
1		11,01	1,93	88,00	40,00	92,00	40,00
		159,12	17,75				
2		10,36	1,93	14,80	-0,08	104,69	0,90
		140,16	12,80	144,65	12,80	159,72	17,75
3		0,00	-2,09	4,16	0,00	8,01	1,93
		10,36	1,93	11,01	1,93	14,87	0,18
		104,69	1,21	140,16	13,03	144,66	13,05
		159,12	17,75	159,72	17,75	161,76	17,75
		171,43	14,81	180,00	14,72		
4		104,69	1,21	104,69	0,90		

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Soil parameters - effective stress state

No.	Name	Pattern	φ_{ef} [°]	c_{ef} [kPa]	γ [kN/m^3]
1	liner slope		11,00	0,00	18,00
2	waste		30,00	15,00	11,00
3	liner base		21,00	0,00	18,00
4	residual clay		29,00	14,00	18,00

Soil parameters - uplift

No.	Name	Pattern	γ_{sat} [kN/m^3]	γ_s [kN/m^3]	n [–]
1	liner slope		18,00		
2	waste		15,00		
3	liner base		18,00		
4	residual clay		18,00		

Soil parameters

liner slope

Unit weight : γ = 18,00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 11,00 °
Cohesion of soil : c_{ef} = 0,00 kPa
Saturated unit weight : γ_{sat} = 18,00 kN/m^3

waste

Unit weight : γ = 11,00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 30,00 °
Cohesion of soil : c_{ef} = 15,00 kPa
Saturated unit weight : γ_{sat} = 15,00 kN/m^3

liner base

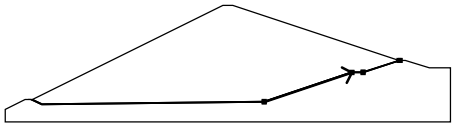
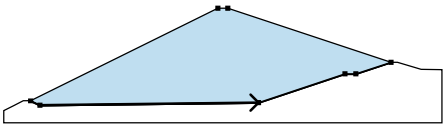
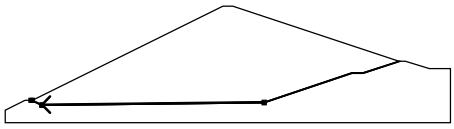
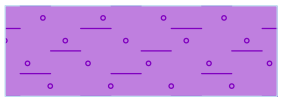
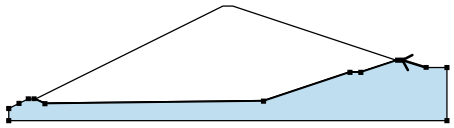
Unit weight : γ = 18,00 kN/m^3
Stress-state : effective

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Angle of internal friction : φ_{ef} = 21,00 °
Cohesion of soil : c_{ef} = 0,00 kPa
Saturated unit weight : γ_{sat} = 18,00 kN/m^3

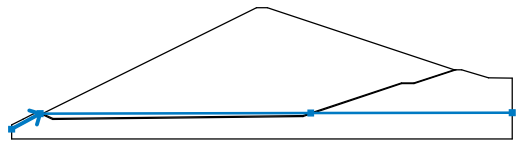
residual clay
Unit weight : γ = 18,00 kN/m^3
Stress-state : effective
Angle of internal friction : φ_{ef} = 29,00 °
Cohesion of soil : c_{ef} = 14,00 kPa
Saturated unit weight : γ_{sat} = 18,00 kN/m^3

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m]				Assigned soil
		x	z	x	z	
1		104,69	0,90	140,16	12,80	liner slope
		144,65	12,80	159,72	17,75	
		159,12	17,75	144,66	13,05	
		140,16	13,03	104,69	1,21	
2		14,87	0,18	104,69	1,21	waste
		140,16	13,03	144,66	13,05	
		159,12	17,75	92,00	40,00	
		88,00	40,00	11,01	1,93	
3		104,69	1,21	14,87	0,18	liner base
		11,01	1,93	10,36	1,93	
		14,80	-0,08	104,69	0,90	
4		171,43	14,81	161,76	17,75	residual clay
		159,72	17,75	144,65	12,80	
		140,16	12,80	104,69	0,90	
		14,80	-0,08	10,36	1,93	
		8,01	1,93	4,16	0,00	
		0,00	-2,09	0,00	-7,09	
		180,00	-7,09	180,00	14,72	

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m]					
		x	z	x	z	x	z
1		0,00	-3,61	10,26	1,93	107,55	2,16
		180,00	2,33				

Tensile crack

Tensile crack not input.

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Earthquake

Earthquake not included.

Settings of the stage of construction

Design situation : permanent

Results (Stage of construction 1)

Analysis 1 (stage 1)

Circular slip surface

Slip surface parameters							
Center :	x =	139,36	[m]	Angles :	α_1 =	-56,54	[°]
	z =	73,40	[m]		α_2 =	22,61	[°]
Radius :	R =	60,58	[m]				
Analysis of the slip surface without optimization.							

Slope stability verification (all methods)

Bishop : FS = 2,62 > 1,50 **ACCEPTABLE**
Fellenius / Petterson : FS = 2,43 > 1,50 **ACCEPTABLE**
Spencer : FS = 2,61 > 1,50 **ACCEPTABLE**
Janbu : FS = 2,60 > 1,50 **ACCEPTABLE**
Morgenstern-Price : FS = 2,60 > 1,50 **ACCEPTABLE**

Analysis 2 (stage 1)

Polygonal slip surface

Coordinates of slip surface points [m]									
x	z	x	z	x	z	x	z	x	z
85,70	38,86	139,24	12,94	144,80	12,94	159,36	17,63	159,69	17,75
Analysis of the slip surface without optimization.									

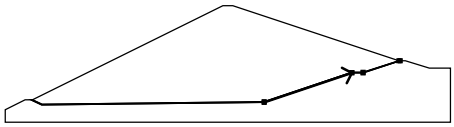
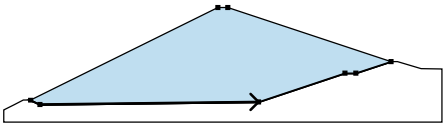
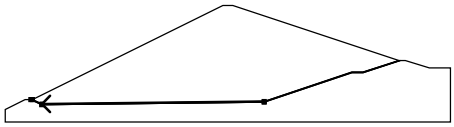
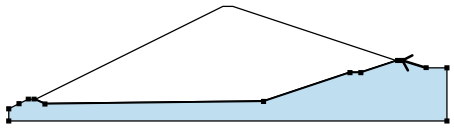
Slope stability verification (all methods)

Sarma : FS = 2,29 > 1,50 **ACCEPTABLE**
Spencer : FS = 2,27 > 1,50 **ACCEPTABLE**
Janbu : FS = 2,25 > 1,50 **ACCEPTABLE**
Morgenstern-Price : FS = 2,26 > 1,50 **ACCEPTABLE**

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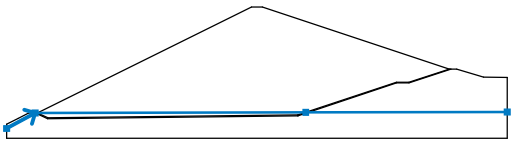
Input data (Stage of construction 2)

Assigning and surfaces

No.	Surface position	Coordinates of surface points [m] x	z	x	z	Assigned soil
1		104,69	0,90	140,16	12,80	liner slope
		144,65	12,80	159,72	17,75	
		159,12	17,75	144,66	13,05	
		140,16	13,03	104,69	1,21	
2		14,87	0,18	104,69	1,21	waste
		140,16	13,03	144,66	13,05	
		159,12	17,75	92,00	40,00	
		88,00	40,00	11,01	1,93	
3		104,69	1,21	14,87	0,18	liner base
		11,01	1,93	10,36	1,93	
		14,80	-0,08	104,69	0,90	
4		171,43	14,81	161,76	17,75	residual clay
		159,72	17,75	144,65	12,80	
		140,16	12,80	104,69	0,90	
		14,80	-0,08	10,36	1,93	
		8,01	1,93	4,16	0,00	
		0,00	-2,09	0,00	-7,09	
		180,00	-7,09	180,00	14,72	

Water

Water type : GWT

No.	GWT location	Coordinates of GWT points [m] x	z	x	z	x	z
1		0,00	-3,61	10,26	1,93	107,55	2,16
		180,00	2,33				

Tensile crack

Tensile crack not input.

Earthquake

Horizontal seismic coefficient : K_h = 0,1000
Vertical seismic coefficient : K_v = 0,0000

Settings of the stage of construction

Design situation : seismic

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Results (Stage of construction 2)

Analysis 1 (stage 2)

Circular slip surface

Slip surface parameters							
Center :	x =	138,07	[m]	Angles :	α_1 =	-56,52	[°]
	z =	72,32	[m]		α_2 =	21,35	[°]
Radius :	R =	58,59	[m]				
Analysis of the slip surface without optimization.							

Slope stability verification (all methods)

Bishop : FS = 1,98 > 1,10 **ACCEPTABLE**
Fellenius / Petterson : FS = 1,84 > 1,10 **ACCEPTABLE**
Spencer : FS = 1,97 > 1,10 **ACCEPTABLE**
Janbu : FS = 1,98 > 1,10 **ACCEPTABLE**
Morgenstern-Price : FS = 1,98 > 1,10 **ACCEPTABLE**

Analysis 2 (stage 2)

Polygonal slip surface

Coordinates of slip surface points [m]									
x	z	x	z	x	z	x	z	x	z
121,83	30,11	124,04	26,56	140,05	12,94	144,22	12,91	150,50	14,81
159,39	17,64	159,71	17,75						
Analysis of the slip surface without optimization.									

