



BASHAN CONSULTING SERVICES
(Geosciences & Geotechnical Business Consulting)

ALIWAL NORTH – PROPOSED LANDFILL SITE

Geotechnical Investigations Report: Proposed Aliwal North Landfill Site, Aliwal North, Eastern Cape.

Report Number: BCS-24-0029

(Effective Date: 30 September 2024)

Prepared by Bashan Consulting Services for:

Afroteam Consultants

Acting for and on-behalf of:

The Walter Sisulu Local Municipality.



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Author(s):	Kelvin Chimanya <i>Pr.Sci.Nat.</i>	Project Consultant (Engineering Geosciences)	BSc (Hons) Geology, SACNASP, MSAIEG, MGSSA, MSAICE, MNHBRC
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	Bashan Consulting Services	(01)	



Author
Kelvin Chimanya *Pr.Sci.Nat.*
Project Geotechnical Consultant

BASHAN CONSULTING SERVICES

Ruimsig Office Estate
72 Hole-In-Hole Avenue
Ruimsig, 1724
Gauteng, South Africa

T: (+27) (0) 11 029 1981

M: (+27) (0) 81 551 9343

F: (+27) (0) 86 556 0890

E: consulting@bashancorp.co.za

W: www.bashancorp.co.za

02 September 2024

The Project Manager
Afroteam Consultants
Unit 8 – CK Business Center
Mbuque Extension Mthatha,
5099
Eastern Cape
South Africa

Attention:

RE: ALI WAL NORTH – PROPOSED LANDFILL SITE

We are glad to submit our final, geotechnical investigations report conducted at the proposed construction site of a municipal landfill near Aliwal North in the Eastern Cape Province. The purpose of the investigation was to establish the geotechnical conditions on site for the proposed infrastructure.

The final geotechnical assessments contained in this Report are based on the results of a desktop study, reconnaissance survey, DCP testing survey, trial / test pitting, soil sampling activities, engineering materials testing and data analysis.

We trust the above is in order and should you wish to seek further information and/or clarifications, kindly do not hesitate to contact the undersigned at your earliest convenience.

Yours faithfully,



Kelvin Chimanya Pr.Sci.Nat.

Project Geotechnical Consultant

EXECUTIVE SUMMARY

The Walter Sisulu Local Municipality (“WSLM” or the “Client”) has retained Afroteam Consultants (or “Afroteam”) to conduct a near-surface, geotechnical site investigation programme at the proposed construction site of a municipal landfill, situated approximately 5km west of the town of Aliwal North. Although the development details remain scarce, it is envisaged that proposed development is likely to consist of a double volume steel frame structure which will be utilised as a recycling shelter.

A total of six (06) test pits were excavated on site. In addition, six (06) dynamic cone penetrometer (DCP) tests were performed at the project site. Other activities included soil profiling and sample collection and the conducting of laboratory testing of the soil materials testing at an engineering materials testing laboratory at Roadlab.

Based on the 1:250 000 Geological map of the Aliwal North region (sheet 3026), the oldest rocks at the site are assigned to the Tarkastad Subgroup (Trt), a stratigraphic unit within the Beaufort Group of the Karoo Supergroup. The Trt lithologies are comprised of a brownish red to greenish-grey mudstone with intercalated fine to medium-grained, Sandstone. Dolerite intrusions (Jd) in the form of sills, sheets and dykes cover the upper parts of the project site.

The current geotechnical site investigation programme has revealed a minimum of three (03) soil material sublayers divided into Hillwash (0 – 1 023 mm); Colluvium (1 023 – 1 203 mm) and the, Residual soils (1 023 – 3 000 mm). The profiles generally indicate a similar material layering across the site.

Based on the available geotechnical data, the recommended geotechnical site class at the proposed construction site is C - C1 / S – S1. Bashan Consulting highly recommends the removal of the upper cohesionless, un-engineered, Hillwash prior to the commencement of the construction stage. The foundation types recommended for a double volume steel frame structure which will be utilised as a recycling shelter at the proposed site are the Spread Footings (or pad footings) and / or the Strip Footings with light reinforcement. The concrete floor slab (surface bed) with a light reinforcement (193 Ref Mesh) should be founded on a completed terrace compacted using G6 or G7 quality material of a minimum of 200 mm thickness with a subgrade reaction of 67MPa / m.

For building installations, the recommended founding options for a double-storey building are reinforced concrete ground beams or solid slabs on piled or pier foundations (as the first option). Option 2 includes stiffened strip footings, stiffened raft / cellular raft. Alternative foundation systems for heavier structures could include a 500 mm layer of imported G6 / G7 gravel material compacted to 95% of Mod AASHTO density below the footings to increase the allowable bearing pressures to 150 kPa. Compaction of in-situ soils below individual footings, coupled with deep strip foundations and / or soil raft is highly recommended.

All pavement layers requiring material classifications better than G8 will need to be imported from a commercial source or nearby barrow pit. It is recommended that should terracing be required, a minimum quality G7 material be considered for the backfilling.

Isolated areas where the materials do not meet the required specifications will have to be determined at the construction stage. This will have to be undercut to the depth of the layer and replaced with a suitable material or modified by means of stabilization.

Special care must be taken to ensure adequate surface drainage to prevent the accumulation of water next to structures. Isolated areas where the materials do not meet the required specifications will have to be determined at construction stage. This will have to be undercut to the depth of the layer and replaced with a suitable material or modified by means of stabilization.

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1 INTRODUCTION AND TERMS OF REFERENCE

1.1 General

The project developer at the proposed construction site of the Aliwal North landfill site is the Walter Sisulu Local Municipality ("WSLM" or the "Client"). The WSLM has appointed Afroteam Consultants (or "Afroteam") to conduct a near-surface, geotechnical site investigation programme at the proposed Aliwal North Landfill site. Bashan Consulting Services ("Bashan Consulting" or the "Consultant") has been retained by Afroteam to act as the Project Geotechnical Consultant on their behalf.

Bashan Consulting understands that one of the greatest risks is in the ground. The soil, rock and water form the core foundations upon which our social and physical infrastructure is built. Hence, our specialist knowledge of these components during the early planning phases (and through to design and construction) of a project can ensure success.

A landfill site (also known as a tip, dump, rubbish dump, garbage dump, or dumping ground) is a site for the disposal of waste materials. Operators of well-run landfills for non-hazardous waste meet predefined specifications by applying techniques to confine waste to as small an area as possible and compact waste to reduce volume. They can also cover waste (usually daily) with layers of soil or other types of material such as woodchips and fine particles.

Landfills have the potential to cause a number of social and environmental issues. Infrastructure disruption (such as damage to access roads by heavy vehicles) may occur. Pollution of local roads and watercourses from wheels on vehicles when they leave the landfill can be significant and can be mitigated by wheel washing systems. Pollution of the local environment (such as contamination of groundwater or aquifers or soil contamination) may occur, as well.

1.2 Terms of Reference

The primary objective of the investigation was to assess the soil and rock profile across the sites and obtain engineering parameters for the design of foundations for the proposed development. The general scope of the appointment for this project is to conduct a systematic approach towards establishing the site stability and a materials investigation aimed at investigating the suitability of the site for the intended engineering use at the project site. The current site investigation is thus being used to access soil materials, define soil properties and, soil behaviour (on site) and to establish the site-specific geotechnical conditions at the proposed project. The scope, inter alia, has included the following:

- Evaluation of the site-specific geology and hydrogeology of the site and to establish the subsoil conditions prevailing at the proposed project site.
- Conducting a near surface soil investigations horizon that seeks to establish an in-situ evaluation of the ground profile to a minimum target depth of 2.0 m (or to refusal depth), identification of the soil profile below the site using trial holes and collecting representative samples from the main soil horizons.
- Identifying conditions and constraints (such as mining-related problems, areas of outcrop, slope instability, contaminated land, unconsolidated fill, etc.), and provide information on the drainage of the site.
- Submitting appropriate recommendations regarding the existing conditions and risk mitigating measures for the sole aim of aiding the effective design of the proposed structures on site.
- Conducting a review of the general geotechnical stability conditions on site and proposing new recommendations.

1.3 Project Scope

The area of interest consists of a virgin land located 5 km west of the town of Aliwal North. Although the development details remain scarce, it is envisaged that proposed development is likely to consist of a double volume steel frame structure which will be utilised as a recycling shelter. The scope of work includes establishing the following:

- The type of material at the proposed site
- The thickness of all existing materials on site
- Engineering properties of the existing soil materials on site.
- Identification of any geotechnical problems.

1.4 Available Information

1.4.1 Topographic Data

- 1: 50 000 scale topographic map series of the Aliwal North area (Sheet 3026 DA) issued by the Chief Directorate-Surveys and Mapping, Department of Land Affairs, 2001

1.4.2 Geological Information

- 1:250 000 scale geological map series of the Aliwal North region (Sheet 3026) issued by the Geological Survey of South Africa (Council for Geoscience), 1983.

1.5 Limitations

The information provided in this Geotechnical Report is based on information provided by the Client and or the client's representatives, published geoscientific / geotechnical literature, maps, and information published in the public domain and that collected by Bashan Consulting during the current geotechnical survey.

1.6 Author's Credentials and Declaration of Independence

The Author of this Report, Kelvin Chimanya is a professional, independent, Engineering Geologist, registered with the South African Council of Natural and Scientific Professions (Pr.Sci.Nat #: 400297/12). Kelvin has over 20 years' experience obtained in the mining and civil engineering sectors. He has conducted several geotechnical investigations for infrastructural development including foundation investigations for housing developments (in line with the generic specification, GFSH-2, 2001) locally within the Republic of South Africa and Southern Africa (as a region).

In addition, Kelvin is a Member of the Geological Society of South Africa (GSSA #: 966428), a Member of the South African Institution of Civil Engineering (SAICE #: Member 201701195) and is an Associate Member of the South African Institute for Engineering and Environmental Geologists (SAIEG #: 17/489).

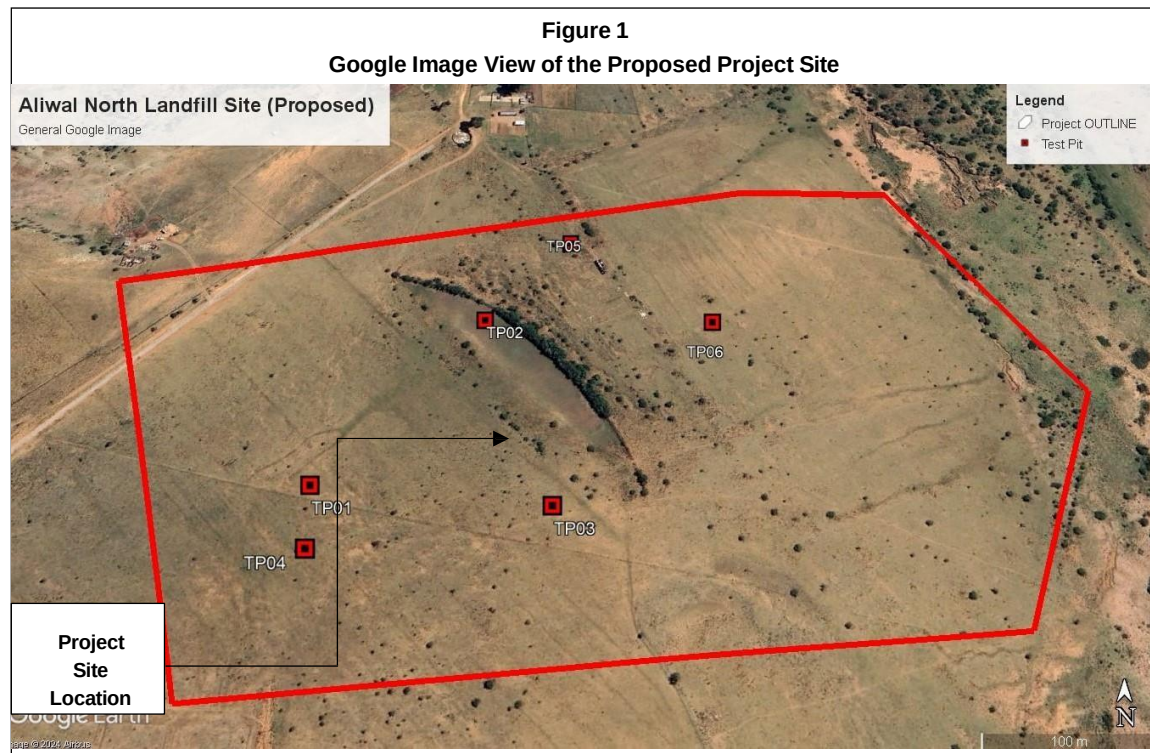
The compilation of the Report and any other work done by Bashan Consulting for the Client and /or its Associates is strictly in return for professional fees. Payment for the work is not in any way dependent neither on the outcome of the work, nor on the success or otherwise of the Company's own business dealings. As such there is no conflict of interest in undertaking the geotechnical investigation survey by Bashan Consulting (as contained in this document).

2 SITE DESCRIPTION

2.1 Site Location

The proposed project site is centred at Latitude 30°41'26.86"S and Longitude 26°39'56.38"E and is approximately a km west of the existing municipal landfill located at the Maletswai Landfill. The proposed landfill site measures approximately 20 hectares (in surface area) and is located approximately 5 km west of Aliwal North (officially Maletswai).

Aliwal North located on the banks of the Orange River is a medium-sized commercial centre in the northernmost part of the Eastern Cape. The town is part of the Walter Sisulu Local Municipality of the wider Joe Gqabi District Municipality. The Google Earth image of the project outline is shown in Figure 1 whilst the regional site map is shown in Figure 2.



2.2 Physiography

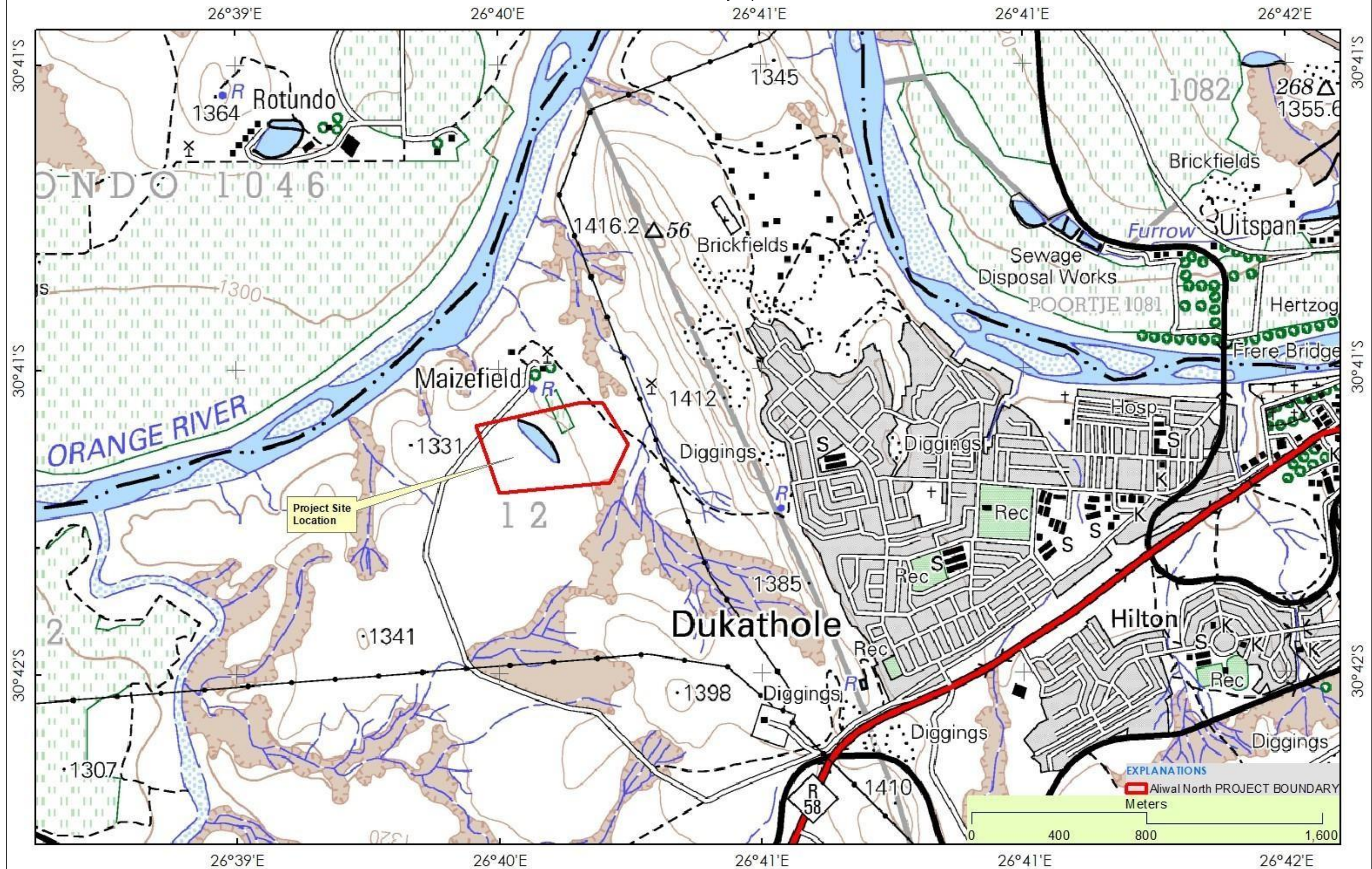
Topographically, the area has a variable gradient comprising of gentle slopes. The area is considered naturally stable as far as slope instability is concerned with elevation varying between 1318 m (in the east) to 1327 m above mean sea level towards the west with a slope gradient of approximately 3 %.