Slope stability verification (all methods)

Sarma : FS = 1,78 > 1,10 **ACCEPTABLE**
Spencer : FS = 1,69 > 1,10 **ACCEPTABLE**
Janbu : FS = 1,71 > 1,10 **ACCEPTABLE**
Morgenstern-Price : FS = 1,71 > 1,10 **ACCEPTABLE**

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APPENDIX D

CONSTRUCTION QUALITY ASSURANCE PLAN

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TONKMETER RESOURCE FACILITY
TONKMETER CLASS B LANDFILL AREA E
PHASE 1 DEVELOPMENT
CONSTRUCTION QUALITY ASSURANCE PLAN

Report No : P077-03-Rev1



Submitted to:

Vereeniging Refractories (Pty) Ltd
P O Box 117
Vereeniging
1930

DISTRIBUTION:

1 Copy		Gauteng Department of Agriculture & Rural Development
1 Copy		Department of Water & Sanitation
1 Copy	-	Vereeniging Refractories
1 Copy	-	Tonkmeter Resource Facility
1 Copy	-	Peter Legg Consulting

June 2024 P077

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1 INTRODUCTION

Verref owns the Tonk Meter Road Waste Disposal Facility (WDF) that is located on the Remaining Extent of Portion 81 of the Farm Rietfontein 128 IR, adjacent to the existing EMM Rietfontein Landfill in Springs. It lies south of the N17 Highway and it is bounded by Tonk Meter Road to the east, Rietfontein Road to the north and Kwa-Thema to the west.

Verref obtained an Integrated Environmental Authorisation No Gaut: 002/11-12/W0016 & Gaut: 002/11-12/E0228 dated 2 October 2017 from the Gauteng Department of Agriculture, and Rural Development (GDARD) for the development of the New Tonkmeter Road WDF as a Class B landfill. Verref intends the developing the first cells (Cell 1 and Cell 2) of Area E Phase 1 of the Tonkmeter Landfill and the associated drainage infrastructure

This CQA plan addresses the construction quality assurance procedures for the complete construction of Area E Phase 1 including Cell 1, Cell 2, Contaminated water pond, leachate tanks and gravel roads. This program has been developed to assure that the construction of the earthworks, civil works and geosynthetic components are in compliance with the project specifications and to demonstrate that the regulatory requirements for the construction are achieved.

This CQA Plan establishes procedures to verify that construction is in accordance with the Construction Drawings and Construction Specifications, meets the appropriate regulatory requirements, and develops the necessary documentation for submittal to the regulatory authority.

The objective of this plan is to establish:

- Duties of parties responsible for the CQA program
- Qualification requirements of the CQA Engineer
- Inspection activities
- Sampling procedures
- Document control measures
- Procedures for approving the materials used for construction
- Methods for assuring compliance to design standards and Construction Specifications during construction
- Procedures for resolving issues that may occur concerning the design and construction
- Documentation of construction and testing for submittal to the regulatory authority for their review

The intent of the CQA Plan is to provide independent third-party verification and testing, to demonstrate that the Contractors and Installers have met their obligations in the supply and installation of components and materials according to the Construction Drawings, Construction Specifications, and regulatory requirements. Construction of the Tonkmeter Landfill Area E Phase 1 earthworks, drainage systems, liner systems, concrete works and pipe works must be in accordance with the approved Construction Drawings and Construction Specifications. This CQA Plan establishes the CQA monitoring and testing program designed to ensure compliance with the Construction Drawings and Construction

Quality control is provided by the Manufacturers, Installers, and Contractors and refers only to their actions taken to ensure that materials and workmanship meet the requirements of the Construction Drawings and Construction Specifications.

2 PARTIES TO CONSTRUCTION QUALITY ASSURANCE

The following section provides descriptions of the parties referred to in this CQA Plan including their responsibilities and qualifications. Specific qualified personnel will be chosen once the work has been approved and the scheduled is confirmed for the selected CQA project members. (The SANS 10409 standard specification as amended has particular reference.).

2.1 Owner

Tonkmeter Resource Facility is the **Owner** and Operator. For the purpose of this CQA Plan and the Construction Specifications, all references to the **Owner** shall mean Tonkmeter Resource Facility.

2.2 Project Manager

The **Project Manager** is the official representative of the **Owner** and is responsible for all construction activities at the facility, including oversight and construction management. The Project Manager is responsible for coordinating construction and quality assurance activities for the project. The Project Manager shall be responsible for the resolution of all quality assurance issues that arise during the barrier system construction and must be involved in any decisions that may affect future operations of the impoundment.

The Project Manager shall serve as communications coordinator for the project initiating preconstruction and resolution meetings. As communications coordinator, the Project Manager shall serve as a liaison between all parties involved in the project to ensure that ongoing communications are maintained. The Project Manager, Engineer and CQA Engineer shall also be responsible for the resolution of all CQA issues that arise during the installation of the geosynthetics.

2.3 Design Engineer

The Design Engineer, also referred to as the “Designer” or “Engineer,” is the individual or firm responsible for the design and preparation of the Construction Drawings and Construction Specifications. The Designer is responsible for approving all design and Construction Specification changes, modifications, or clarifications encountered during construction. The Design Engineer for the Tonkmeter Landfill Area E Phase 1 barrier system is Peter Legg Consulting, located in Kaapschehoop, Mpumalanga.

2.4 CQA Engineer and CQA Monitor

The CQA Engineer and CQA Monitor will be responsible for understanding this CQA Plan and shall conduct CQA testing, monitoring, documentation, and reporting as required by this CQA Plan. The CQA Engineer will be the Engineer-of-Record and will sign-off the final construction report. The implementation and reporting of this CQA Plan shall be conducted under the direct supervision of an Engineering Council of South Africa registered civil engineer.

2.5 Geosynthetics Manufacturer

The geosynthetics manufacturer(s), also referred to as the "Manufacturer," is responsible for production of the geosynthetic components outlined in this plan. The Manufacturer may be affiliated with the Geosynthetics Installer. Each Manufacturer must pre-qualify that they are able to produce material that meets the requirements of the Construction Specifications (listed in Appendix B and on the certified drawings).

2.6 Geosynthetics Installer

The Geosynthetics Installer, also referred to as the "Geosynthetics Installation Contractor" or the "Installer," is responsible for proper installation of the geosynthetic components in accordance with the Construction Drawings and Construction Specifications. The Installer may be affiliated with the Manufacturer.

The Installer must pre-qualify by meeting the requirements outlined in the Construction Specifications. The Installer shall provide a qualified Superintendent who will provide full-time technical guidance to the field crew. The Superintendent will represent the Installer at all site meetings and will act as the spokesman for the Installer on the project. Welding technicians will be evaluated based on performance. The CQA Engineer, through the Project Manager, reserves the right to reject any welding technician whose performance is unsatisfactory.

2.7 Earthworks Contractor

The Earthworks Contractor, also referred to as the "Contractor," is responsible for completion of the site work as defined by contract with the Owner and in accordance with the Construction Drawings and Construction Specifications except for materials provided by the Owner or Geosynthetics Manufacturer and work performed by the Geosynthetics Installer.

The Earthworks Contractor will be responsible for retaining a surveyor to set lines and grades required for excavation, construction, and preparation of as-built drawings. Surveying shall be performed under the direction of a registered Surveyor.

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

A pre-construction meeting will be held at the site prior to the start of construction. The Design Engineer, Project Manager, CQA Monitor, CQA Engineer, Installer, CONTRACTOR, and others designated by the OWNER shall attend this meeting. The purpose of this meeting will at a minimum:

- Define lines of communication, responsibility, and authority;
- Conduct a site inspection to discuss work areas, work plans, stockpiling, lay-down areas, access roads, haul roads, and related items;
- Review the project schedule;

- Review the Construction Drawings, CQA Plan, and Construction Specifications;
- Review work area security and safety protocol.

This meeting will be documented by the CQA Engineer and copies of the meeting minutes will be distributed to all parties.

3.2 Progress Meetings

Weekly progress meetings will be held. At a minimum, these meetings will be attended by the CQA Monitor, the CQA Engineer or their designee, the Project Manager, the Installer, and the Contractor. The CQA Monitor is responsible for organizing and conducting the progress meetings. The purpose of this meeting will be to:

- Review the previous weeks accomplishments and activities;
- Review upcoming scheduled work and project milestones;
- Discuss any problems or potential construction problems;
- Review the results and status of CQA field and laboratory testing.

This meeting will be documented by the CQA Monitor and the minutes transmitted to all in attendance.

3.3 Resolution Meetings

Special meetings will be held, as needed, to discuss and resolve potential problems or deficiencies. As a minimum, these meetings will be attended by the Project Manager, CQA Engineer, CQA Monitor, and the Installer and/or Contractor. If the problem relates to a design issue, the Design Engineer shall also be present. The meeting will be documented by the CQA Engineer.

When deficiencies (items that do not the project requirements stated in the Construction Specifications) are discovered, the CQA Monitor or CQA Engineer shall immediately determine the nature and extent of the problem and notify the Contractor. If unsatisfactory test results identify a deficiency, additional tests will be performed to define the extent of the deficient material or work area.

The Installer or Contractor shall correct the deficiency to the satisfaction of the CQA Engineer. If unable to correct the problem, the CQA Engineer will notify the CQA Monitor who will assist during problem resolution. If the solution involves a design revision, the Project Manager and Design Engineer shall also be contacted. Design revisions can only be made by the Design Engineer.

The corrected deficiency shall be re-tested and/or approved before any additional related work is performed by the Installer or Contractor. Retest results shall be recorded by the CQA Monitor and included in the final report documentation.

4 EARTHWORK CONSTRUCTION QUALITY ASSURANCE

Construction of the Tonkmeter Landfill earthworks and base liner must be in accordance with the approved Project Drawings and Project Specifications. This CQA Plan establishes the CQA monitoring and testing program designed to ensure compliance with the Project Drawings and Project Specifications. The earthworks quality assurance testing program consists of testing of soil and rock materials used during the excavation and the construction of the Tonkmeter Landfill's NEMWA 2013 compliant containment barrier system. Quality assurance testing and observation is required during

excavation of subgrade, placement of the engineered fill, and construction of the liner system components for the barrier system.

4.1 Construction Monitoring and Testing

All components of the construction shall be observed and tested as required by the CQA Monitor to verify that the construction is in accordance with the Project Specifications. The Design Engineer shall review the work performed by the CQA Monitor and identify inadequate construction methodologies or materials which may adversely impact the performance of the Module 6 base liner. Visual observations and verification of the independent survey required for specific layers throughout the construction process shall be made to evaluate whether the materials are placed to the lines and grades as shown on the Project Construction Drawings.

The CQA Monitor and Design Engineer will give the Project Manager sufficient notice of anticipated completion of the construction components so that related CQA documentation may be reviewed and accepted without delay to the CONTRACTOR. Specific CQA observation and/or testing are required for the following:

- Engineered Fill;
- Subgrade Preparation including subsoil drainage;
- Compacted clay liner (CCL);
- Anchor trench backfill;
- Liner protection layer;
- Drainage Gravel and LCS Drainage Layer;
- Operations pioneering waste layer.

In addition to the above components, the CQA Monitor and Design Engineer will observe the construction of the aggregate base surfacing (geomembrane protection layer) and HDPE pipes for compliance with the Project Drawings and Project Specifications.

4.2 Engineered Fill and Anchor Trench Backfill

The CQA Monitor shall observe and document the subgrade preparation prior to placement of engineered fill and shall include:

- Monitoring the stripping of vegetated soil, and growth media to be stockpiled, if directed, in the area designated by the Owner;
- Monitoring that appropriate dust control measures are implemented;
- Visually inspecting the excavation for moisture seeps, soft or excessively wet areas, and unstable slopes;
- Monitoring subgrade preparation and confirming that the surface of the subgrade is free of soft, organic, and otherwise deleterious materials, and that the surface is firm and unyielding and in accordance with the Project Specifications (compaction density);
- Verify that the subgrade is suitable for supporting any overlying geosynthetic layers as required by the Project Specifications. Borrow materials for engineered fill and anchor trench backfill will be

obtained from the excavation area within the cell or the clay stockpile. CQA observation and/or testing is required during construction to verify that the materials and construction are in accordance with the Project Specifications. The tests to be performed, including testing frequency, are shown on Table 1. The testing frequencies specified in Table 1 may be increased when construction conditions warrant additional tests. Additional tests may be recommended by the CQA Monitor and approved by the Design Engineer.

Table 1: Engineered fill and anchor trench backfill construction testing

Test Designation	Applicable test method	Frequency
Visual-Method Soil Classification	ASTM D2488	Continual during excavation and placement of soils
Moisture-Density	SANS 3001-GR30	1 per 5 000 m^3 or each material type
Sieve Analysis	SANS 3001-AG1	1 per 1 500 m^3 or each material type
Atterberg Limits	SANS 3001-GR10	1 per 1 500 m^3 or each material type
Nuclear Moisture/Density[1]	SANS 3001-NG5	1 Per 500 m^3, one per lift, or one per day – whichever results in a higher number of tests
Moisture Content	SANS 3001-GR20	1 per 20 nuclear moisture tests
Sand replacement Cone Test	SANS 3001-GR35	1 Per 20 Nuclear Density Tests

Notes to Table 1:

1. Tests shall be performed on an even grid to provide adequate testing coverage. For large fills in small areas, the testing frequency shall be increased as necessary to ensure testing for each lift of soil placed.

4.3 Compacted Clay Liner

4.3.1 Test Pad Construction

Purpose and Scope

The purpose of the test pad is to establish the placement and compaction procedures to be used to construct the compacted clay liner component of the liner system and to ensure conformance with the Construction Specifications, and state and federal requirements. The test pad program is intended to establish methods, equipment, and procedures for attaining the specified properties, not to pre-qualify materials for the compacted clay liner. Once the methods and procedures have been verified by completing a successful test, the Contractor must use the same method and procedures to construct the compacted clay liner.

Subgrade Preparation

- The test pad shall be in an area of the project site designated by the Project Manager;
- The area within the limits of the test pad shall be cleared and grubbed of all trees, debris, stumps, and any other vegetation. After clearing and grubbing, the area shall be stripped of topsoil and/or organic materials;

- The surface of the subgrade shall be proof-rolled with a heavy-wheeled vehicle to detect soft zones, irregularities that may require removal and replacement. The finished subgrade surface shall be sloped at a grade of 1% to 3%;
- Construction of the test pad shall not commence until the condition of the subgrade has been examined and documented by the CQA Monitor.

Test Pad Construction

The test pad shall be constructed in a rectangular shape to a minimum plan area of 10 m by 15 m. The test pad should consist of a minimum 600 mm thick compacted clay liner placed and compacted in accordance with the requirements of the Project Specifications. The compacted clay liner in the test pad shall be constructed in four lifts not exceeding 200 mm loose and 150 mm in compacted thickness. The soil material shall be compacted within the specified moisture-density window. If appropriate, the moisture-density window may be modified by the Design Engineer to improve permeability or constructability based on the results of the test pad.

The Design Engineer shall finalize the moisture-density compaction window in writing prior to full-scale construction of the compacted clay liner and inform the CQA Monitor. Only when the CQA Monitor and Design Engineer have determined that each lift meets the target dry density and moisture content requirements, the following lift shall be constructed. The completed compacted clay liner shall be sealed by rolling with appropriate equipment (e.g., rubber tired or smooth drum roller). Overbuilding the test pad and trimming back may be necessary to obtain a sufficiently smooth top of clay surface and to protect the test pad from desiccation and cracking.

Monitoring and Testing

The CQA Monitor shall monitor and document the borrow material and construction of each lift of the test pad and shall ensure that construction is performed in accordance with the appropriate sections of the Project Specifications. Monitoring and documentation shall include:

- Weather conditions during construction;
- Equipment used in construction;
- Manner in which equipment was used;
- Soil type and USCS classification;
- Moisture content and dry density measurements for each lift;
- Approximate thickness of each uncompacted and compacted soil lift.

Field and laboratory testing shall be performed by the CQA Monitor, as a minimum, during construction of the subgrade and compacted clay liner in the test pad and shall include those tests presented in Table 2.

Table PSD1: Test pad construction testing

Test Designation	Applicable Test Method	Compacted Clay	Subgrade
Visual-Method Soil Classification	ASTM D2488	Continuous	1 per Soil Type
Moisture-Density	SANS 3001-GR30	1 Test	1 Test
Atterberg Limits	SANS 3001-GR10	1 Test	NA
Nuclear Moisture/Density[1]	SANS 3001-NG5	3 Tests Per Lift[1]	3 Tests
Moisture Content	SANS 3001-GR20	3 Tests Per Lift [2]	1 Test
Sand replacement Cone Test	SANS 3001-GR35	1 Per 20 Nuclear Density Tests	NA
Hydraulic Conductivity	ASTM D2434	2 for test pad[3,4]	NA
Field-Scale Infiltration Test & Permeability Evaluation Guelph Permeameter	ASTM D5126-16e1	1 on third layer and 1 on final layer	

Notes to Table 2:

1.Nuclear gauge tests for moisture content and dry density shall be performed at evenly spaced locations in a grid pattern within the footprint of the test pad. Acceptance will be based on test results that fall within the compaction window developed by the Design Engineer, or as modified by the Design Engineer based on pre-construction testing.

2. A correlation shall be developed between the moisture contents as determined by the nuclear gauge and conventional oven and/or microwave oven methods during construction of the test pad in order to facilitate construction testing and placement of compacted clay liner during full scale operations.

3. Upon completion of the test pad, samples shall be collected using 3-inch outside diameter thin walled sampling tubes (Shelby tubes) in accordance with ASTM D1587 or by the block sampling technique in accordance with ASTM D4220, at the discretion of the Design Engineer. Two samples in each lift shall be collected to represent the compacted clay liner. Samples should be collected outside of the future location of the field scale infiltration test.