No preferential drainage channels were noted on site, and it is expected that drainage, particularly during periods of heavy or prolonged rainfall would be in the form of sheetwash down towards the south..

2.3 Climate

Climatic indicators available for the proposed project site is characterised as tropical and subtropical steppe climate. The warmest month is January with an average temperature of 21°C whilst the coolest month is June with an average temperature of 7°C (Source: www.weatherbase.com). The average annual precipitation is 533 mm with February being the month with the most precipitation (84 mm) and July at 10 mm being the month with the least precipitation.

Figure 2
General Site Locality Map



PROPOSED ALI WAL NORTH LANDFILL SITE

REGIONAL SITE LOCALITY PLAN

1:15,000

A3 PAPER SIZE



MAP DATUM

WGS 84
(ZONE 35 S)

DATE

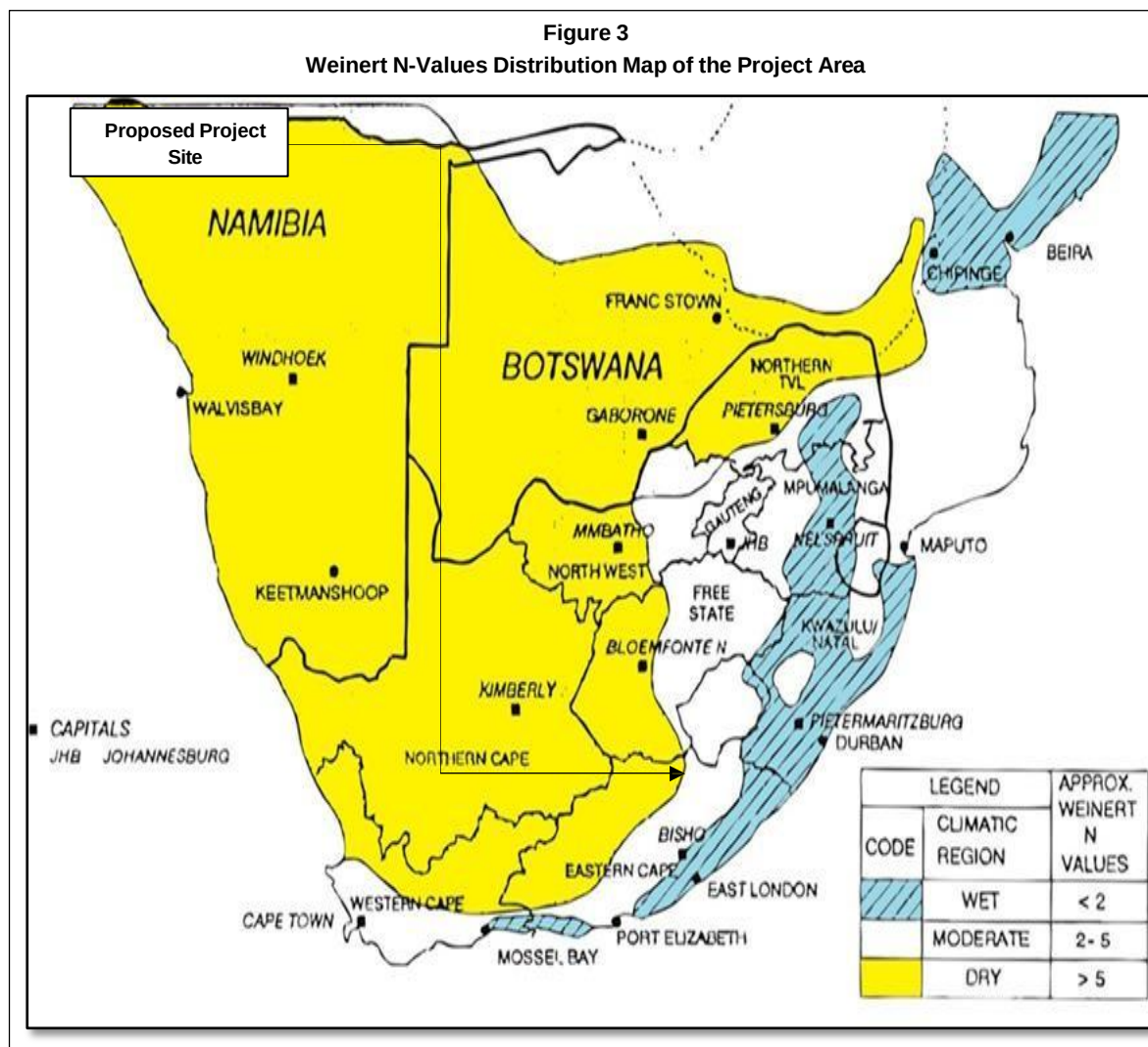
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2.4 Macro-climatic indicators

The macro-climatic regime of an area plays a fundamental role in the development of a soil profile and the weathering of rocks. From an engineering point of view, a soil can be regarded as any uncemented or a weakly cemented accumulation of mineral particles formed mainly by the weathering of rocks, with void spaces filled with air and / or water between the particles. Weinert (1980) developed the N-value to differentiate between regions of similar weathering characteristics (Figure 3).



A brief review of the above map confirms that the Weinert's N-Value for the project site is moderate (2-5). This indicates that the weathering of primary minerals is predominantly via chemical decomposition with deep weathering profiles producing significant secondary minerals.

The residual soils produced are a mixture of resistant primary minerals (such as quartz), insoluble weathering products (such as alumina or silica) and other new or secondary minerals such as clays. They may also contain soluble products such as chlorides, sulphates and bicarbonates of sodium, potassium, magnesium, or calcium, which may subsequently be leached out. In this climatic zone, montmorillonite changes to kaolinite (in the topsoil layers) thereby producing deep weathering profiles. The soil types, which are common in the study area, include fertile loam soils, clay-loam and sandy- to clayey loam soils.

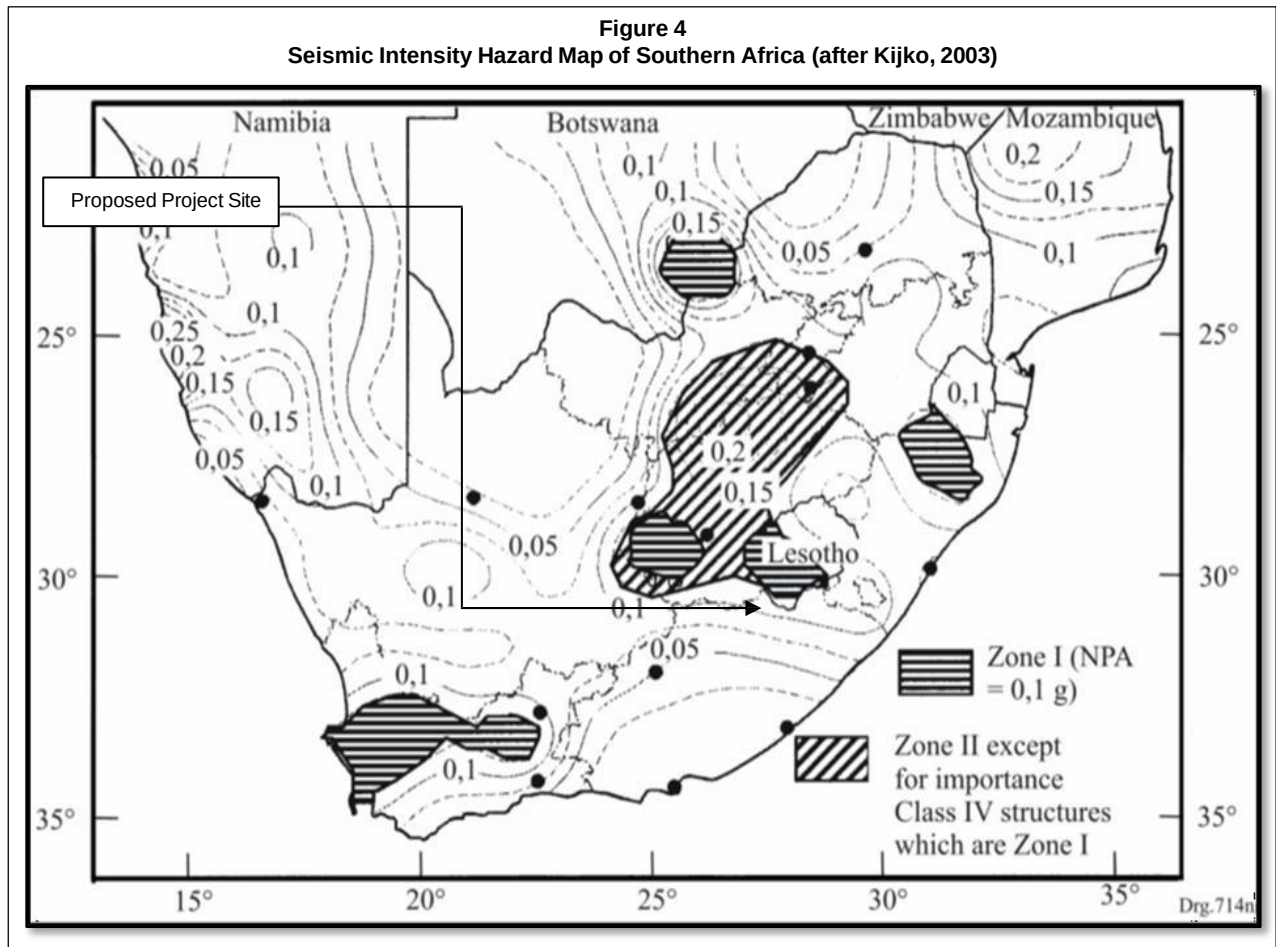
3 GEOHAZARDS

3.1 Seismic Hazards

Generally, there are two types of seismic activities that occur in South Africa, namely.

- Regions of natural seismic activity (Zone I)
- Regions of mining-induced and natural seismic activity (Zone II).

The seismic zones are determined from the seismic hazard map (Figure 4) which represents peak ground acceleration with a 10% probability of being exceeded in a 50-year period and this includes both natural and mining-induced seismicity. The site falls within Zone II, as shown in Figure 4.



Both the seismic hazard zones map shows that the site is situated in an area where the peak ground acceleration with a 10% probability of exceedance in a 50-year period is approximately between 0.08 to 0.12 m/s². Although the effects of these events are much less serious than those caused by larger earthquakes, extensive damage has been reported in the past, Bashan Consulting is of the opinion that the proposed project site is situated outside of the seismic risk areas and as such, no provision must be made for seismic loading in the design of the structures and/or foundations.

3.2 Ground Subsidence

Subsidence occurs in areas with large underground cavities typically resulting from large scale shallow to very shallow mining and from dolomite/limestone dissolution. It may also appear where thick deposits of unconsolidated material exist. No signs of previous subsidence were evident during the site investigation and no mining activity has occurred in this area.

3.3 Sinkhole Formation

Like subsidence, sinkhole formation occurs in areas with very large to extremely large underground cavities resulting from poorly designed shallow underground activities. Dissolution of dolomites or limestones over millions of years, may lead to cavity formations which later manifest as sinkholes. The available geological map and field observations indicate that the site is not underlain by dolomite.

3.4 Slope Stability and Erosion

No evidence of erosion (such as gulleys and erosional channels) was observed on site during the site investigations and Bashan Consulting is of the opinion that the materials found at the project site have a very low erodibility potential. During flood events, however, local creeks may serve as drainage channels for storm water resulting in erosion of soft sediment and bedrock.

3.5 Landslides and Mudslides

The probability of landslides and mudslides occurring within this area is remote. This is primarily due to the low relief and the relatively shallow gradient along the road alignment.

3.6 Rockfalls and Rockslides

The probability of the occurrence of rockfalls and rockslides is low due to the low relief and shallow gradient.

4 EVALUATION PROCEDURES USED IN THE INVESTIGATIONS

4.1 Test Pit Excavations

The investigation comprised a desktop study, visual assessment of the site, excavation of test pits, dynamic probing and laboratory testing. A literature review of the available data and assimilation of other information (as part of the preliminary investigation).

The field work was conducted on the 16th of September 2024. A total of six (06) test pits were excavated across the site. The test pits were backfilled and compacted in layers. The test pit collars (coordinates and excavated depth) are provided in Table 4.1_1 whilst the test pit collar locality plan is shown in Figure 5.