4. The hydraulic conductivity evaluated in the laboratory (ASTM D5084) for the 3-inch diameter samples shall be correlated to the hydraulic conductivity evaluated in the field scale testing. Effective confining pressures of 5 psi (35 KPa) shall be applied during the test. The correlation is to provide a means for establishing criteria for laboratory and field testing of the full-scale (construction) compacted clay liner. In addition, in-situ hydraulic conductivity data is to provide information demonstrating the feasibility of constructing a compacted clay liner meeting the Project Specifications.

Test Pad Data Interpretation

The interpretation of the test results shall focus on the feasibility of constructing a full-scale compacted clay liner in conformance with the project and regulatory requirements. A letter report summarizing the test results shall be issued by the Design Engineer to the CQA Monitor at the completion of the test pad testing program. This letter report shall also be included as a part of the final project CQA documentation.

4.3.2 Compacted Clay Liner Construction Monitoring and Testing

CQA observation and/or testing is required during construction to verify that the compacted clay liner construction is in accordance with the Project Specifications. The tests to be performed, including testing frequency, are presented in Table 3. The testing frequencies specified in Table 3 may be increased when construction conditions warrant additional tests. Additional tests shall be recommended by the CQA Monitor and approved by the Design Engineer.

Table 3: Compacted clay liner construction testing

Test Designation	Applicable Test Method	Frequency[1]
Visual-Method Soil Classification	ASTM D2488	Continual during excavation and placement of soils
Moisture-Density	SANS 3001–GR30	1 per 5 000 m^3 or each material type
Sieve Analysis	SANS 3001–AG1	1 per 1 500 m^3 or each material type
Atterberg Limits	SANS 3001–GR10	1 per 1 500 m^3 or each material type
Nuclear Moisture/Density	SANS 3001–NG5	1 per 250 m^3
Moisture Content	SANS 3001–GR20	1 per 5 nuclear moisture tests
Sand Replacement Cone Test	SANS 3001–GR35	1 Per 20 Nuclear Density Tests
Laboratory Hydraulic Conductivity on Field Collected Sample [2,3,4]	ASTM D5084	1 per 1 500 m^3

Notes to Table 3:

1. Specified frequency or one per material type and source, whichever is greater.

2. Tests shall be performed on an approximately even grid to provide adequate testing coverage.

3. Samples will be collected and transported to the laboratory using the same procedures selected by the DEsign Engineer for the test pad (i.e., Shelby tubes or block samples).

4. Laboratory samples will be tested at confining pressures of 15 psi (104 KPa).

Construction observation and monitoring during the compacted clay liner placement includes:

- Verify that the Contractor obtains compacted clay liner material from the approved excavation or borrow location;
- Observe construction staking and/or grade control methods to verify that the compacted clay liner is placed to the lines, grades, and elevations shown on the Project Drawings without puncturing geotextile filters or geomembranes;
- Verify that fill is placed in loose lifts no more than 200 mm thick that result in a nominal compacted thickness of 150 mm or less;
- Verify that the Contractor adequately moisture conditions the borrow soils;
- Perform field testing in accordance with Table 3 to verify that the fill materials are placed to the moisture and density requirements indicated in the Project Specifications;

- Perform laboratory testing in accordance with Table 3 to verify that the compacted clay liner exhibits the required material properties;
- Promptly notify the Contractor of test results that affect the work. Notify the Project Manager of construction progress and of the results of all testing. In the event of failing tests, verify that the Contractor adequately reworks the areas which do not meet the Project Specifications;
- Observe that the Contractor takes adequate protective measures to maintain the surface of the compacted clay liner and prevent desiccation cracking;
- Verify that the CQA survey has been completed and that the Record Drawing furnished by the surveyor indicates compliance with the lines, grades, elevations, and tolerances as indicated by the Project Drawings and Specifications;
- Verify that the test samples and survey stake holes are remediated to not affect the performance of the layerworks.

4.4 Drainage Gravel and LCS Drainage Layer Placement

Drainage gravel is used in the LCS collection system and drainage system (28 to 38 mm stone) overlying the protective cushion geotextile over the geomembrane. Both pre-construction and construction testing are required for this material. Pre-construction testing consists of testing proposed materials from samples obtained at the aggregate or on-site borrow source. Construction testing consists of testing performed from samples obtained during delivery of materials during the cell construction. The tests to be performed, including testing frequency, for each material type are presented in Table 4. The testing frequencies specified in Table 4 may be increased when construction conditions warrant additional tests. Additional testing may be performed on suspect materials as recommended by the Design Engineer.

Table 4: Drainage gravel and LCRS drainage layer construction testing

Test Designation	Applicable Test Method	Designation Frequency
Sieve Analysis	SANS 3001–AG1	1 per 1 500 m^3 or each material type
Hydraulic Conductivity	ASTM D2434	1 per 1 500 m^3 or each material type
Visual-Method Soil Classification	ASTM D2488	Continual during excavation and placement of soils

Construction observation and monitoring required during the drainage gravel and LCS drainage layer includes:

- Verification that all pre-construction testing has been performed and that laboratory test results indicate compliance with the Project Specifications. The CQA Monitor shall assure that the Project Engineer and the Contractor receive prompt notification of material conformance;
- Verify that the material upon which the gravel will be placed (HDPE geomembrane) has been installed in accordance with the Project Drawings and Specifications, and that all required testing, and as-built documentation have been completed;
- Observe that care is taken when placing the protective cushion geotextile and drainage gravel as well as the LCS drainage layer on the HDPE geomembrane and that the geomembrane is not punctured or damaged during placement operations;

- Observe and document that appropriate light ground pressure equipment is used and that such equipment avoids sharp turns;
- Observation and monitoring of hauling equipment and spreading equipment to verify that the minimum thickness is maintained for spreading and hauling equipment above the HDPE geomembrane;
- Collect and transmit to the laboratory the required number of samples for testing;
- Communicate with the laboratory to verify that the materials tested comply with the Project Specifications;
- Visually observe the gravel and drainage materials to inspect for any variability in the material including variation in gradation, excess fines or any deleterious material present;
- Verify that the CQA Survey has been completed and that the Record Drawings furnished by the surveyor indicates compliance with the lines, grades, elevations, and tolerances as indicated by the Project Drawings and Specifications.

If the equipment or gravel placement procedures do not comply with the Project Specifications, the geomembrane shall be exposed and inspected for potential damage.

4.5 Pioneering Waste Layer Placement

Construction observation and monitoring required during pioneering waste layer placement includes:

- Observation and monitoring of hauling and spreading equipment to verify that the minimum thickness is maintained between equipment and the underlying geosynthetic materials;
- Verify the integrity of the geotextile layer by final inspection of all seams and geotextile panels;
- Verify that the pioneering waste layer fill materials meet the Project Specifications;
- Observe that pioneering waste layer material is pushed upslope on side slope areas;
- Verify that the thickness of pioneering waste layer required by the Project Drawings is achieved.

4.6 Surveying

Surveying shall be conducted such that all applicable standards are followed. The Surveyor shall furnish "Record Drawings" (also referred to as "as-built" drawings) for review by the Design Engineer. The CQA Monitor shall also review and approve the drawings prior to placement of a new system component over the work. Required Record Drawings shall be as specified in the Project Specifications. All surveying shall be performed under the direction of a registered surveyor. All Record Drawings shall be signed and certified by the registered surveyor who directed the CQA survey work. Record Drawings shall be at a scale not smaller than 1:1 000 scale. The accuracy of the surveying shall be sufficient to determine if the measurements are within the tolerances specified in the Project Specifications.

The required surveying of the barrier system elevations shall be carried out on a maximum 20 m square grid. Additional survey locations shall be recorded to define the following features in the barrier system: toe of slope, crest of slope, grade breaks, ridges and valleys, anchor trench, drainage system piping, perimeter drainage ditch, and position of barrier penetrations and instrumentation. The thickness of the geosynthetic barrier system components on the Project Drawings shall be interpreted

as negligible. Refer to the Project Specifications for details of the minimum requirements for surveys, Record Drawings, and grades, lines, and levels.

5 GEOSYNTHETICS CONSTRUCTION QUALITY ASSURANCE

Construction of the specified geosynthetics must be in accordance with the approved Project Drawings and Project Specifications. This Quality Assurance program consists of reviewing Geosynthetics Manufacturer's and Installer's Quality Control submittals, material conformance testing, and construction monitoring and testing.

The types of geosynthetics used in the barrier system construction include geotextile geomembrane, geosynthetic clay liner, and HDPE pipe and fittings. These geosynthetics are defined in the Project Specifications. Prior to and during construction, these geosynthetics shall be sampled and tested to determine if they conform to the Project Specifications. All geosynthetic conformance testing shall be the responsibility of the Geosynthetic Installer and the CQA Monitor.

5.1 Review Quality Control Submittals

Prior to geosynthetic installation, the Design Engineer shall review the Geosynthetic Installer's Quality Control submittals to evaluate and confirm that these materials meet the Project Specifications. The Design Engineer shall review the following submittals for each geosynthetic material specified in the Project Specifications.

- Geosynthetic material samples, name of Manufacturer, and minimum material specifications which shall include the Manufacturer's minimum physical properties of the material, test methods (SANS and ASTM Standards) used, and factory and site seaming methods;
- Manufacturer's Quality Control Manual followed during the manufacturing process;
- The origin (supplier's name and production plant), identification (brand name and number) and material properties of the resin used to manufacture the product;
- Geosynthetics Installer's Quality Control Manual including Project Specific Quality Control Plan and Method Statement, for the installation and testing of the geosynthetic;
- Resumé (Curriculum Vitae) of the Installer Superintendent, Master Seamer, and Seamers to be assigned to this project (geomembrane only);
- A copy of each of the Quality Control Certificates on each lot of resin issued by the resin Supplier for the specific material for this project. Geomembrane submittals shall include certification of the resin for extrusion welding rod;
- The result of quality control testing conducted on the resin used in manufacturing the specific material for this project;
- A listing which correlates the resin to the individual geosynthetic rolls and extruded materials;
- A copy of the geosynthetic roll Quality Control Test Certificates which shall be supplied at a minimum frequency of one (1) per every five thousand (5 000) square metres of geosynthetic material continuously produced and supplied to the project unless otherwise presented in the Project Specifications;

- Panel layout drawings for both GCL and geomembrane components showing the proposed installation layout identifying field seams as well as any variance or additional details which deviate from the Project Drawings;
- A detailed installation schedule for the project;
- Certification that the extrusion welding rod to be used is comprised of the same resin type as the geomembrane to be used (geomembrane only).

5.2 Materials Conformance Testing

5.2.1 Geomembrane Materials Conformance Testing

Prior to geosynthetic installation, the CQA Monitor shall obtain samples of the both the smooth and textured geomembranes for conformance testing to confirm that these materials meet the Project Specifications. One set of tests will be carried out at commencement, and as directed should the materials suppliers change of if the material batch is changed. 6 No samples each 300 mm x 300 mm are to be taken across the entire width of a roll of each geomembrane type, not including the first 1.5 m. The CQA Engineer shall mark each sample with the machine direction, roll number on the sample, and date the sample was obtained. The samples are to be sent to an accredited Geosynthetics testing laboratory where the following conformance tests are to be performed (according to the relevant ASTM standard test method):

Material Property	**ASTM Test method**
• Density	D1505
• Thickness	D5199
• Asperity height	D7466
• Tensile strength and elongation at break	D6639
• Stress Crack Resistance (SP-NCTL)	D5397, App
• Standard Oxidative Induction Time (OIT)	D3895
• High Pressure Oxidative Induction Time (HP-OIT)	D5885
• UV Resistance	D7238
• Oven Aging	D5721

All conformance tests shall be performed in compliance with the specification requirements of GRI-GM13 (Rev 15 2019) “Test Methods, Test Properties and Testing Frequencies for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes”, Table 1(b) and Table 2(b). The Engineer shall review the test results and shall report any non-conformance to the Owner and the Geosynthetics Installer.

5.2.2 GCL Materials Conformance Testing

Prior to geosynthetic installation, the CQA Monitor shall obtain samples of Geosynthetic Clay Liner (GCL) for conformance testing to confirm that the GCL meets the Project Specifications. One set of tests will be carried out at commencement, and as directed should the materials suppliers change of if the material batch is changed. 6 No samples each 300 mm x 300 mm are to be taken across the

entire width of a roll of the GCL, not including the first 1.5 m. The samples are to be marked with the machine direction, roll number on the sample, and date the sample was obtained. The samples are then to be sent to an accredited Geosynthetics testing laboratory where the following conformance tests are to be performed (according to the relevant ASTM standard test method):

	Material Property	ASTM Test method
•	Bentonite Swell Index	D 5890
•	Bentonite Dry Mass/ Unit Area	D 5993
•	GCL Peel Strength	D 6496
•	GCL Index Flux	D 5887

The conformance tests shall be performed in compliance with the specification requirements of GRI-GCL3 (Rev 5 2019) “Test Methods, Test Properties and Testing Frequencies for Geosynthetic Clay Liners (GCLs)”, Table 1(a). The Engineer shall review the test results and shall report any non-conformance to the Owner and the Geosynthetics Installer.

As a minimum the CQA Monitor shall independently confirm the mass per unit area of bentonite at zero percent (0%) gravimetric moisture content and the swell index of the bentonite in the GCL, along with the name of GCL manufacturer, roll identification markings, type of bentonite, type of carrier geotextiles and location of manufacturing plant.

5.2.3 Geotextile Materials Conformance Testing

Prior to geosynthetic installation, the CQA Monitor shall obtain samples of Protective Cushion Geotextile for conformance testing to confirm that the Cushion Geotextile CL meets the Project Specifications. One set of tests will be carried out at commencement, and as directed should the materials suppliers change of if the material batch is changed. 6 No samples each 600 mm x 600 mm are to be taken across the entire width of a roll of the Geotextile, not including the first 1.5 m. The samples are to be marked with the machine direction, roll number on the sample, and date the sample was obtained. The samples are then to be sent to an accredited Geosynthetics testing laboratory where the following conformance tests are to be performed (according to the relevant ASTM standard test method):

	Material Property	ASTM Test method
•	Mass per unit area	D 5261
•	Grab tensile strength	D 4632
•	Grab tensile elongation	D 4632
•	Trapezoidal tear strength	D 4533
•	Puncture strength (CBR)	D 6241

The conformance tests shall be performed in compliance with the specification requirements of GRI-GT12(a) (Rev 2 2016) “Test Methods and Properties for Nonwoven Geotextiles Used as Protection (or Cushioning) Materials”, Table 1(b). The Engineer shall review the test results and shall report any non-conformance to the Owner and the Geosynthetics Installer.

5.3 Geosynthetics Construction Monitoring and Testing

All geosynthetic components of the construction shall be monitored and tested to verify that the construction is in compliance with the Project Specifications. The Design Engineer shall identify inadequate construction methodologies or materials which may adversely impact the performance of the facility being constructed and existing structures. Visual observations throughout the construction process shall be made to evaluate whether materials are placed to the lines, grades and orientations as shown on the Project Drawings.

The CQA Monitor shall review the following submittals provided by the Installer during the project:

- Quality control documentation recorded during installation;
- Daily reports detailing arrival and departure times, the personnel present on-site, the progress of the work, the arrival of materials, and any problems encountered;
- Subgrade surface acceptance certificates for each area to be covered by the liner system, signed by the Geosynthetics Installer's Supervisor.

The CQA Monitor shall observe and document the geosynthetic installation including:

- Delivery and unloading of geosynthetic materials to the site to verify that the materials are not damaged and are properly labelled;
- Obtaining geosynthetic packaging identification slips for verification and generation of an on-site materials inventory;
- Subgrade conditions prior to liner installation and verify that any deficiencies (e.g. surface irregularities, protrusions, excessively soft areas, stones, desiccation cracks) noted are corrected;
- Verification that the Design Engineer has reviewed completed surveys;
- Handling of geosynthetic materials from storage to the work area;
- Temporary and permanent anchoring of geosynthetics to verify that Project Specifications are met;
- Verification that required overlap distances are met;
- A record of all incidents such as storms and fires which may have an influence on materials properties.
- Monitor and record ambient air temperatures;
- Verification that no continuous horizontal seams are placed on slopes unless approved by the Engineer; and
- Observation and documentation that ballast materials placed on top of the geosynthetics are done in such a manner as to ensure that the geosynthetics are not damaged.

5.3.1 Geosynthetic Clay Liner (GCL)

During installation, the CQA Monitor shall observe and document deployment, adequate overlap, seaming, and repairs to evaluate whether the installation is in accordance with the Project Specifications.

The CQA Monitor shall ensure that manufacturing quality control documentation from the Manufacturer of each type of GCL is supplied and submitted for approval and should include dates of manufacture, batch numbers and roll numbers, length and width.

Specifications

The supply and installation of the GCL will be according to the following specifications and codes of practice:

- GIGSA 1200 W(GCL) - 2008: Pro-Forma Standardised Specification for Reinforced Needlepunched Geosynthetic Clay Liners (GCLs);
- GRI-GCL3 (Rev 5 2019): Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCLs);
- EN-ISO or any other standard which can be shown to be equivalent to the standards specified may be acceptable;
- Project Specifications for Geosynthetic Clay Liners.

Quality Control Submissions

Manufacturing Quality Control documentation from the Manufacturer of each type of GCL supplied will be submitted for approval by the Engineer and CQA Monitor. Submittals to include:

- Dates of manufacture;
- Batch plant numbers and roll numbers, length and width;
- Documentation of the manufacturer's specific quality control program, which shall provide test data indicating the actual test values per roll op per batch, as may be applicable as called for in GRI-GCL3.

Materials General

The GCL rolls will be supplied to site in panels no less than 5 m wide to minimise the number of site seams required.

The products will be packaged, transported, unloaded and stored in accordance with the manufacturer's instructions.

Any accessory bentonite used for sealing seams, penetrations, or repairs, will be high-quality powdered or granular sodium bentonite from a recognized producer and must comply with the same specifications as the GCL itself, as contained in GRI-GCL3.

Each GCL roll will be visually inspected by the Installer and CQA Monitor when unloaded to determine if any packaging or material has been damaged during transit.

Storage of the GCL rolls will be the responsibility of the Installer. All GCL rolls will be stockpiled and maintained dry in a well-drained flat area away from high-traffic areas but sufficiently close to the active work area to minimize handling.

Installation

Prior to commencing GCL installation the Installer will submit a proposed panel layout drawing showing the proposed deployment pattern and sequence. The location of overlaps and details thereof must also be shown. In preparing the panel layout, the Installer will take into account the construction schedule, access restrictions and seam location limitations.