Table 4.1_1 Test Pit Collar Information						
Test Station	Latitude	Longitude	Total Depth (m)	Shallow Seepage (m)	Refusal (Y / N)	Reason For Termination / Excavation Stoppage
TP01	30°41'28.93"S	26°39'47.32"E	3.00	-	N	Target depth of 3.00 m reached
TP02	30°41'24.62"S	26°39'51.30"E	3.00	-	N	Target depth of 3.00 m reached
TP03	30°41'29.40"S	26°39'53.19"E	3.00	-	N	Target depth of 3.00 m reached
TP04	30°41'30.37"S	26°39'47.45"E	3.00	-	N	Target depth of 3.00 m reached
TP05	30°41'22.19"S	26°39'53.58"E	3.00	-	N	Target depth of 3.00 m reached
TP06	30°41'24.59"S	26°39'57.42"E	3.00	-	N	Target depth of 3.00 m reached

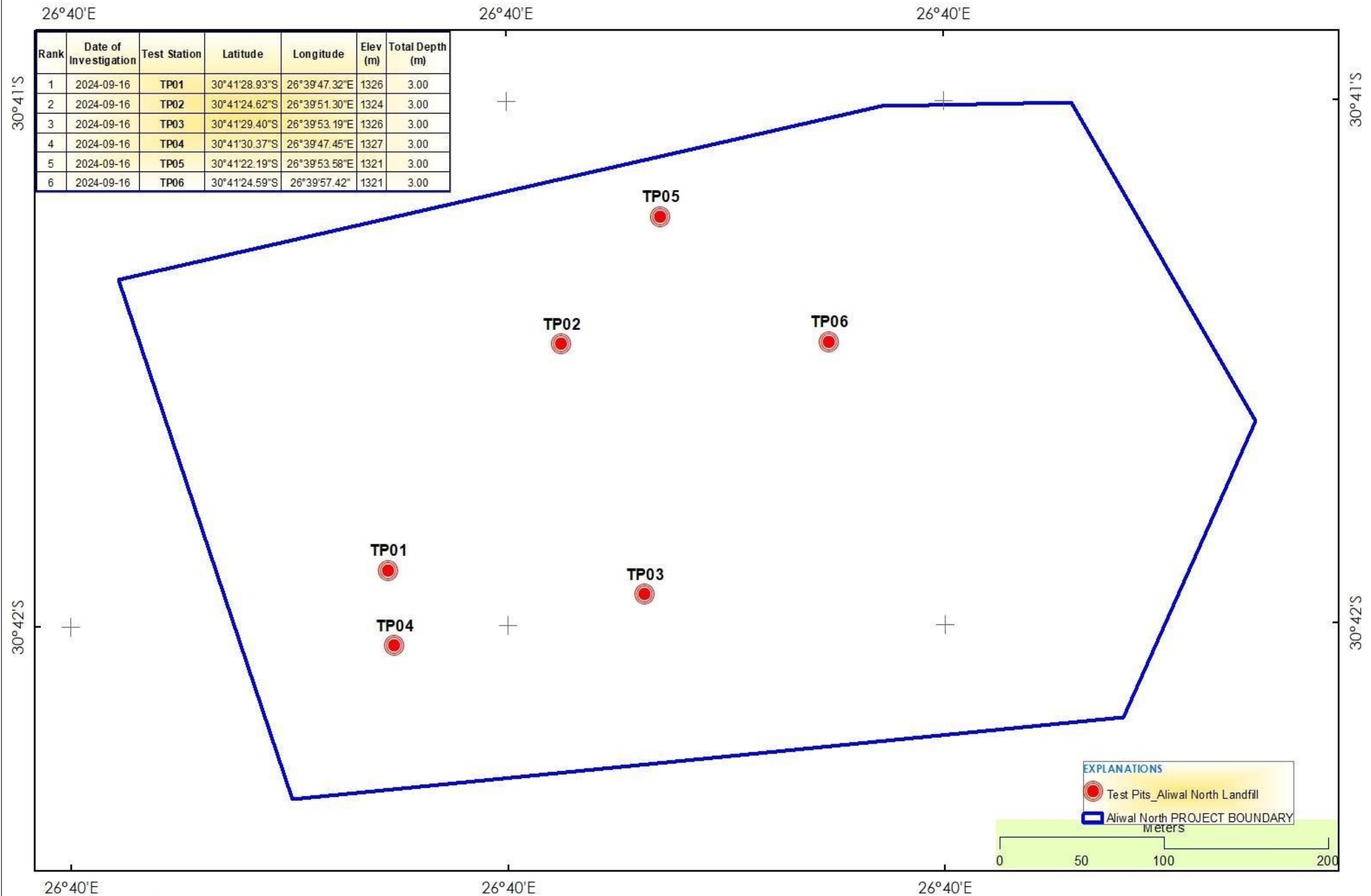
The mode of excavation was via mechanical methods (utilising a hired TLB) and excavating to a target depth of 3.00 m (or at the interception of the country bedrock layer and/or to refusal or flooding). In conducting this excavation, emphasis was placed on accessing the soil materials found inside the test pit, conducting in-situ profiling, soil sampling, slope stability, visual assessment of groundwater seepage, conducting a detailed study of the test pit sidewalls and the appraisal of the Excavatability.

The test pit sidewalls were critically studied along the vertical face of the excavation and an in-situ soil profiling exercise conducted in line with the guidelines promulgated by Jennings et al (1973) and Brink, A.B.A. and Bruin, R.M.H. (editors); 2001; Guidelines for Soil and Rock Logging in South Africa.

4.2 Dynamic Cone Penetrometer

Dynamic Cone Penetrometer (DCP) tests were conducted adjacent to each test pit along the pipe centreline to obtain an indication of the in-situ material consistencies and CBR values for the subsoil. The DCP test is conducted by driving a 60° steel cone, 20 mm in diameter, into the ground by an 8kg hammer falling through 575 mm. The penetration resistance is expressed as a penetration rate (mm) for every 5 consecutive blows. The test is used as a guide in compaction control. DCP tests were conducted adjacent to each test pit. Measurements were taken at depths varying from the surface up to 1.0 m or refusal below ground level.

Figure 5
Test Pit Locality Plan



PROPOSED ALIWAL NORTH LANDFILL SITE

TEST PIT LOCALITY PLAN

1:2,000
A3 PAPER SIZE



MAP DATUM
WGS 84
(ZONE 35 S)

DATE
2024-09-30

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5 SITE GEOLOGY AND GROUNDWATER

Based on the 1:250 000 Geological map of the Aliwal North region (sheet 3026), the oldest rocks at the site are assigned to the Tarkastad Subgroup (Trt), a stratigraphic unit within the Beaufort Group of the Karoo Supergroup (Table 5.1_1). An extract of the Geological map is shown in Figure 7.

Table 5.1_1 Regional Geological Stratigraphy of the Aliwal North Area					
Supergroup	Group	Subgroup	Formation	Strat Code	Rock Type
					Alluvium
					Sand; Dune sand; Dune rock
			Mgazana	Kmb	Conglomerate (dark greenish-grey); Sandstone
			Karoo Dolerite	Jd	Dolerite; Gabbro; Picrite
	Drakensburg			Jdb	Volcaniclastic rocks
KAROO			Clarens	TRc	Sandstone (pale orange to pink, fine-grained)
			Elliot	TRe	Mudstone (brownish-red and grey); Sandstone
			Molteno	TRm	Mudstone (grey); Shale, Sandstone (gritty); Occasional coal seams
	Beaufort	Tarkastad (TRt)	Burgersdorp	TRb	Mudstone (brownish red); Sandstone (grey)
			Katberg	TRk	Sandstone; Mudstone (brownish-red and grey)
		Adelaide		Pa	Mudstone (grey and brownish red); Sandstone (yellow and grey, fine-grained)
	Ecca			Pe	Shale (dark grey); Mudstone; Sandstone
			Dwyka	Pd	Tillite
NATAL				Pp	Quartzitic Sandstone (light grey)

The Trt lithologies are comprised of a brownish-red to greenish-grey mudstone with intercalated fine to medium-grained, Sandstone. In certain localities, it is common to intersect the upper Molteno Formation (Trm) characterised by grey mudstones and shales occurring in 1 to 10 m layers and also often intercalated by a subordinate, grey, Sandstone that occurs in 1 to 2 m thick layering.

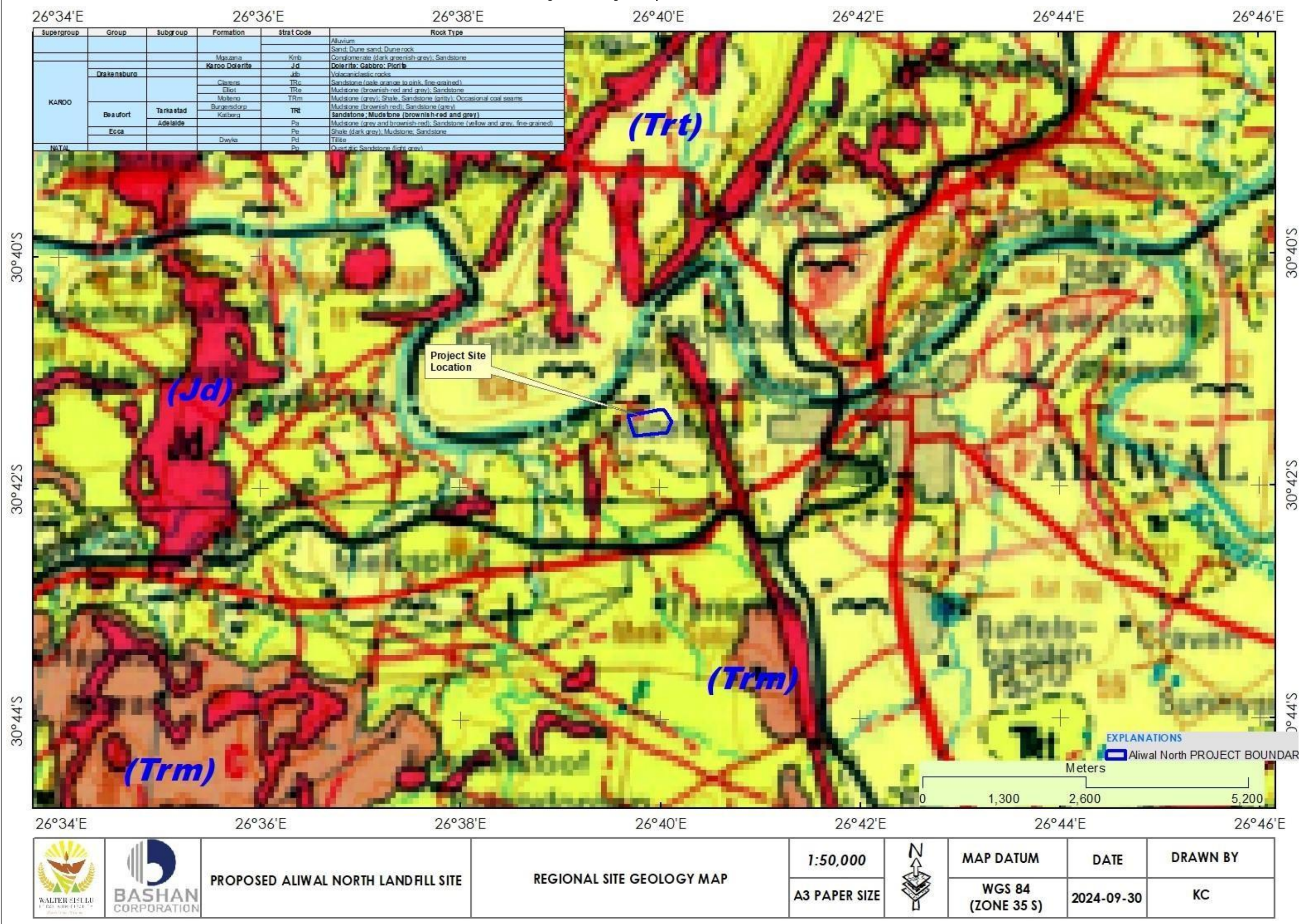
Dolerite intrusions (Jd) in the form of sills, sheets and dykes cover the upper parts of the project site. On a regional scale, dolerite occurs as a network of dykes, sills or sheets intruded into the sedimentary strata along the bedding planes and fractures.

Younger, unconsolidated sediment deposits of Quaternary age (Q) consisting of a discontinuous cover of alluvium cover the residual soils and the solid bedrock, often occurring in pockets and along streams. Sporadic deposits of river terrace gravels that locally could signify the presence of the reworked, Basement Complex and comprising of a variable mixture of unconsolidated deep coloured, silty to gravelly-sand intermixed with sub-rounded to sub-angular fragments of quartzite can be observed locally. Occasional slickenside surfaces are present and joint surfaces (when present) are often stained brown and may have preserved within them, fine rootlets.

5.1 Groundwater

The water-holding capacity of construction materials (such as soil) is dependent on a range of engineering properties including particle size distribution, clay type present, amount of organic matter, bulk density and structure of the soil. Groundwater movement within the project site is controlled by geological and / or structural factors. Groundwater seepage was not intercepted during the current geotechnical investigations on site. It should be noted that the fact that some slightly moist to moist conditions were observed during the investigation indicates that groundwater seepage should be anticipated at shallow depth across the project site and precautionary measures must be incorporate into design parameters.

Figure 6
Regional Geological Map of the Lusikisiki Area



5.2 Site Soils Overlying Bedrock Geology

Table 5.2_1 shows a summarised table of the intercepted soil layer materials across the proposed mine sites. Detailed soil profiling information is depicted in Appendix A attached to this Report.

Table 5.2_1 Summarised Soil Profiling Results										
RANK	TEST PITS	TRANSPORTED SOILS						RESIDUAL SOILS		
		HILLWASH			COLLUVIUM			RESIDUAL SOIL		
		From (mm)	To (mm)	Run (mm)	From (mm)	To (mm)	Run (mm)	From (mm)	To (mm)	Run (mm)
1	TP01	0	620	620	620	1700	1080	1700	3000	1300
2	TP02	0	880	880	-	-	-	880	3000	2120
3	TP03	0	2800	2800	-	-	-	2800	3000	200
4	TP04	0	600	600	-	-	-	600	3000	2400
5	TP05	0	600	600	-	-	-	600	3000	2400
6	TP06	0	640	640	-	-	-	640	3000	2360

5.2.1 Hillwash

The upper soil layer found across the site is comprised of an undifferentiated topsoil of Hillwash materials defined as a dry to slightly moist, brown to black, medium dense, intact, Silty Clayey SAND often intercalated with some gravelly materials and accompanied with some organics in the top layers. This material layer is intercepted from the existing ground surface to a maximum depth of 2 800 mm and an average depth of 1 023 mm (below the existing ground level).

5.2.2 Colluvium

The second layer found on site is comprised of a dry, orangish brown to brown, loose to medium dense, intact, Silty Clayey SAND, also accompanied by organics. This material was intercepted at a single site (TP01) between the depth of 620 to 1 700 mm (below the existing ground level).

5.2.3 Residual Soil

The residual soil layer is the soil horizon where both the macro and micro-structures inherited from the parent rock remain visible. This soil layer was intercepted at all points of investigation at the project site between 600 – 3 000 mm. It is described as a dry to slightly moist, olive to yellowish-brown, medium dense, intact, Silty SAND.

6 ENGINEERING MATERIALS TESTING RESULTS

6.1 Results of the Dynamic Cone Penetrometer

The results of the DCP sounding survey indicates that the soil layers on site are mainly comprised of a triple, pan-cake, sublayer stratigraphy comprised of a loose to medium dense layer in the upper horizons with an average CBR value of 12. The middle soil layers are associated with a loose to medium dense, soil layer with an average in-situ CBR of 11 whilst the lowest soil layers have a rather medium dense to dense consistency with an average CBR value of 22.

The estimated maximum bearing capacity of the site soils ranges between 64 to 162 kPa (with an average of 98 kPa) and is thus estimated not exceed 100 kPa in the lowermost horizon. Summarised results are shown in Table 6.1 _1 while detailed DCP results are included in Appendix B.

6.2 Foundation Indicators

Disturbed soil samples, considered to be representative of the materials on site were subjected to road indicator testing (as per SANS 3001 test methods). Foundation Indicator tests to determine the to determine the soil grading, Atterberg limits and potential expansiveness of the founding soils. The grain size analysis is widely used in classification of materials and determination of the engineering behaviour of materials. The distribution of different grain sizes affects the engineering properties of materials. Grain size analysis provides the grain size distribution, and it is required in classifying the material. Particle size grading of the soils or sieve analysis is the easiest way to describe the relative amounts of gravel, sand, silt, and clay within the soil. A summary of the foundation indicator test results is shown in Table 6.2 _1 whilst detailed test results are depicted in Appendix C attached to this Report.

The potential expansiveness of the soil materials encountered on site was interpreted using the method suggested by Van der Merwe (1964), as revised by Savage (2007). Under the above criteria, a clayey soil material with $PI < 12\%$ is normally regarded as having low expansiveness; $12\% > PI < 23\%$ as medium expansiveness, and a $PI > 23\%$ as high expansiveness.