Prior to deployment of GCL over the prepared subgrade, the Installer will inspect all surfaces on which the GCL is to be placed. The subgrade should be firm and unyielding, with no abrupt elevation changes, voids or open cracks, and free from standing water. The CQA Monitor shall verify and sign-off that the subgrade is suitable for deployment of the GCL.

The subgrade surface should be smooth and consist primarily of fine-grained soil (maximum particle size should not exceed 5 mm) that is free of sharp and protruding edges, construction debris and other foreign matter.

Deployment

The GCL will be installed on the approved areas shown on the drawings, using methods and procedures that ensure a minimum of handling. The orientation of the GCL, i.e., which side faces up, will be in accordance with the drawings, or as instructed by the Design Engineer.

When possible, GCL deployment should begin at the higher elevations and proceed to the lower elevations. Care shall be taken to prevent damage to the bottom surface of the GCL when it is finally positioned across the sub grade.

The GCL will be placed one panel at a time in a relaxed condition with the required overlap so that it is in intimate contact with the underlying surface at all locations and free of tension or stress upon completion of the installation. All necessary precautions, including installing extra material, will be taken to avoid bridging of the material.

It is important to ensure that the GCL is not left exposed to the elements and therefore the subsequent covering activities must be co-ordinated accordingly with the GCL installation.

The Installer shall only deploy as much GCL that can be covered in a reasonably short time in the event of precipitation or as can be covered by the end of the working day with the geomembrane liner.

Seaming

The CQA Monitor shall verify sufficient overlap and that the specified seam procedures were followed as required in the Project Specifications. The CQA Monitor shall verify that all repairs are performed in accordance with Project Specifications.

GCL seams will be used where called for on the Drawings, shown on the approved panel layout, or as directed by the Design Engineer or CQA Monitor. The seam shall be created by overlapping adjacent edges by a minimum of 300 mm, and enhancing the seam as recommended by the manufacturer, or as instructed by the Design Engineer.

Overlaps will be shingled in the direction of anticipated water flow.

If the GCL does not incorporate a mechanism to ensure longitudinal overlap sealing (such as a bentonite impregnated edge) overlap areas will require on site overlap bentonite sealing. Edges are

pulled back and bentonite of the same source to that used in the product should be poured continuously along all seam edges.

When the GCL is cut to fit into small areas, in corners or around structures adjacent panels should be overlapped a minimum 300mm, adding abundant bentonite in overlapped areas.

Protection

The CQA Monitor shall observe and document that all geocomposite materials are covered with the approved material and that traffic or hauling equipment does not damage the material during installation. In the presence of wind, the geosynthetic layers will be securely anchored with sandbags or equivalent.

Submittal

The Installer shall submit an as-built GCL panel layout to the CQA Monitor.

5.3.2 Geomembrane

During geomembrane installation, the CQA Monitor shall observe and document deployment, trial seams, field seaming, non-destructive and destructive seam testing, and repairs to assess that the installation is in compliance with the Project Specifications.

Specifications

The supply and installation of the HDPE liners will be according to the following specifications and codes of practice:

- SANS 1526 (2015): Thermoplastics polyolefin sheeting for use as a geomembrane
- GRI-GM13 (2024): Test Methods, Test Properties and Testing Frequency for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes
- SANS 10409 (2020): Design, Selection and Installation of Geomembranes
- GRI-GM14 (2013): Selecting variable intervals for taking geomembrane destructive seam samples using the method of attributes.
- GRI-GM19a (2021): Seam Strength and Related Properties of Thermally Bonded Homogeneous Polyolefin Geomembranes/Barriers.

Storage and Handling

Geomembrane shall be stored in accordance with the Manufacturer’s recommendations, if approved by the Design Engineer, at a site selected by the Project Manager. Rolls shall be off-loaded using the appropriate equipment and straps. Rolls shall not be placed directly on the ground and shall be stacked no higher than three rolls. Only soft-sole shoes will be allowed on the deployed geomembrane and rub sheets shall be placed under equipment.

Deployment

The CQA Monitor shall verify that only approved materials are used, that each panel is given a unique panel number, that no geomembrane is placed during inclement or unsuitable weather conditions, that the geomembrane is not damaged during installation, that excessive wrinkles are not present, must confirm that the sand bags used and their contents for temporary or permanent ballast comply with Project Specifications and at least the same constraints as the soil protection layer in respect of geomembrane strain limitation, and that anchoring is performed in accordance with the Project Specifications and Project Drawings. The CQA Monitor shall record the deployment on a deployment log form including whether sand bag placement is by dragging or carrying to the point of deposition..

Trial Seams

The CQA Monitor shall verify that seaming conditions are adequate, tests are performed at required intervals, specified test procedures are followed, and that re-testing is performed in accordance with the Project Specifications. The Installer shall perform pre-weld testing at the beginning of each crew shift and immediately following any work stoppage (e.g., for lunch, weather, etc.) of 30 minutes or more. Seaming operation shall not commence until the CQA Monitor has determined that the seaming process meets the Project Specifications. Testing shall include visual observation of a trial seam a minimum of 1 m long on the geomembrane material. The Installer shall mark the trial seam with date, ambient temperature, geomembrane temperature, welding machine number, welding technician's initials, machine temperature, and speed. For extrusion welding, the Installer shall record the nozzle and extrusion settings and for fusion welding, the wedge temperature and machine speed shall be recorded. A 300 mm portion of each trial seam sample shall be archived by the CQA Monitor at the site. The CQA Monitor shall record the trial seam test results on a trial seam log form.

Field Seaming

The CQA Monitor shall verify that only approved equipment and personnel perform welding, all welding is performed under suitable conditions as specified in the Project Specifications, specified overlaps are achieved, seams are oriented in compliance withproject requirements, and that grinding techniques and extrudate meet project requirements for extrusion welding. The CQA Monitor shall record all field seaming on field seaming log forms.

Seaming shall not proceed at an ambient temperature below 5°C or above 45°C unless the Installer demonstrates the capability of achieving acceptable results through the utilization of special seaming techniques and total tensile strain limitations are not exceeded. Such cold or hot weather seaming shall be proven by an approved program presented in the Project Specifications or presented otherwise by the Design Engineer. If seaming operations are conducted at night, lighting equipment shall be sufficient to allow the Installer and CQA Monitor to adequately and safely perform their duties.

Non-Destructive Seam Continuity Testing

The CQA Monitor shall verify that the seams and repairs are non-destructively tested by CQC personnel in compliance with the Project Specifications. If a seam cannot be tested, the CQA Monitor shall observe cap strip operations and verify that test equipment and gauges are functioning properly and that test procedures are in compliance with the Project Specifications. The CQA Monitor shall verify that all failing tests are repaired and re-tested until passing results are achieved. The CQA Monitor shall record all non-destructive test locations on the vacuum test and pressure test log forms.

Destructive Seam Testing

The Installer shall obtain samples, at locations selected by the CQA Monitor, of the field seamed geomembrane approximately 600mm along and 300mm across the seam and centred over the seam as follows:

- All areas identified as suspect during non-destructive testing/monitoring;
- Seams that appear suspect to the CQA Monitor;
- A minimum of one sample per day;
- A minimum of one sample for each geomembrane seaming apparatus;
- A minimum of one sample for each representative working conditions (e.g.weather condition);
- A minimum of one sample every 150 metres of seaming for each apparatus.

Two types of samples shall be obtained at each location. The first sample shall consist of two specimens, each cut approximately 25 mm wide by 200 mm long, taken 1 m apart. These specimens shall be tested for peel and shear strength in the field by the Installer using a calibrated field tensiometer capable of quantitatively measuring peel and shear strengths. The CQA Monitor shall observe all field tests and record the test results.

If one or both of the specimens fail, the Installer shall take additional test samples 3 m from the point of the failed test in each direction and repeat the field test procedure. If these additional tests fail, then the procedure shall be repeated until the length of the poor quality seam is established. If the initial field tests pass, the second type of sample shall be taken between the passing specimens. The second sample type shall be approximately 1 m long and 300 mm across. The sample shall be divided into three equal sections and distributed and tested as follows:

- One sample - Manufacturer/Installer for their use;
- One sample - CQA Monitor for destructive testing;
- One sample - CQA Monitor for site archives.

Each sample shall be subject to the following destructive tests at a GRI-LAP accredited CQA geosynthetics laboratory (or similar approved by the CQA Monitor) or at the CQA Site Office and tested per ASTM D6392 with appropriate calibrated equipment:

- Seam shear strength (five tests);
- Seam peel strength (five tests).

For fusion seams, one peel strength test refers to testing of both sides of the seam. A passing test must have all five passing tests for the shear test and peel test.

Failed destructive tests shall be subject to additional testing until a passing area is found. The Installer shall take another test sample 3 m from the point of the failed test in each direction and repeat the field test procedure. If subsequent tests fail, then the procedure is repeated until the length of the poor quality seam is established. Once the field tests have passed, a second sample shall be taken between the passing specimens and tested by the Independent CQA Laboratory. Failed seams shall

be tracked according to the welding apparatus and the machine operator. All failed seams shall be bounded by locations from which passing Independent CQA Laboratory tests have been taken.

The Installer shall be responsible for patching all areas cut for test samples in accordance with the Construction Specifications and the Manufacturer's recommended procedures, and for non-destructive testing (e.g. vacuum box, etc.) of the patched seams. The CQA Monitor shall record all test locations, results, actions taken in conjunction with destructive test failures, and repairs.