6.3 Moisture / Density Relationships and Compaction

These tests are used to test for the parameters that often yield the results of the bulk density, and the compaction effort required on site to achieve the desired soil density on disturbed soils that are compacted in-situ. Laboratory tests to determine the moisture / density relationship as well as the compaction characteristics at modified AASHTO compaction was completed. The test results are summarized in Table 6.3 _1 whilst detailed test results are depicted in Appendix D attached to this Report.

These tests often yield values of the optimum moisture, the moisture-density relationships, and the California Bearing Ratio (CBR) or capacity values. This test method provides for the determination of the CBR values of a material at an optimum water content and/or a range of water content from a specified compaction test and a specified dry unit weight. The suitability of the near surface soil materials on site for infrastructure construction purposes can be inferred from the Committee of Land Transport Officials (COLTO) and the Technical Recommendations for Highways (TRH14) soil classifications

Table 6.1_1 Estimated CBR Summaries and Layer Intercepts Interpreted from the DCP Survey							
Layer	Test Pit	TP01	TP02	TP03	TP04	TP05	TP06
LAYER 1	Start Depth (mm)	0	0	0	0	0	0
	Boundary Depth (mm)	918	700	722	580	463	790
	Layer Depth (mm)	918	700	722	580	463	790
	Material Consistency	Medium Dense	Loose to Medium Dense	Medium Dense to Dense	Medium Dense to Dense	Medium Dense to Dense	Medium Dense to Dense
	Average In-situ CBR	12	6	13	15	13	12
LAYER 2	Estimated Maximum Bearing Strength (kPa)	73	50	71	80	74	71
	Start Depth (mm)	918	700	722	580	463	790
	Boundary Depth (mm)	1 388	1430	1 702	980	770	1 450
	Layer Depth (mm)	470	730	980	400	307	660
	Material Consistency	Medium dense	Loose	Loose to Medium Dense	Dense	Medium Dense to Dense	Medium Dense to Dense
LAYER 2	Average In-situ CBR	8	4	6	20	22	3
	Estimated Maximum Bearing Strength (kPa)	60	39	47	685	102	36
	Start Depth (mm)	1 388	1 430	1 702	980	770	1 450
	Boundary Depth (mm)	1 988	1 990	1 732	2 000	1 981	1970
	Layer Depth (mm)	600	560	30	1 020	1 211	520
LAYER 2	Material Consistency	Loose	Medium Dense to Dense	Medium Dense to Dense	Loose to Medium Dense	Loose to Medium Dense	Loose to Medium Dense
	Average In-situ CBR	4	10	31	71	8	5
	Estimated Maximum Bearing Strength (kPa)	38	67	118	61	56	43

Table 6.2_1 Foundation Indicators – Summarised Laboratory Results															
Trial Pit	From (mm)	To (mm)	Material Description	Clay (%)	Silt (%)	Sand (%)	Gravel (%)	Wtd PI (%)	LL (%)	PI (%)	LS (%)	Potential Expansion	Surface Heave (mm)	Vertical Heave (mm)	USSC Class
HILLWASH SOIL LAYER															
TP01	0.00	0.62	Clayey Silty SAND	11.7	20.8	67.4	0.1	6.0	30	6	3	Low	-	-	-
TP05	0.00	0.60	Silty Clayey SAND	19.8	15.0	65.1	0.1	6.0	25	6	3	Low	-	-	CL & ML
TP03	0.00	2.80	Clayey Silty SAND	19.6	19.7	60.1	0.6	10.9	20	11	6	Low	-	-	-
COLLUVIAL SOIL LAYER															
TP01	0.62	1.70	Silty Clayey SAND	24.2	17.3	56.4	2.1	11.7	20	12	5	Low	-	-	-
RESIDUAL SOIL LAYER															
TP02	0.88	3.00	Silty SAND	6.6	15.7	77.7	0.1	-	-	NP	-	Low	-	-	-
TP03	2.80	3.00	Silty Clayey SAND	18.2	17.5	60.6	3.6	9.2	29	10	4	Low	-	-	CL

Table 6.3_1 Compaction Rating – Summarised Laboratory Results																			
Trial Pit	From (mm)	To (mm)	Material Description	GM	LL (%)	PI (%)	LS (%)	OMC (%)	MDD (kg/m³)	Comp (MC) Rating	Swell (%)	CBR @ 100%	CBR @ 98%	CBR @ 97%	CBR @ 95%	CBR @ 93%	CBR @ 90%	HRB Class	TRH14 Class
HILLWASH SOIL LAYER																			
TP01	0.00	0.62	Clayey Silty SAND	0.30	30.0	6.0	3.0	14.5	1569	14.2	1.65	2	2	2	2	2	1	A-6 (7)	-
TP03	0.00	2.80	Clayey Silty SAND	0.30	20.0	11.0	6.0	15.3	1769	15.2	1.83	6	5	4	3	3	2	A-6 (7)	-
COLLUVIUM SOIL LAYER																			
TP01	0.62	3.00	Silty Clayey SAND	0.40	20.0	12.0	5.0	14.7	1864	14.7	0.72	3	2	2	2	2	1	A-6 (7)	-

7 GEOTECHNICAL EVALUATION

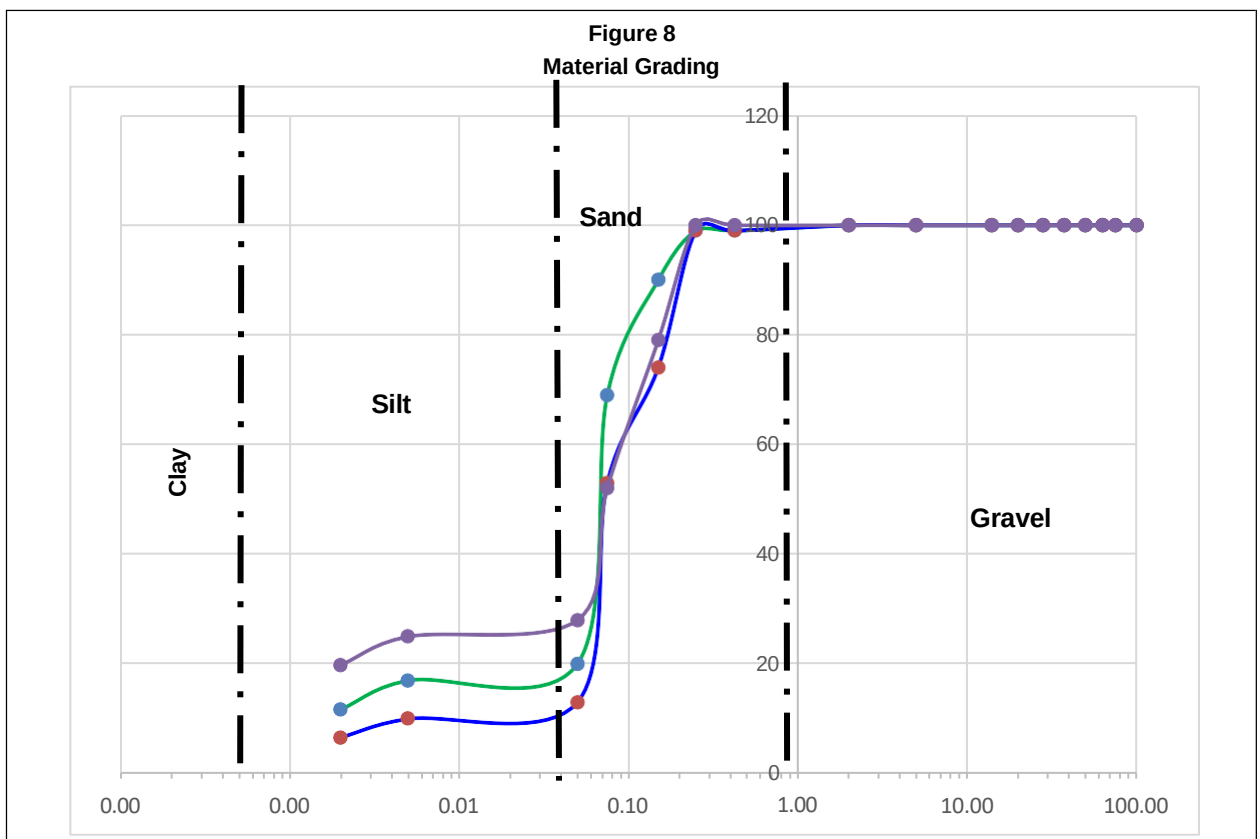
The engineering characteristics associated with the types of soil materials prevalent on site include a low to marginally medium, collapsible structure, a low to moderate compressible grain structure, heaving and swelling soils, foundation settlement and a potentially, low bearing capacity in certain localised, upper pockets. Our findings are based on a set of final data interpretations and on our experience of working in the region.

7.1 Site Topography

The topography at the proposed construction site is characterised as a flat terrain rolling gently towards the east. The project site is characterised by elevations of between 1318 m (in the east) to 1327 m above mean sea level towards the west with a slope gradient of approximately 3 %. m above mean sea level (amsl). The general slope gradient is approximately less than 2% and it is not expected to cause adverse calamities at the proposed construction site.

7.2 Material Grading

The grain size analysis is widely used in classification of materials and determination of the engineering behaviour of materials. The distribution of different grain sizes affects the engineering properties of materials. Grain size analysis provides the grain size distribution, and it is required in classifying the material. Particle size grading of the soils or sieve analysis is the easiest way to describe the relative amounts of gravel, sand, silt, and clay within the soil. The test results are summarized in Figure 8.



The predominant coarse soil fraction ranges between 52 % and 91 % and the subsoils classify either as inorganic silts, micaceous or diatomaceous, fine, sandy or silty soils (CL) or as inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity (ML) in terms of the unified soil classification system.

The soils display a low liquid limit ranging between 20 - 30% and a plasticity index ranging between 6 – 11.7 %. These materials are anticipated to exhibit low to medium potential expansiveness (heave and shrinkage with changes in moisture content) in terms of the Van der Merwe method of heave prediction.

7.3 Material Characteristics

7.3.1 Hillwash

The general visual inspection of the Hillwash indicates a potentially moderate to high compressibility of the soils (if subjected to various saturated conditions). These soils are also associated with a low to medium, collapsible grain structure within the soil matrix is likely to occur.

The Hillwash material is generally described as a Clayey Silty SAND (CSS) with a low, weighted plasticity index ranging between 6 to 11 %. This material has returned a low weighted plasticity index and was thus not subjected to the heave calculations using the method suggested by Van der Merwe (1964), as revised by Savage (2007).

The laboratory results indicates that the potential volume change (expansiveness) of the colluvium soil material found on site is expected to be low. However, precautionary measures during the design phase will have to be considered to avoid geotechnical problems.

The moisture / density test yields a maximum dry density averaging 1 669 kg/m³ at an average optimum moisture content of 14.9%. The average CBR value is 3 and 2 at compaction densities of 93% and 95% Modified AASHTO, respectively. The Hillwash soil materials found on site classify as A-6 (7) on the HRB classification system and although these soils should offer excellent to good subgrade rating (if compacted to the right density), we are of the view that as a precaution, the reduction of the in-situ material volume due to collapse must be included into the calculation of the required volumes of construction materials. As a rule, it is recommended that the Hillwash soil material should be spoilt.

7.3.2 Colluvium

Colluvium on site is generally described as Silty Clayey SAND (SCS) with a low, weighted plasticity index of 11.7 %. This soil layer had returned a low weighted plasticity index and was thus not subjected to the heave calculations using the method suggested by Van der Merwe (1964), as revised by Savage (2007).

The laboratory results indicates that the potential volume change (expansiveness) of the colluvium soil material found on site is expected to be low. However, precautionary measures during the design phase will have to be considered to avoid geotechnical problems.

The moisture / density test yields a maximum dry density averaging 1 864 kg/m³ at an average optimum moisture content of 14.7%. The average CBR value is 2 at both compaction densities of 93% and 95% Modified AASHTO, respectively. The colluvium soil materials found on site classify as ranging between A-6-7 on the HRB classification system and should offer excellent to good subgrade rating.

7.3.3 Residual Soil

The Residual Soil material on site is generally described as generally ranging between a Silty SAND (SS) to a Silty Clayey SAND (SCS) with a low, weighted plasticity index ranging between being non plastic to a high of 9.2 %. This soil material layer on site returned a low, weighted plasticity index and was thus not subjected to the heave calculations using the method suggested by Van der Merwe (1964), as revised by Savage.

The laboratory results indicates that the potential volume change (expansiveness) of the colluvium soil material found on site is expected to be low. However, precautionary measures during the design phase will have to be considered to avoid geotechnical problems.

7.4 Collapsible Soil

Collapsibility can occur in any open-textured, clayey to silty sandy soil having a high void ratio and a generally low in-situ dry density (<1 600 kg/m³). If water is added to such soils beyond the optimum moisture content, the water will occupy the extra space since there is no air volume and dry density will reduce.

It was not possible to collect any undisturbed soil samples during the current site investigations. The fact that some soil materials in the upper section of the stratigraphy at TP01 returned an in-situ dry density of less than 1600 kg/m³ is recipe for concern as these materials would offer high collapsibility if not spoilt prior to the commencement of the construction phase. In addition, the general field observations point to the presence of collapsible soils with a low to moderate collapsibility potential.

7.5 Soil Compressibility

A compressible soil is one that, when subjected to direct actions, undergoes a gradual settlement as volume changes occur. Soil compressibility is assessed via a detailed interpretation of soil profiles, laboratory consolidation tests (on undisturbed block samples) and via in-situ penetration (such as DCP) tests. Based on the field interpretations, the upper soil layers indicate a potentially low to moderate compressibility (if subjected to various saturated conditions).

7.6 Active Clays and Potential Heave

Heaving is the process that accompanies the wetting of active soils above a deep-water table with a change in volume. The activity of a soil (i.e., its susceptibility to volume change upon changes in the moisture content) is the ratio of the plasticity index (PI) to the clay fraction. Substantial shrinkage (the reverse process) can occur when the water table is lowered under drought conditions, a scenario that is likely to produce differential settlement in buildings. A structure constructed over expansive soils is likely to experience engineering failure as volume changes can cause extensive structural deformation and/or cracking of the structure.

The laboratory results indicate that the potential volume change (expansiveness) of the soil materials found on site is expected to be low. However, precautionary measures during the design phase will have to be considered to avoid geotechnical problems.

7.7 Sidewall Stability

Sidewall excavation stability which refers to the competence of the sides of the walls of the test pits at the site was evaluated on site. The sidewalls of the excavated test pits had remained stable after the pit excavation. However, care must be exercised during the construction phase of the project. All safety precautions should be adhered to during the construction phase as unstable sidewall conditions could be expected in areas where the material is moist, and the excavations are deeper than 1 500 mm. Bashan Consulting recommends that all site contractors should constantly assess the sidewall safety on site and will have to provide shoring (where necessary).

7.8 Differential Settlement

The potential of experiencing soil settlement due to the consolidation of soils resulting from imposed loads is low. However, differential settlement is expected to increase if foundations were made across soil and rock boulders in which case, specific site consideration and monitoring will be required.

7.9 Excavatability and Rippability

Difficulty of excavation or Excavatability generally occurs where shallow bedrock and/or perorates are encountered within the top 1.5 m depth of the soil profile or where TLB refusal occurs (whichever is first). Table 7.9_1 shows the site's Excavatability and rippability assessment classifications.

The ground surface should be considered as having “soft” to “medium” excavation conditions. In areas where a weathered bedrock of mud rocks and/or dolerite will be intercepted during the excavation phase, it is highly likely that additional costs will be incurred to rip the bedrock.

Table 7.9_1
Excavatability and Rippability Assessment

From (mm)	To (mm)	Rippability Assessment	Material Hardness	Description
0	1 700	Easy ripping	Soft	☐ Easy excavation by pick and spade. Material can easily be ripped and excavated with a TLB.
1 700	3 000	Moderate ripping	Medium	☐ Cannot be excavated by pick and spade. ☐ Excavation will be slow with TLB, and machine will likely refuse (ca. 3 000 mm) into hard mud rock.