Repairs

The CQA Monitor shall observe and document that all repair materials, techniques, and procedures used for repairs are approved in advance and meet the requirements of the Project Specifications. The CQA Monitor shall verify that all repairs are marked, recorded, repaired, tested, and that wrinkles are addressed, prior to being covered by other materials; and that repairs are performed as specified, including specified type of repair according to type of damage and proper patch size or dimension. The CQA Monitor shall record defects and repairs on repair log forms.

Protection

The CQA Monitor shall observe and document that all soil and stone drainage materials placed on top of the geosynthetics are done in such a manner as to ensure that the geosynthetics and underlying materials are not damaged.

5.3.3 Geotextile

During geotextile installation, the CQA Monitor shall observe and document deployment, overlaps, field seaming, and repairs to confirm that the installation is in compliance with the Project Specifications.

Deployment

The CQA Monitor shall confirm that the subgrade is in conformance with Project Specifications, any underlying installations are complete, installed as designed, and as-built documentation has been obtained.

Seams and Repairs

The CQA Monitor shall verify sufficient overlap and that the specified seaming (eg. Hot-air-gun welding or stitching etc.) and patch procedures were followed as required in the Project Specifications. The CQA Monitor shall verify that all repairs are performed in accordance with Project Specifications and that the seaming method, be it by hot-air-gun or stitching does not induce detrimental impacts on adjacent materials.

Protection

The CQA Monitor shall observe and document that all geosynthetics materials have no damage, no slippage of geosynthetics underlying layers and no excessive tensile stresses in the geosynthetics.

Testing and Acceptance

The CQA Monitor shall take samples from the geotextile material prior to deployment to evaluate conformance with the GRI-GT12(a) and GRI-GT13(a) as amended standard specifications and the Project Specifications for cushion protection geotextiles and filter separator geotextiles respectively.

As a minimum the CQA Monitor shall independently confirm the mass per unit area and CBR puncture maximum force and elongation, and 3 in-field seam strength test results for cushion protection geotextiles, along with the name of geotextile manufacturer, roll identification markings, type of polymer and filament and location of manufacturing plant.

Similarly as a minimum the CQA Monitor shall independently confirm the mass per unit area, CBR puncture strength and either the Apparent Opening Size or Gradient Ratio filter compatibility for separator and filter geotextile installations as required by the Standard and Project Specifications.

5.3.4 HDPE Pipe and Fittings

During high-density polyethylene (HDPE) pipe installation, the CQA Monitor shall observe and document that the installation is in accordance with the Project Specifications.

Placement

Observation that the handling procedures used do not damage the pipe, backfill is placed in accordance with the requirements of the Project Specifications so as not to damage the pipe, any foreign material is removed from the interior of the pipe, and indentations on the pipe are within the allowable limits.

Joints and Connections

Monitoring of the jointing and connection operations to verify that the Installer follows the Project Specifications or at least the pipe Manufacturer's recommendations, verification that the pipes are clean when installed, and that perforated sections of pipe are aligned properly prior to connection.

Non-destructive Testing

Observe and document any required testing of the pipe to verify accordance with the Project Specifications. The CQA Monitor shall keep a photographic record of post installation pipe pressure tests in progress along with the field data of date, time, location and product suppliers' identification markings.

5.4 Liner Interface Shear Testing

In order to verify the assumptions used in the landfill stability analysis carried out as part of the landfill design, it is necessary to conduct liner interface shear testing using the actual liner materials installed on site. The CQA Monitor shall arrange for samples of the two geomembrane types as well as the protection geotextile to be cut from rolls of the materials delivered to site. 6 No samples from each geosynthetic type, 600 mm x 600 mm are to be taken across the entire width of a roll of the geosynthetic material, not including the first 1.5 m. The samples are to be marked with the machine direction, roll number on the sample, and date the sample was obtained. The samples are then to be sent to the approved geotechnical testing laboratory interface shear testing.

The geotechnical properties of the CCL material shall also be sent to the laboratory in order for the laboratory staff to simulate the clay in the large shear box used for the interface shear testing. The Design Engineer shall coordinate the interface shear testing and rerun the stability analysis using the actual interface shear strength parameters obtained from the laboratory testing.

The revised stability analysis results shall be included in the construction completion report.

5.5 Liner Protection Load Testing

In order to verify that the installed liner will not exceed the design maximum strain value of 3%, it is necessary to conduct a vertical load test on test pads of the barrier system constructed in accordance with the design, using the actual materials of construction.

The Design Engineer shall issue details for the construction of the barrier test pad. The CQA Monitor shall witness and verify the construction of the test pad, the load testing, the scanning of the exposed geomembrane and the results. The proposed methodology for the liner protection load testing shall be as follows:

- Th CCL test pad constructed as described in section 4.3.1 shall be used for the liner protection load test. The surface of the CCL test pad shall be inspected and trimmed and rolled as deemed necessary to represent the clay surface before installation of the geomembrane liners.
- An aluminium test sheet 350 mm x 350 mm (provided by the testing company) is to be placed on top of the surface of the CCL.
- A sample of the 1,5 mm single textured HDPE geomembrane liner supplied to site (1.2 m x 1,2 m) is to be carefully over the aluminium test sheet with the textured side facing upwards, taking care not to deform the sheet before loading.
- The protection geotextile as supplied to site is to be placed on top of the geomembrane.
- A 300mm thick layer of the 38 mm stone supplied to site is to be placed over the protection geotextile.
- A sand bedding layer is to placed on top of the aggregate allowing for better seating of the load plate and application of the load.
- A plate load test to be conducted on top of the stone layer using a 310 x 310 x 40 mm square steel plate and jacking against the underside of a large excavator for kentledge.
- A load, as specified by the Engineer, is to be applied and maintained for 24 hours before being unloaded. The pressure must be monitored during this period.
- The load and liner layers are to be removed taking care not to apply any additional pressure over the area of the aluminium test sheet that may affect the results.
- The areas of worst indentation are identified on the aluminium test sheet, which are scanned and the % strain calculated.
- The results of this testing exercise are to be written up in a short report for submission to the regulatory authority.

5.6 LCS stone layer construction

The LCS stone layer must be placed in a way that minimises wrinkles in the HDPE geomembrane. The CQA Monitor must therefore ensure that the LCS stone placement is accrued out in the following manner:

- After installation of the protection cushion geotextile over the geomembrane, placement of the stone layer must only be done in the early morning while the geomembrane is cool and wrinkles are minimal.
- “Roadways” of stone approximately 3 m wide and minimum 300 mm thick must be created in a grid pattern of 10 m x 10 m across the landfill by means of end-tipping rather than pushing the stone. This grid will then lock the geomembrane liner in position. The thickness of the stone “roadway” will be determined by the total volume of stone required to cover the entire area between the “roadways” at the required final thickness of 200 mm.
- Once the entire base has been “locked” with the “roadway” grid, the excess stone in the roadways is to be spread over the open liner to the require thickness.
- A similar process is to be followed for the placement of the LCS stone layer over the side slopes. On the slopes, all the stone placement must be performed from the bottom upwards.
- The entire process of the LCS stone placement must be carried out under the fulltime supervision of the CQA Monitor, who must also monitor and verify the final thickness of the LCS stone layer.
- All wrinkles must be monitored and recorded.

5.7 Electric leak location survey

After placement of the LCS stone layers on both the base and side slopes of the landfill, an Electric Leak Location Survey of the completed system shall be carried out by an independent accredited company, according to either ASTM D8265 or ASTM D7007. The CQA Monitor shall witness and verify the findings of the ELLS.

6 DOCUMENTATION

An effective Quality Assurance program depends on thorough monitoring and documentation of all construction activities during all phases of construction and as a minimum shall comply with SANS 10409 as amended and the all-important inception meeting. Documentation shall consist of daily record keeping (including minutes of meetings), construction problem resolutions, design and specification changes, photographic records, weekly progress reports, chain of custody forms for test sample tracking, and a certification and summary report. During construction, all documentation shall be kept on-site and will be available for review by the Project Manager, Design Engineer, or CQA Monitor.

No section of the barrier system may be covered up until the CQA Monitor and Design Engineer (as appropriate) observes and approves the completed section of the barrier system and assures that all CQA documentation has been completed.

6.1 Daily Record Keeping

Daily records shall consist of field notes, observation and testing data sheets, summary of the daily meeting with the Installer and Contractor, and reporting of construction problems and resolutions. This information shall be submitted weekly along with a weekly summary to the CQA Monitor. Copies of all CQA documentation shall be maintained at the site and be made available for review by the Project Manager.

6.2 Soils Observation and Testing Data Sheets

Soils observation and testing data sheets generally include the following information:

- Date, project name, location, and weather data;
- A reduced-scale site plan, or full-scale plots, showing work areas and test locations;
- Descriptions of ongoing construction;
- Summary of test results and samples taken, with locations and elevations;
- Off-site materials received including quarry certificates;
- Test equipment calibrations, if necessary;
- Signature or initials of the CQA Monitor.

6.3 Geosynthetic Observation and Testing Forms

Geosynthetic observation and testing forms generally include the following information:

- Date, project name, location, and weather data;
- Identification of panel or seam number;
- Numbering system identifying test or sample number;
- Location and identification of repairs and date of repair;
- Length and/or thickness measurements for geomembrane panels or seams;
- Welding machine temperatures and settings;
- Welding machine and technician identifications;
- Location of tests and test results;
- Identification of testing technicians and time of tests;
- Signature or initials of the CQA Monitor.

6.4 Construction Problem and Resolution Documentation

Any construction problem which cannot be resolved between the Installer, Contractor, and CQA Monitor may require a special meeting in order to resolve the problem. The problem should be discussed with the Project Manager, and Design Engineer if a design issue is involved. Specific written documentation of that problem should be prepared, if warranted, and will generally include the following information:

- Detailed description of the problem;
- Location and cause of the problem;
- How and when the situation or deficiency was identified;
- How the problem was resolved;
- Any measures taken to prevent similar problems in the future;
- Signature of the Design Engineer and CQA Monitor'

Copies of all Construction Problem and Resolution correspondence will be submitted to the Project Manager.

6.5 Photo Documentation

All phases of construction shall be sufficiently photographed and/or audio video recorded by the CQA Monitor. Photographs shall be identified by separate photographic log by location, time, date, and name of the person taking the photograph. A camera which records the time and date shall be used. Representative photographs will be included in the certification report.

6.6 Design and Specification Changes

If it is necessary to address Project Drawings and/or Projects Specification changes, modifications, or clarifications during construction, the CQA Monitor or Design Engineer will inform the Project Manager. Project Drawing and Project Specification changes shall only be made with written agreement from the Project Manager and Design Engineer, and approval of the regulatory authorities if required.

6.7 Construction Certification Report

At the completion of construction, a construction certification report shall be prepared and signed by the CQA Monitor and Design Engineer to certify that the work has been performed in compliance with the licence conditions, Project Drawings and Project Specifications and will contain the following general information:

- Summary of all construction activities;
- Observation and test data summary sheets, inclusive of a table reflecting statistical analyses i.e. for each test method on all materials the number of tests; minimum, maximum and mean values; standard deviation; number of non-compliances and rectification shall be included;
- Photographic documentation;
- Details of CQA procedures adopted;
- Summaries of all QC and CQA testing;
- Earth works quality control documents;
- Concrete works quality control documents;
- Geosynthetic quality control documents;

- Liner installation observations, such as for deployment, trial seams, defect repair, destructive testing and non-destructive testing;
- Details of defects and repairs carried out;
- Sampling, testing locations, and test results;
- Confirmation of interface shear strength parameters (peak and residual) using the actual geosynthetic materials supplied and installed on site;
- A description of significant construction problems and the resolution of these problems;
- Changes to the Project Drawings or Project Specifications and the justification for these changes;
- As-built drawings; and
- A certification statement signed and certified by the Design Engineer and CQA Monitor, by whom the CQA activities were supervised and work performed in responsible charge.

The As-built Drawings shall be prepared by the Surveyor and shall accurately locate all construction items including the lines, grades, and thickness of all soil components for the liner system.

LIST OF APPENDICES

Appendix A: Parties involved in the CQA implementation

Appendix B: Design parameters (materials strength, internal and interface shear resistance required; strain limitation; slopes; service life requirement (in years); assessed leakage rate and action leakage rate in ℓ/ha/d).

Appendix C: Standard specifications

Appendix D: List of Design Drawings as submitted to the Regulator

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APPENDIX A

PARTIES INVOLVED IN THE CQA IMPLEMENTATION

Company Name	Representative Name	Email address	Contact number
Owner			
Project Manager			
Design Engineer			
CQA Monitor			
Geosynthetics Manufacturer			
Geosynthetics Installer			
Earthworks Contractor			
Independent CQA Laboratory			

APPENDIX B

DESIGN PARAMETERS

1. SITE CONDITIONS

1.1 Surface Water

- The nearest water course: *Kleinblesbokspruit is 800 m to the north of the site*
- Shortest distance to the 1:100 year flood line: *700 m*
- Regional Rainfall: *700 mm/annum*
- Regional evaporation: *1 700 mm/annum*
- 1:50 year 24hr duration storm event: *160 mm*

1.2 Groundwater

- Depth to groundwater: *20 m (shallow perched) and 40 m (deep) below NGL*
- Aquifer classification: *Both non-aquifers with low potential sustained yield in Vryheid (perched) and Dwyka / Malmani (deep).*

1.3 Geotechnical profile

- Depth to pebble marker: *0,9 m to 1,8 m*
- Transported soil: *Aeolian sands*
- Residual soil: *Residual shale (silty clay) overlying residual mudrock (clay)*
- Depth to hard rock: *18 m below NGL*
- Foundation rock description: Shale (Vryheid) and mudrock (Dwyka), Malmani Dolomite at 28 m to 38 m
- Collapsible structures: *No, Aeolian sands have been removed by quarrying operation. Dolomite risk hazard for the development of sinkholes and subsidences is low; Inherent Hazard Class (IHC) 1.*
- Bearing capacity and consolidation: *450 KPa at base of landfill excavation*
- Seismicity: *Low due to historical undermining ; 0,1g m/s^2)*

1.4 Topography

- The side slopes: 1:3 (V:H)
- Minimum slope: 2%
- Maximum slope: 10%

2. MATERIALS PROPERTIES

2.1 Clay for barrier systems

- PI: *17 - 20*
- Standard proctor MDD: *1 720 KN/m^3*
- Optimum Moisture Content: *16,8%*
- Percentage clay: *23 - 49%*
- Maximum particle size: *4,75 mm*
- Permeability: *2,7x10^{-8} to 6,7x10^{-9}cm/s*
- Minimum shear strength: *29,4° and 14,3 KPa*

2.2 Geomembrane

- Indicator parameters: compliance with SANS1526 (2015), with no project specific deviations
- Performance parameters: ≤ 3% total tensile strain

2.3 Liner Protection

- Description of protection: *1500 g/m^2 nonwoven needle punched polyester geotextile overlain with 200 mm layer of 38 mm stone*.
- Particle size distribution: mm

2.4 Waste *(municipal general waste)*

- Waste Type assessed as: *Type 2 as per NEMWA R635 of 2013*
- Material Strength Properties (density and strength): *10 KN/m3; 25° and 15 KPa*
- Degree of saturation: *30%*
- Permeability: *10^{-2} – 10^{-3} cm/s*
- Estimating pollution period (years): *30 years*
- Maximum depth and rate of rise: *36 m; 7 m per year*

3. SYSTEM PERFORMANCE CRITERIA

- Predicted mechanism of failure: *1:7 height of waste to width ratio; failure mechanism is sliding followed by slip circle combination.*
- Minimum factor of safety: *2.09 on 1:3 final slopes and 1.29 on 1:2 interim operating slopes.*
- Frequency of PCD overtopping: *never based on the fact that the adjacent cell provides a buffer capacity for extreme rainfall events*.
- Action leakage rate: l/ha/d over ha of barrier system No.
- Strain limitation: *3% on base and 4% on side slopes*
- Elevated temperature and duration: *35°C and 30 years*:

Interface shear strength: test results to be determined after contractor has been appointed.

APPENDIX C

STANDARD SPECIFICATIONS

APPENDIX B: Standard specifications

Standard Specification	Description
SANS 1200 C	Site Clearance
SANS 1200 D	Earthworks
SANS 1200 DB	Earthworks (pipe trenches)
SANS 1200 DM	Earthworks (Roads, subgrade)
SANS 1200 GA	Concrete (Small Works)
SANS 1200 L	Medium pressure pipelines
SANS 1200 LB	Bedding (Pipes)
SANS 1200 LE	Stormwater drainage
SANS 1200 ME	Subbase
GRI-GCL3 (2019)	Test Methods, Required Properties, and Testing Frequencies of Geosynthetic Clay Liners (GCLs)
SANS 1526 (2015)	Thermoplastics polyolefin sheeting for use as a geomembrane
SANS 10409 (2020)	Design, Selection and Installation of Geomembranes
GRI-GM13 (2024)	Test Methods, Test Properties and Testing Frequency for High Density Polyethylene (HDPE) Smooth and Textured Geomembranes
GRI-GM14 (2013)	Selecting variable intervals for taking geomembrane destructive seam samples using the method of attributes.
GRI-GM19a (2021)	Seam Strength and Related Properties of Thermally Bonded Polyolefin Geomembranes
GRI-GT12(a) (2016)	Test Methods and Properties for Nonwoven Geotextiles Used as Protection (or Cushioning) Materials
GRI-GT13(a) (2017))	Test Methods and Properties for Geotextiles Used as Separation Between Subgrade Soil and Aggregate
SANS 1083 (2014)	Aggregate
ASTM D7007 ASTM D8265	Electric leak location survey to be undertaken post placement of LCS stone layer. Standard Practices for Electrical Methods for Mapping Leaks in Installed Geomembranes

APPENDIX E

TONKMETER LANDFILL AREA E - DRAWINGS

Drawing No	Rev	Title
P077E-300	A	Tonkmeter Landfill – Overall Layout
P077E-301	A	Tonkmeter Landfill Area E - General Arrangement
P077E-302	A	Tonkmeter Landfill Area E – Sections and Details
P077E-303	A	Tonkmeter Landfill Area E – Sections
P077E-304	A	Tonkmeter Landfill Area E – Proposed Final Landform