8 RECOMMENDATIONS AND SOLUTIONS

8.1 Potential Geotechnical Constraints

The variable engineering impacts associated with the soil materials prevalent across the proposed construction site are summarised in Table 8.1_1.

Table 8.1_1 Potential Geotechnical Constraints of the Proposed Construction Site	
Constraint	Explanation
Settlement of foundations and structures	☐ Quaternary deposits are poorly consolidated and unlithified. ☐ These deposits may settle if unsupported or overloaded. ☐ The presence of localised, un-engineered fill material may also result in a differential settlement hazard.
Collapse	☐ Silty sands, sand and sandy gravels are potentially compressible and collapsible. ☐ The nature of the soil materials is potentially susceptible to compressibility and collapse.
Slope stability	☐ Residual soils from the weathered bedrock and/or dolerite can be susceptible to shallow soil failure (soil creep, slumping, collapse) in areas of increased gradient.
Bearing capacity	☐ The bearing capacity within the soft, silty clayey soils could result in the loss of stability or potential collapse of structures during the construction phase of the development and proper considerations of the bearing capacity of the foundations is critical.

Residual soils derived from fine-grained rocks of the Karoo typically form silty sands that may exhibit a collapsible grain structure. These soils may competently bear imposed loads when dry but when inundation occurs and the soil is under load the colloidal 'bridges' in these voided soils fail and the soil typically collapses into a denser state, leading to differential settlement. The differential settlement may result in structural damage. Cracking of buildings in corners is often observed in buildings built in areas with collapsible soils.

The variable engineering impacts associated with the types of soil materials prevalent on site include the presence of a low to moderate compressibility potential, a collapsible grain structure, an uneven bedrock surface and a potentially, low bearing capacity in certain pockets.

8.2 Design Parameters

Based on the results of the geotechnical investigations conducted, the summarised project design parameters materials are shown in Table 8.1_1. Figure 9 shows the stratigraphic correlation of the soil materials available on site.

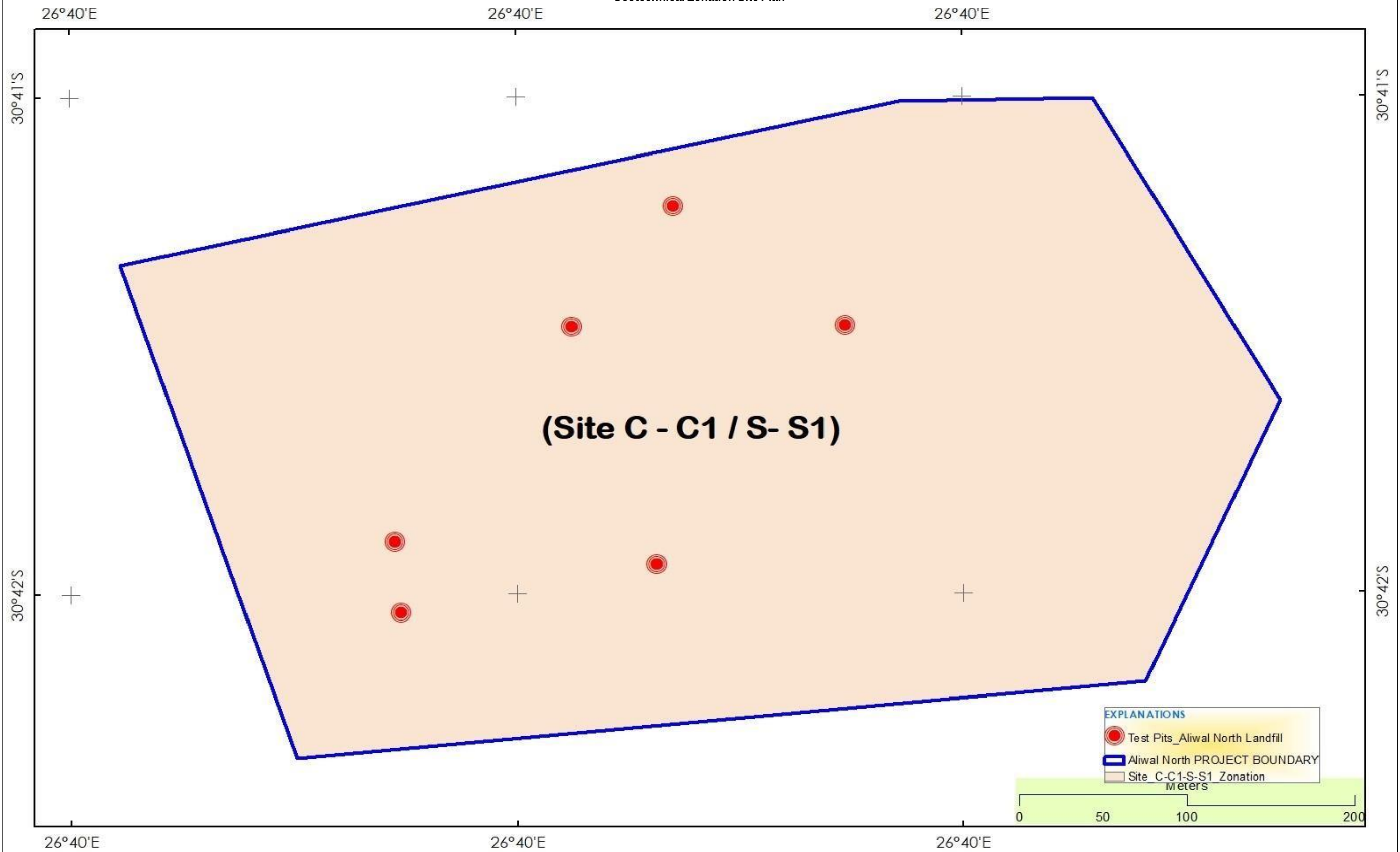
Table 8.2_1 Project Design Parameters					
Layer Number	Soil Horizon	From Depth (mm)	To Depth (mm)	TRH14 Class	Allowable Bearing Capacity (kPa)
1	HILLWASH	0	1023	ND	50
2	COLLUVIUM	1023	1203	ND	80
3	RESIDUAL SOIL	1203	3 000	ND	100

ND: Not Determined

8.3 The Geotechnical Site Class

The generated geotechnical data indicates that the proposed sites fall within an area that relates to the most favourable site and where normal development can take place without considering precautionary measures. Based on the available geotechnical data, the recommended geotechnical site class at the proposed construction site has been determined to be **C - C1 / S – S1** (across the entire site). The geotechnical zonation plan is shown in Figure 9.

Figure 9
Geotechnical Zonation Site Plan



 WALTER SISULU UNIVERSITY	 BASHAN CORPORATION	PROPOSED ALI WAL NORTH LANDFILL SITE	SITE GEOTECHNICAL ZONATION PLAN	1:2,000		MAP DATUM	DATE	DRAWN BY
				A3 PAPER SIZE		WGS 84 (ZONE 35 S)	2024-09-30	KC

Table 8.3_1 Interpreted Geotechnical Site Class Zonation Classes				
Terrain Area	Geotechnical Site Class	Geotechnical Characteristics of Founding Material	Estimated Total Soil Movement (mm)	Construction Methodology
TP01-TP06	C – C1	Compressible and collapsible soils	0 - 10	☐ Compaction of in-situ soils below individual footings ☐ Reinforced concrete ground beams or solid slabs on piled or pier foundations ☐ Stiffened strip footings, stiffened or cellular raft
	S – S1	Compressible soils	10 - 20	

8.4 Development Recommendations

A detailed foundation design falls outside the scope of the current geotechnical investigations. It should be noted that all larger, commercial, or multi-storey structures require site-specific design parameters that are not catered for via the current shallow pitting (and the accompanying routine soil testing) as performed. The choice of foundation type and the design of the foundations will need to be done by a competent structural engineer once the final structural layouts have been determined.

Conformance to the relevant industry guidelines in respect of foundation design and building procedures should be observed prior to the commencement of the construction phase. Foundation earthworks should be constructed as directed by the project engineers and in accordance with SABS 1200D, COLTO 3300 or other applicable standards.

8.5 Foundation Assessment

Bashan Consulting highly recommends the removal of the upper cohesionless, un-engineered, Hillwash prior to the commencement of the construction stage. All topsoil to be stripped to spoil to a depth of 300 mm below existing ground surface and foundations can be laid at a depth of 600 mm.

Precautions limited to the compaction of founding soils to at least 93% of Modified AASTHO density to a depth of 1.5 times the foundation width, light reinforcement in foundations and additional stormwater drainage, service and plumbing precautions will assist in the safe delivery of the project. Compaction should be carried out using a trench rammer / wacker when the soil is in a slightly moist state. An estimated allowable bearing pressure of 70 kPa (with a subgrade modulus of 35kPa) can be assumed in such conditions.

Remove any incompressible or wet soils (in the upper soil layers) and replace with imported G7 material or suitably drier organic-free, in-situ soil (or as directed by the resident engineer). Backfill above the foundations should be compacted to the same density as the remainder of the terrace to avoid zones of high porosity and permeability around the foundations. Compaction should be carried out using a trench rammer / wacker when the soil is in a slightly moist state. An estimated allowable bearing pressure of 70 kPa (with a subgrade modulus of 35kPa) can be assumed in such conditions.

8.6 Proposed Foundation Design

Foundation earthworks should be conducted in accordance with SABS 1200D, COLTO 3300 or other applicable standards. A founding depth of 600 mm is recommended. The choice of foundation type and the design of the foundations will need to be done by a competent engineer once the final structural layouts have been determined. The foundation types recommended for the proposed site are as follows: -

- Spread footings (or Pad footings)
- Strip footings with light reinforcement.

8.6.1 Spread Footings

The proposed columns for the building structure should be founded on concrete bases / spread footings adequately reinforced and individually compacted. The excavations of foundation bases should be carried through to a residual layer at a depth of 1 000 mm below the current ground level and a width of $1.5 \times B$ (where B is the width of a footing).

Upon reaching the founding depth of 1 000 mm, the excavation base should be compacted in-situ to 93% MOD AASHTO at optimum moisture content. The foundation for the proposed structures could then be founded onto the residual compacted layer, adequately reinforced with steel and adopting a safe allowable bearing pressure of 150 kPa.

8.6.2 Strip Footings

The brick wall for the proposed structure should be founded on strip footings of typically 600 mm width and 250 mm thickness with light reinforcement at a depth of 600 mm below the surface. In-situ material should be removed and replaced with a suitable material compacted to 93% Mod AASHTO density at -1% to $+2\%$ OMC. Strip footings can then be placed on compacted in-situ soils utilizing an allowable bearing capacity of 100 kPa.

8.7 Foundation Development

Foundations should be laid on top of the surface of properly prepared fill. Proper compaction of the classified fill material in layers not exceeding 200 mm (where mechanical equipment is to be used) or 100 mm (where hand-held compactors are to be used) must take place for all foundation types. Compaction should be carried out at optimum moisture content (OMC) to 95% of the Mod. AASHTO density to a minimum depth of 300 mm below the underside of the foundation. Compaction should be carried out using a trench rammer / Wacker when the soil is in a slightly moist state. Foundation systems for heavier structures could include a 500 mm layer of imported G6 / G7 gravel material compacted to 95% of Mod AASHTO density below the footings to increase the allowable bearing pressures to 150 kPa. Compaction of in-situ soils below individual footings, coupled with deep strip foundations and / or soil raft is highly recommended.

For building installations, the recommended founding options for a double-storey building are reinforced concrete ground beams or solid slabs on piled or pier foundations (as the first option). Option 2 includes stiffened strip footings, stiffened raft / cellular raft. Alternative foundation systems for heavier structures could include a 500 mm layer of imported G6 / G7 gravel material compacted to 95% of Mod AASHTO density below the footings to increase the allowable bearing pressures to 150 kPa. Compaction of in-situ soils below individual footings, coupled with deep strip foundations and / or soil raft is highly recommended.

The brick wall for the proposed structure should be founded on strip footings of typically 600 mm width and 250 mm thickness with light reinforcement at a depth of 600 mm below the surface. In-situ material should be removed and replaced with a suitable material compacted to 93% Mod AASHTO density at -1% to $+2\%$ OMC. Strip footings can then be placed on compacted in-situ soils utilizing an allowable bearing capacity of 100 kPa. Backfill above the foundations should be compacted to the same density as the remainder of the terrace to avoid zones of high porosity and permeability around the foundations.

8.8 Terraces

The results of the compaction and indicator tests have been used to classify the in-situ soils to determine their suitability for use in the construction of terraces and pavement layers. Terraces and pavement layers requiring material classifications better than G8 will need to be imported from a commercial source or nearby borrow pit.

It is recommended that should terracing be required, a minimum quality G7 material be considered for the backfilling.

8.9 Floor Slab (Surface Bed)

The concrete floor slab (surface bed) with a light reinforcement (193 Ref Mesh) should be founded on a completed terrace compacted using G6 or G7 quality material of a minimum of 200 mm thickness with a subgrade reaction of 67 MPa / m.

8.10 Access Roads and Parking Areas

The design of all access roads and parking areas should take cognisance of the saturated CBR's existing materials on site. The laboratory test results indicate that the insitu soils generally characterise as ranging between G6 to G9 (on the TRH14 class). It is recommended that an allowance is made for the importation of a G7 upper selected layer to improve the road subgrade. The subbase and base course material will also have to be imported (as per normal procedure). The access road pavement should be laid on the newly constructed earthworks, approximately 300 mm thick:

- Roadbed treatment that shall include the excavation of the un-engineered fill and/or topsoil to a depth of 300 mm and then replacing these soils with natural sands compacted to a minimum 90% Mod AASHTO.
- Remove any incompressible or wet soils and replace with imported G7 material or suitably drier organic-free, in-situ soil (or as directed by the resident engineer).
- The recommended engineered fill (wherever required) should be G7 or a better quality) material placed in 150 mm thick layers and compacted to at least 93% of the maximum dry density at minus 2% optimum moisture content (OMC).
- Provision should be made for cable ducts beneath all internal site roads and should be placed at a depth with a minimum cover of 750 mm. Cable ducts placed at shallower depths should be adequately encased in concrete.

8.11 Materials Use

Cognizance should be taken of the potentially compressible, collapsible, and expansive nature of the soil horizons. It is recommended that should terracing be required, a minimum quality G7 material be considered for the backfilling.

8.12 Stormwater Management

The field investigations have revealed that there is no shallow seepage associated with the soil profiles on site. However, the fact that moist soil conditions were observed in most of the soils indicate that a shallow seepage (associated with a low water table) may pose challenges during the rainy days.

The most important factor in the stable development and operation of the road pavement is the control and removal of both surface and groundwater from the construction site. Bashan Consulting recommends the installation of good surface drainage measures on site. Precautionary measures will include the placement of storm water drainage systems. Special care must be taken to ensure adequate surface drainage to prevent the accumulation of water next to structures. Storm water diversion measures such as ponding pools are recommended to control peak flows during thunderstorms. Appropriate cut-off drains should be allowed during the construction phase of the project.

8.13 Slope Design

Sidewalls of excavation stations shall be battered back to 1 (vertical) or 0.5 (horizontal) distance. Should groundwater seepage be encountered, then sidewall seepage will necessitate battering of open trenches to 1 (vertical) to 2 (horizontal) to reduce possible block sliding due to groundwater uplift pressure. De-watering mechanism will be required should earthwork construction commence during the rainy seasons.

8.14 Construction Site Management

Site clearance of all organic material must be conducted prior to excavating and laying of the foundations. All existing subsurface utilities that are abandoned should either be fully grouted or removed, and all the trenches backfilled and compacted.

Cobbles and boulders larger than two thirds ($\frac{2}{3}$) of the layer thickness and all clayey or organic material must not be included in the fill material. Care should be exercised during the construction phase by removing all loose and clayey soils below the foundations. Any deleterious soil materials (such as uncontrolled fill) should be cut to spoil, unless or otherwise directed by the resident engineer. To stabilise the base of the excavation, deepening of the earthworks (coupled with the laying of a crushed rock at the base of the excavation) is recommended.

8.15 Treatment of Problem Soils

In assessing the mitigation measures of addressing the collapsible soils, the issue of shallow or deep cohesive soils at a site must be considered. The paramount objective of treating collapsible soils should be to stabilise these soils before construction. It is usually far much less expensive to solve the problem prior to the commencement of the construction phase than wait until after the project is built. This way, the maintenance costs are reduced, project operation is more efficient, and the life of a structure is extended.

8.16 Special Precautionary Measures and Construction Monitoring

Compaction control is a prerequisite, and all foundations should not span across residual soils and un-compacted fill, since fill that is not adequately compacted, can translate into significant differential settlement even with light loads and minimal total settlement of less 10 mm. Where this is unavoidable, the Design Engineer's final design parameters should accommodate both the potential total, as well as the differential settlement.

The ponding of storm water around the exterior of new buildings can be avoided by shaping the ground levels around the exterior to create a fall away from the concrete washing plants and constructing a 1 m wide concrete apron with a 10% fall away from the buildings. This will assist in maintaining ground moistures stable and minimising settlement.

The burial of any form of corrosive metal piping and/or tanks will require some form of galvanising (with the use bitumen coating to be considered). All the reinforcing material must be totally submerged in concrete to limit the moisture exposure that might cause steel corrosion leading to concrete spalling. In addition, the use of plastic (or PVC) pipes is highly recommended. Adequate rebar cover in buried concrete foundations is highly recommended to mitigate against rust. Standard HDPE or uPVC pipe products will be suitable, and all buried metallic valves and fittings should be powder-coated as a standard precaution.

8.17 Additional Investigations

No additional, geotechnical site investigations are considered necessary for the assessment of the subsoils. However, the Developer should consider commissioning a flood line investigation to ascertain the likelihood of a flood occurring due to the influence of the Orange River.

8.18 Construction Monitoring

It is recommended that all foundations be inspected by a Competent Person, prior to placing of any concrete in the ground. In addition, regular checks on the quality and compaction of the backfill to the foundations and terraces should be made.

9 CONCLUSIONS

9.1 Conclusions

This Geotechnical Site Investigations Report presents the findings of a materials investigation conducted for the sole purpose of drawing material characteristics to pave way for the establishment of a municipal landfill site near Aliwal North in the Free State Province. A landfill site (also known as a tip, dump, rubbish dump, garbage dump, or dumping ground) is a site for the disposal of waste materials. Although the development details remain scarce, it is envisaged that proposed development is likely to consist of a double volume steel frame structure which will be utilised as a recycling shelter.

Bashan Consulting has noted that at the prevailing site conditions, all indications are that all future conventional engineering and construction methods should be expected to proceed. The variable engineering impacts associated with the types of soil materials prevalent on site include the presence of a low to moderate compressibility potential, a collapsible grain structure, an uneven bedrock surface and a potentially, low bearing capacity in certain pockets. Isolated areas where the materials do not meet the required specifications will have to be determined at construction stage. This will have to be undercut to the depth of the layer and replaced with a suitable material or modified by means of stabilization.

Based on the available geotechnical data, the recommended geotechnical site class at the proposed construction site is C - C1 / S – S1 (across the entire site). Bashan Consulting highly recommends the removal of the upper cohesionless, un-engineered, Hillwash material from the site prior to the commencement of the construction phase. The foundation types recommended for a double volume steel frame structure which will be utilised as a recycling shelter at the proposed site are the Spread Footings (or pad footings) and / or the Strip Footings with light reinforcement. The concrete floor slab (surface bed) with a light reinforcement (193 Ref Mesh) should be founded on a completed terrace compacted using G6 or G7 quality material of a minimum of 200 mm thickness with a subgrade reaction of 67MPa / m.

For building installations, the recommended founding options for a double-storey building are reinforced concrete ground beams or solid slabs on piled or pier foundations (as the first option). Option 2 includes stiffened strip footings, stiffened raft / cellular raft. Alternative foundation systems for heavier structures could include a 500 mm layer of imported G6 / G7 gravel material compacted to 95% of Mod AASHTO density below the footings to increase the allowable bearing pressures to 150 kPa. Compaction of in-situ soils below individual footings, coupled with deep strip foundations and / or soil raft is highly recommended.

All pavement layers requiring material classifications better than G8 will need to be imported from a commercial source or nearby barrow pit. It is recommended that should terracing be required, a minimum quality G7 material be considered for the backfilling.

9.2 Discussions

The ground conditions described in this Report refer specifically to those conditions encountered inside of the test pits excavated at the site. It is highly likely that certain conditions that may be at variance with those discussed above could be encountered at the site in the future.

Bashan Consulting wishes to elaborate that this investigation was thorough (at the required level) and was in line with the Client's objectives). Whilst every effort has been made during the fieldwork phase of this investigation with the aim of identifying the various soil horizons (including the associated potential problems and their likely distribution), it is highly impossible to guarantee that isolated zones of poor material could have been missed. Any significant deviation from the expected geotechnical conditions should be brought to the author's attention (for comment).

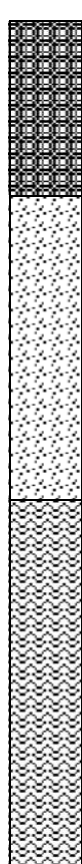
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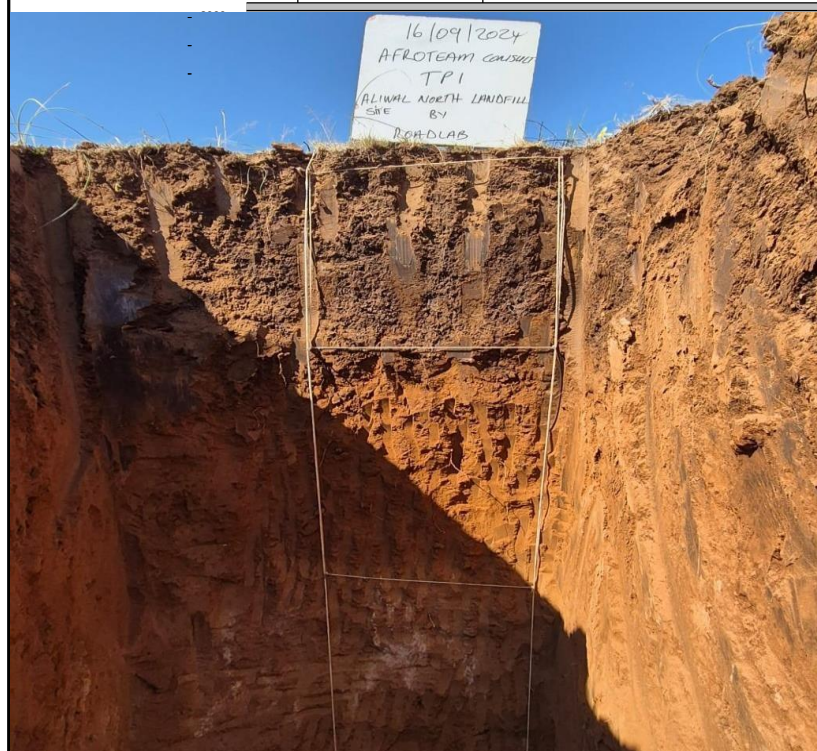
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Appendix A

Soil Profiles

TEST PIT	01	COORDINATES:	S30°41'28.93" E 26°39'47.32"	DATE PROFILED	16-09-2024
CLIENT:	AFROTEAM CONSULTANTS			STARTING DEPTH	0mm
PROJECT:	ALI WAL NORTH LANDFILL SITE (PROPOSED)			END DEPTH	3000mm

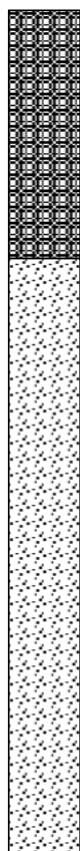
 Depth (mm)	0 - 620mm	Dry, brown to black, medium dense, intact, Silty Clayey SAND; Has some organics in the top layers, HILLWASH; TRANSPORTED SOIL	<u>DISTURBED SAMPLE</u> : Collected between 0 - 600 mm for Foundation Indicators; MoD & CBR
	620 - 1700mm	Dry, orangish-brown to brown, loose to medium dense, intact, Silty Clayey SAND; Contains some organics in the upper layers; COLLUVIUM, TRANSPORTED SOIL	<u>DISTURBED SAMPLE</u> : Collected between 620 - 1 700 mm for MoD & CBR
	1700 - 3000mm	Dry, olive to brownish-grey, medium dense to dense, intact, Silty SAND; nas some soil	



NOTES:

Engineering Geologist: K.C. Chimanya

TEST PIT	02	COORDINATES:	S 30°41'24.62" E 26°39'51.30"	DATE PROFILED	16-09-2024
CLIENT:	AFROTEAM CONSULTANTS			STARTING DEPTH	0mm
PROJECT:	ALI WAL NORTH LANDFILL SITE (PROPOSED)			END DEPTH	3000mm



Depth (mm)

0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
2200
2300
2400
2500
2600
2700
2800
2900
3000
3100
3200

0 - 880mm	Slightly moist, reddish- brown with black spots, loose to medium dense, intact, Silty Clayey Gravelly SAND; Contains some organics in the upper layersHILLWASH; TRANSPORTED SOIL	DISTURBED SAMPLE: Collected between 880 - 3 000 mm for Foundation Indicators; MoD & CBR
880 - 3000mm	Slightly moist, olive to yellowish-brown, medium dense, intact, Silty SAND; RESIDUAL SOIL	

3000

Maximum Depth Achieved



NOTES:

Engineering Geologist: K.C. Chimanya

TEST PIT	03	COORDINATES:	S30°41'29.40" E26°39'53.19"	DATE PROFILED	16-09-2024
CLIENT:	AFROTEAM CONSULTANTS			STARTING DEPTH	0mm
PROJECT:	ALI WAL NORTH LANDFILL SITE (PROPOSED)			END DEPTH	3000mm



Depth (mm)

0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
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2300
2400
2500
2600
2700
2800
2900
3000
3100
3200

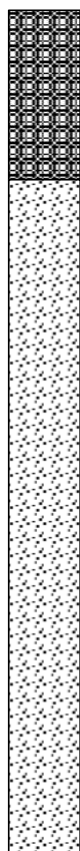
0 - 2800mm	Slightly moist, olive to reddish- brown, loose to medium dense, intact, Clayey Silty SAND; HILLWASH; TRANSPORTED SOIL	<u>DISTURBED SAMPLE</u> : Collected between 0 - 2 800 mm for Foundation Indicators; MoD & CBR
2800 - 3000mm	Dry, cream to brown, medium dense to dense, intact, Silty Clayey SAND; RESIDUAL SOIL	<u>DISTURBED SAMPLE</u> : Collected between 2 800 - 3 000 mm for Foundation
3000	Maximum Depth Achieved	



NOTES:

Engineering Geologist: K.C. Chimanya

TEST PIT	04	COORDINATES:	S30°41'30.37" E26°39'53.19"	DATE PROFILED	16-09-2024
CLIENT:	AFROTEAM CONSULTANTS			STARTING DEPTH	0mm
PROJECT:	ALI WAL NORTH LANDFILL SITE (PROPOSED)			END DEPTH	3000mm



Depth (mm)

0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
2200
2300
2400
2500
2600
2700
2800
2900
3000
3100
3200

0 - 600mm	Dry, reddis- brown with block spots, loose to medium-dense, intact, Silty Clayey SAND; HILLWASH; TRANSPORTED SOIL	-
600 - 3000mm	Dry, olive to yellowish-brown, medium dense to dense, intact, Silty SAND; RESIDUAL SOIL	-

3000 Maximum Depth Achieved



NOTES:

Engineering Geologist: K.C. Chimanya

TEST PIT	05	COORDINATES:	S30°41'22.19" E 26°39'53.58"	DATE PROFILED	16-09-2024
CLIENT:	AFROTEAM CONSULTANTS			STARTING DEPTH	0mm
PROJECT:	ALI WAL NORTH LANDFILL SITE (PROPOSED)			END DEPTH	3000mm



Depth (mm)

0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
2200
2300
2400
2500
2600
2700
2800
2900
3000
3100
3200

0 - 600mm	Slightly moist, olive to brownish grey, loose to medium dense, intact, Silty Clayey SAND; Contains some organics in the upper layers; HILLWASH; TRANSPORTED SOIL	<u>DISTURBED SAMPLE</u> : Collected between 0 - 600 mm for Foundation Indicators; MoD & CBR
600 - 3000mm	Dry, olive to brownish grey, medium dense to dense, intact, Silty Clayey SAND; RESIDIAL SOIL	

3000

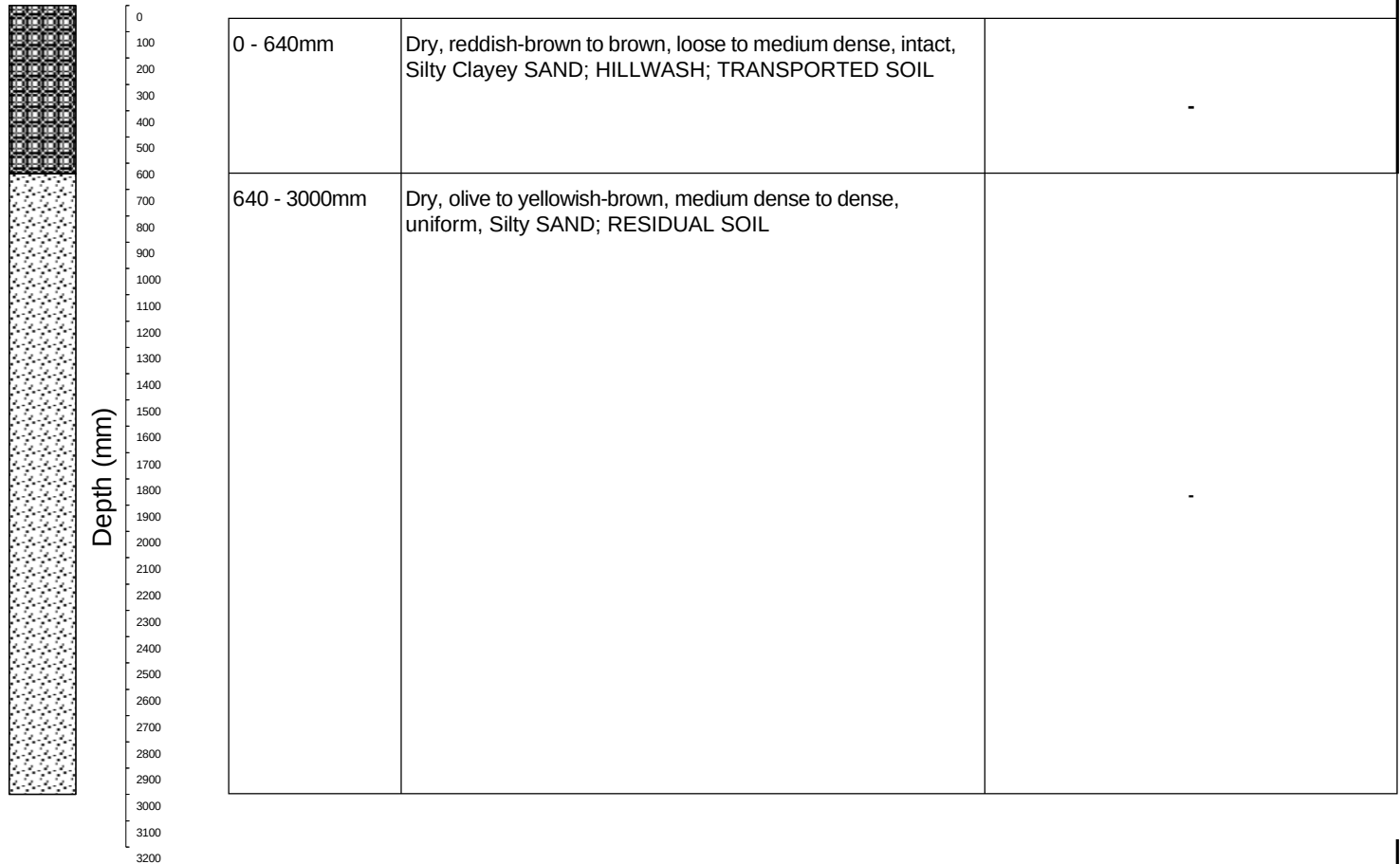
Maximum Depth Achieved



NOTES:

Engineering Geologist: K.C. Chimanya

TEST PIT	06	COORDINATES:	S 30°41'24.59" E 26°39'57.42"	DATE PROFILED	16-09-2024
CLIENT:	AFROTEAM CONSULTANTS			STARTING DEPTH	0mm
PROJECT:	ALI WAL NORTH LANDFILL SITE (PROPOSED)			END DEPTH	3000mm



3000

Maximum Depth Achieved



NOTES:

Engineering Geologist: K.C. Chimanya

Appendix B

DCP Survey Results





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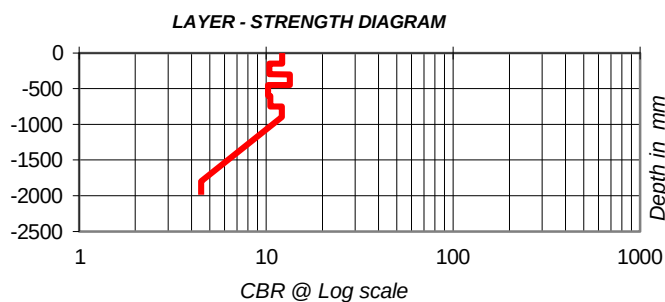
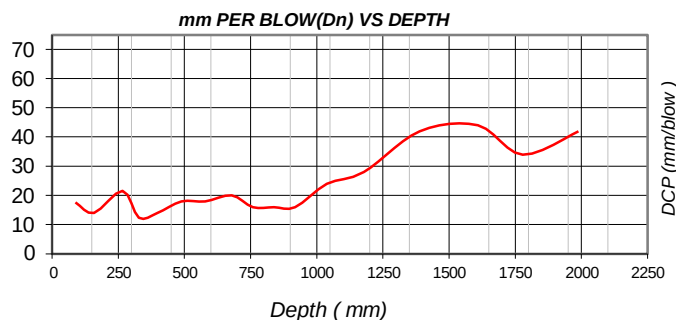
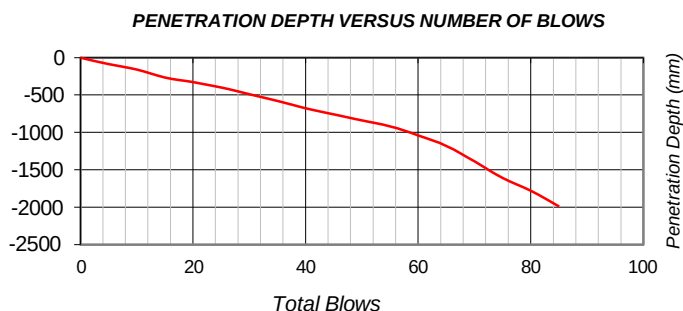
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Job no: MTR00527 Order no: - Date: 19-09-2024
Client: AFROTEAM CONSULTING Area Description: DCP1
Office 8 CK-Building Centre 2 Merrima S Position / TP: TP 1 TP1
Mbuqe -Mthatha Level: GROUND
5099
Attention: Zecks
Project: ALIWAL NORTH LANDFILL SITE

DCP TEST REPORT - TMH6: ST6

NUMBER	DEPTH	mm	NUMBER	DEPTH	mm
OF	READING	PER	OF	READING	PER
BLOWS	(mm)	BLOW	BLOWS	(mm)	BLOW
0	0		205		
5	88	17.6	210		
10	158	14	215		
15	266	21.6	220		
20	328	12.4	225		
25	398	14	230		
30	488	18	235		
35	578	18	240		
40	678	20	245		
45	758	16	250		
50	838	16	255		
55	918	16	260		
60	1038	24	265		
65	1178	28	270		
70	1388	42	275		
75	1608	44	280		
80	1778	34	285		
85	1988	42	290		
90			295		
95			300		
100			305		
105			310		
110			315		
115			320		
120			325		
125			330		
130			335		
135			340		
140			345		
145			350		
150			355		
155			360		
160			365		
165			370		
170			375		
175			380		
180			385		
185			390		
190			395		
195			400		
200			405		



In Situ Layer Strength

No.	Depth(mm) From-To	DN	Roads		Foundations
			CBR	UCS (kPa)	Bearing Capacity (kPa)
1	1-150	16	12	136	209
2	151-300	18	10	118	184
3	301-450	15	13	147	226
4	451-600	18	10	116	182
5	601-750	18	11	119	186
6	751-900	16	12	135	208
Max	901-1988	35	4	56	95

Comments:

*DCP 2m

The Bearing Capacity and CBR should only be considered as an indication of the DCP test

Mr B Taleli
Technical Signatory / Manager



Roadlab Laboratories (Pty) Ltd

Civil Materials Testing
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Registration No: 2011/005423/07
VAT No: 4670261090

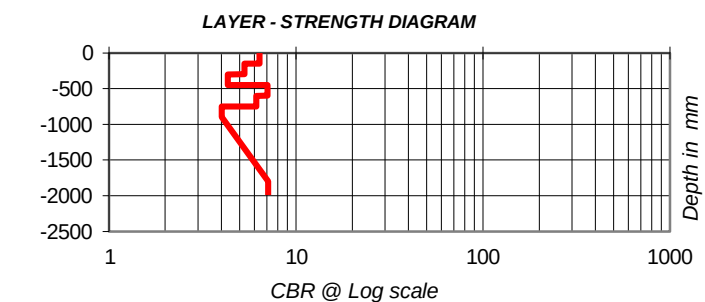
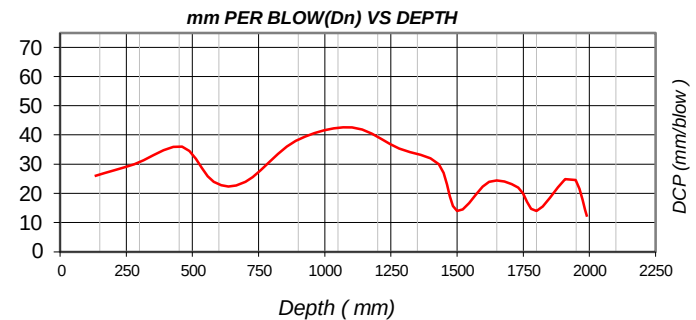
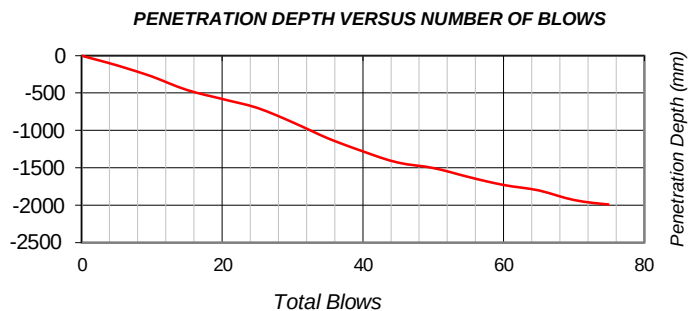
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Office 8 CK-Building Centre 2 Merrima S Position / TP: TP 2 TP2
Mbuqe -Mthatha Level: GROUND
5099
Attention: Zecks
Project: ALIWAL NORTH LANDFILL SITE

DCP TEST REPORT - TMH6: ST6

NUMBER	DEPTH	mm	NUMBER	DEPTH	mm
OF	READING	PER	OF	READING	PER
BLOWS	(mm)	BLOW	BLOWS	(mm)	BLOW
0	0		205		
5	130	26	210		
10	280	30	215		
15	460	36	220		
20	580	24	225		
25	700	24	230		
30	890	38	235		
35	1103	42.6	240		
40	1280	35.4	245		
45	1430	30	250		
50	1500	14	255		
55	1620	24	260		
60	1730	22	265		
65	1800	14	270		
70	1930	26	275		
75	1990	12	280		
80			285		
85			290		
90			295		
95			300		
100			305		
105			310		
110			315		
115			320		
120			325		
125			330		
130			335		
135			340		
140			345		
145			350		
150			355		
155			360		
160			365		
165			370		
170			375		
175			380		
180			385		
185			390		
190			395		
195			400		
200			405		



In Situ Layer Strength

No.	Depth(mm) From-To	DN	Roads		Foundations
			CBR	UCS (kPa)	Bearing Capacity (kPa)
1	1-150	26	6	77	125
2	151-300	31	5	65	108
3	301-450	36	4	54	92
4	451-600	25	7	84	135
5	601-750	27	6	74	121
6	751-900	38	4	51	86
Max	901-1990	24	7	84	136

Comments:

*DCP 2m

The Bearing Capacity and CBR should only be considered as an indication of the DCP test

Mr B Taleli
Technical Signatory / Manager



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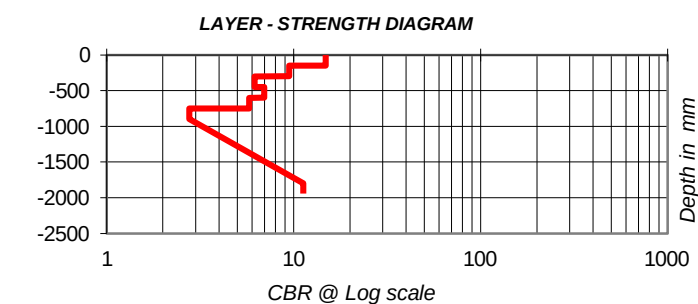
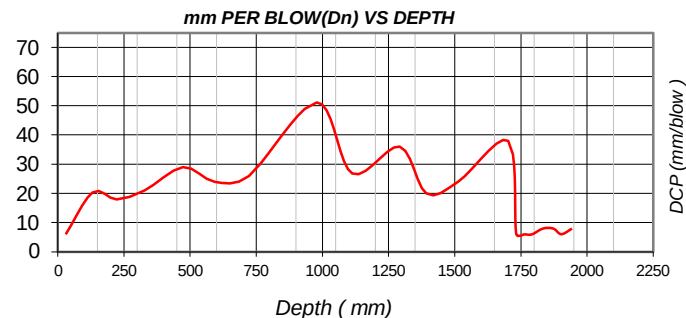
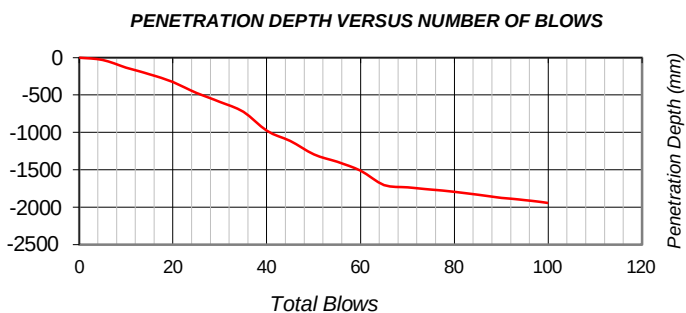
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 Client: AFROTEAM CONSULTING Area Description: DCP3
 Office 8 CK-Building Centre 2 Merrima S Position / TP: TP 3 TP3
 Mbuqe -Mthatha Level: GROUND
 5099
 Attention: Zecks
 Project: ALIWAL NORTH LANDFILL SITE

DCP TEST REPORT - TMH6: ST6

NUMBER	DEPTH	mm	NUMBER	DEPTH	mm
OF	READING	PER	OF	READING	PER
BLOWS	(mm)	BLOW	BLOWS	(mm)	BLOW
0	0		205		
5	30	6	210		
10	132	20.4	215		
15	222	18	220		
20	327	21	225		
25	472	29	230		
30	592	24	235		
35	722	26	240		
40	978	51.2	245		
45	1112	26.8	250		
50	1292	36	255		
55	1392	20	260		
60	1512	24	265		
65	1702	38	270		
70	1732	6	275		
75	1762	6	280		
80	1792	6	285		
85	1832	8	290		
90	1872	8	295		
95	1902	6	300		
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125			330		
130			335		
135			340		
140			345		
145			350		
150			355		
155			360		
160			365		
165			370		
170			375		
175			380		
180			385		
185			390		
190			395		
195			400		
200			405		



In Situ Layer Strength

No.	Depth(mm) From-To	DN	Roads		Foundations
			CBR	UCS (kPa)	Bearing Capacity (kPa)
1	1-150	14	15	161	245
2	151-300	19	9	108	171
3	301-450	27	6	75	122
4	451-600	25	7	83	134
5	601-750	29	6	70	116
6	751-900	51	3	37	64
Max	901-1942	17	11	126	196

Comments:

*DCP 2m

The Bearing Capacity and CBR should only be considered as an indication of the DCP test

Mr B Taleli
 Technical Signatory / Manager



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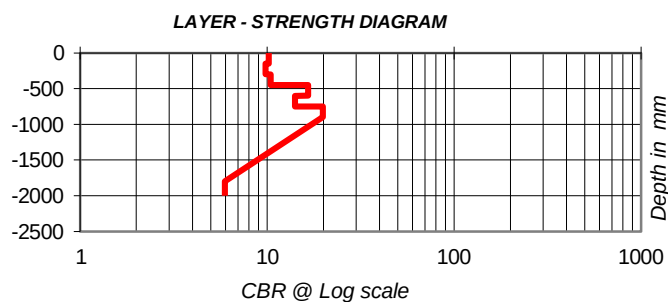
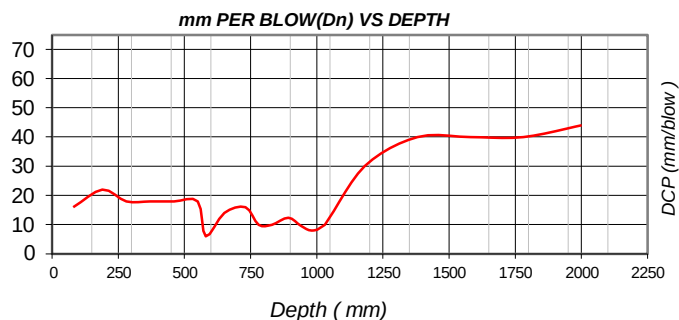
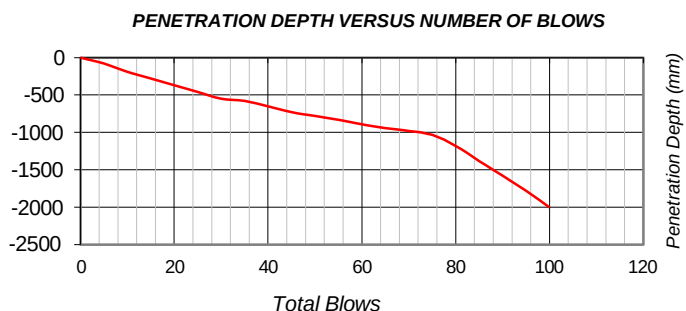
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 Germiston, JHB, 1400

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 Client: AFROTEAM CONSULTING Area Description: DCP4
 Office 8 CK-Building Centre 2 Merrima S Position / TP: 0 TP4
 Mbuqe -Mthatha Level: GROUND
 5099
 Attention: Zecks
 Project: ALIWAL NORTH LANDFILL SITE

DCP TEST REPORT - TMH6: ST6

NUMBER	DEPTH	mm	NUMBER	DEPTH	mm
OF	READING	PER	OF	READING	PER
BLOWS	(mm)	BLOW	BLOWS	(mm)	BLOW
0	0		205		
5	80	16	210		
10	190	22	215		
15	280	18	220		
20	370	18	225		
25	460	18	230		
30	550	18	235		
35	580	6	240		
40	650	14	245		
45	730	16	250		
50	780	10	255		
55	830	10	260		
60	892	12.4	265		
65	940	9.6	270		
70	980	8	275		
75	1030	10	280		
80	1180	30	285		
85	1380	40	290		
90	1580	40	295		
95	1780	40	300		
100	2000	44	305		
105			310		
110			315		
115			320		
120			325		
125			330		
130			335		
135			340		
140			345		
145			350		
150			355		
155			360		
160			365		
165			370		
170			375		
175			380		
180			385		
185			390		
190			395		
195			400		
200			405		



In Situ Layer Strength

No.	Depth(mm) From-To	DN	Roads		Foundations
			CBR	UCS (kPa)	Bearing Capacity (kPa)
1	1-150	18	10	116	182
2	151-300	19	10	112	176
3	301-450	18	10	118	185
4	451-600	13	17	177	267
5	601-750	14	14	154	235
6	751-900	11	20	208	309
Max	901-2000	28	6	72	118

Comments:

*DCP 2m

The Bearing Capacity and CBR should only be considered as an indication of the DCP test

Mr B Taleli

Technical Signatory / Manager



Roadlab Laboratories Pty Ltd

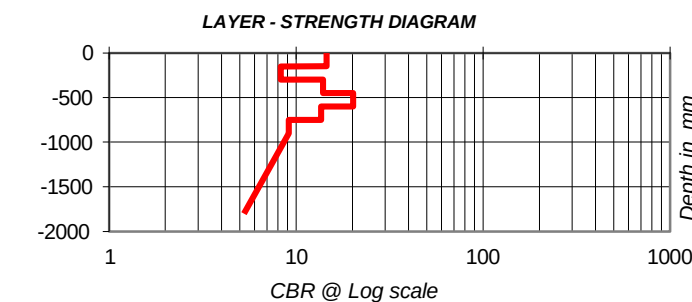
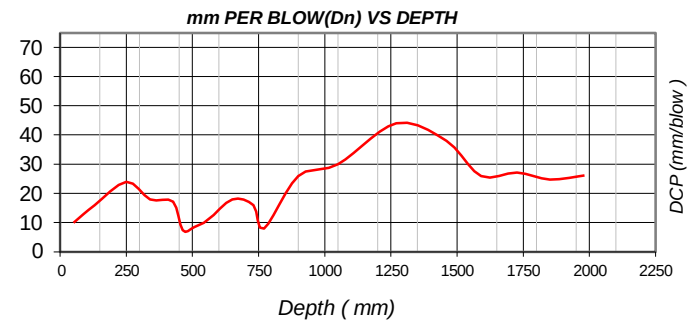
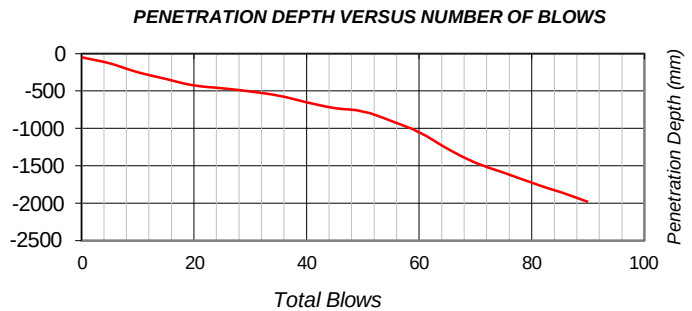
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Job no: MTR00527 Order no: - Date: 19-09-2024
Client: AFROTEAM CONSULTING Area Description: DCP5
Office 8 CK-Building Centre 2 Merrima S Position / TP: 0 TP5
Mbuqe -Mthatha Level: GROUND
Attention: Zecks
Project: ALIWAL NORTH LANDFILL SITE

DCP TEST REPORT - TMH6: ST6

NUMBER	DEPTH	mm	NUMBER	DEPTH	mm
OF	READING	PER	OF	READING	PER
BLOWS	(mm)	BLOW	BLOWS	(mm)	BLOW
0	0		205		
5	50	10	210		
10	130	16	215		
15	250	24	220		
20	340	18	225		
25	426	17.2	230		
30	463	7.4	235		
35	505	8.4	240		
40	560	11	245		
45	650	18	250		
50	730	16	255		
55	770	8	260		
60	900	26	265		
65	1050	30	270		
70	1270	44	275		
75	1460	38	280		
80	1590	26	285		
85	1726	27.2	290		
90	1850	24.8	295		
95	1981	26.2	300		
100			305		
105			310		
110			315		
115			320		
120			325		
125			330		
130			335		
135			340		
140			345		
145			350		
150			355		
155			360		
160			365		
165			370		
170			375		
175			380		
180			385		
185			390		
190			395		
195			400		
200			405		



In Situ Layer Strength

No.	Depth(mm) From-To	DN	Roads		Foundations
			CBR	UCS (kPa)	Bearing Capacity (kPa)
1	1-150	14	15	158	241
2	151-300	22	8	96	154
3	301-450	14	14	152	233
4	451-600	11	20	211	312
5	601-750	15	14	149	229
6	751-900	20	9	105	166
Max	901-1981	31	5	65	107

Comments:

*DCP 2m

The Bearing Capacity and CBR should only be considered as an indication of the DCP test

[Signature]

Mr B Taleli

Technical Signatory / Manager



Roadlab Laboratories Pty Ltd

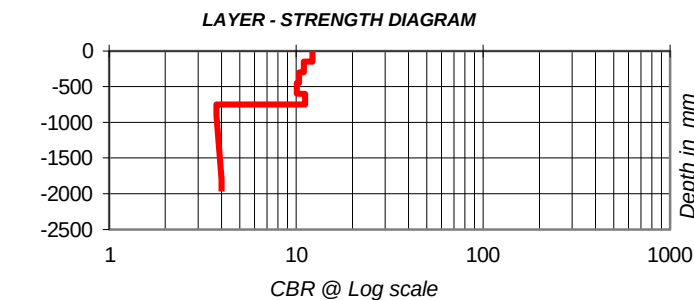
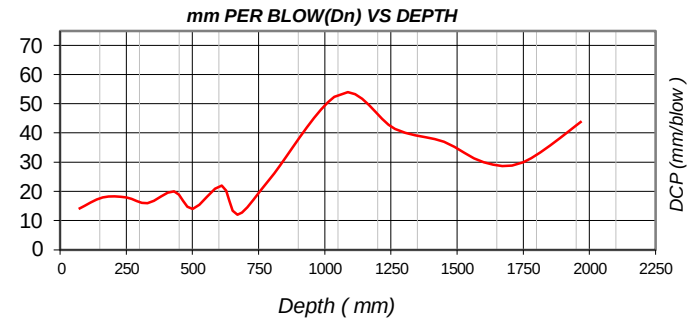
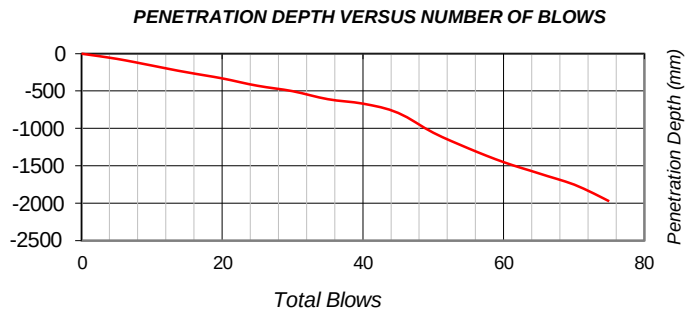
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 Germiston, JHB, 1400

Job no: MTR00527 Order no: - Date: 19-09-2024
 Client: AFROTEAM CONSULTING Area Description: DCP6
 Office 8 CK-Building Centre 2 Merrima S Position / TP: 0 TP6
 Mbuqe -Mthatha Level: GROUND
 5099
 Attention: Zecks
 Project: ALIWAL NORTH LANDFILL SITE

DCP TEST REPORT - TMH6: ST6

NUMBER	DEPTH	mm	NUMBER	DEPTH	mm
OF	READING	PER	OF	READING	PER
BLOWS	(mm)	BLOW	BLOWS	(mm)	BLOW
0	0		205		
5	70	14	210		
10	160	18	215		
15	250	18	220		
20	330	16	225		
25	430	20	230		
30	500	14	235		
35	610	22	240		
40	670	12	245		
45	790	24	250		
50	1058	53.6	255		
55	1265	41.4	260		
60	1450	37	265		
65	1600	30	270		
70	1750	30	275		
75	1970	44	280		
80			285		
85			290		
90			295		
95			300		
100			305		
105			310		
110			315		
115			320		
120			325		
125			330		
130			335		
135			340		
140			345		
145			350		
150			355		
155			360		
160			365		
165			370		
170			375		
175			380		
180			385		
185			390		
190			395		
195			400		
200			405		



In Situ Layer Strength

No.	Depth(mm) From-To	DN	Roads		Foundations
			CBR	UCS (kPa)	Bearing Capacity (kPa)
1	1-150	16	12	136	210
2	151-300	17	11	124	193
3	301-450	18	10	118	184
4	451-600	18	10	115	180
5	601-750	17	11	125	195
6	751-900	40	4	48	82
Max	901-1970	38	4	51	86

Comments:

*DCP 2m

The Bearing Capacity and CBR should only be considered as an indication of the DCP test

Mr B Taleli

Technical Signatory / Manager

Appendix C

Foundation Indicators Results





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OUR REF : MTR 00521

CLIENT : AFROTEAM CONSULTANT

PROJECT : ALIWAL NORTH LAND FILL SITE

DATE RECEIVED : 16/09/2024

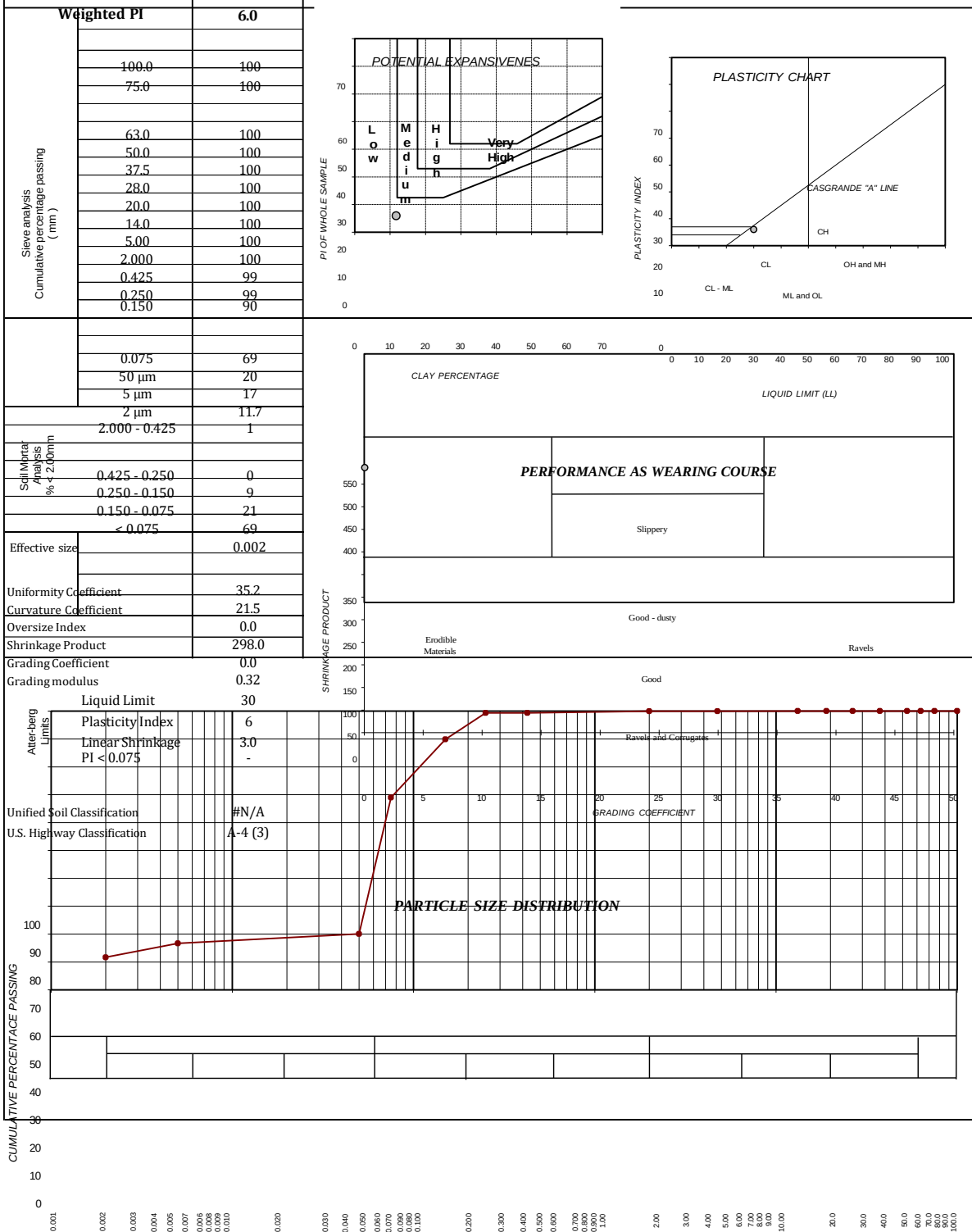
POSITION : TP 01

LAYER : 0.00 - 0.620

SAMPLE No. : RT 482

SAMPLE DESCRIPTION : DARK
Clayey Silty Sand

FOUNDATION INDICATOR - (SANS 3001-GR1, SANS 3001-GR10) & (ASTM Method D422)



CLAY (%) (0.001-0.002)	SILT (%) (0.002-0.060)	SAND (%) (0.060-2.00)	GRAVEL (%) (2.00-60.0)
11.7	20.8	67.4	0.0

MANAGER

DATE RECEIVED : 16/09/2024
POSITION : TP 01
LAYER : 0.620 - 1.700
SAMPLE No. : RT 483
LE DESCRIPTION : BROWN
Silty Clayey Sand

Weighted PI **11.7**

POTENTIAL EXPANSIVENESS

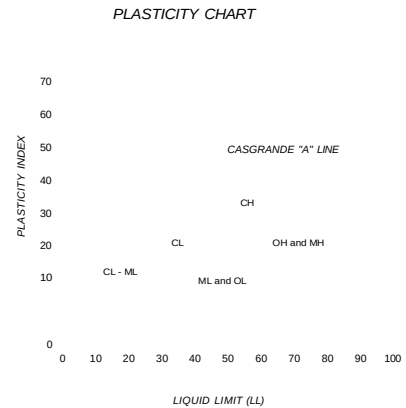
70
60
50
40
30
20
10
0

LOW
MED
HIGH

Very High

0 10 20 30 40 50

CLAY PERCENTAGE



Page 1/3

Grading Coefficient	Shrinkage Product	Material Type
5	250	Erodible Materials
25	320	Good - dusty
25	450	Slippery
25	180	Good
45	250	Ravels
25	50	Ravels and Corrugates

PARTICLE SIZE DISTRIBUTION



RLG.s.028.01

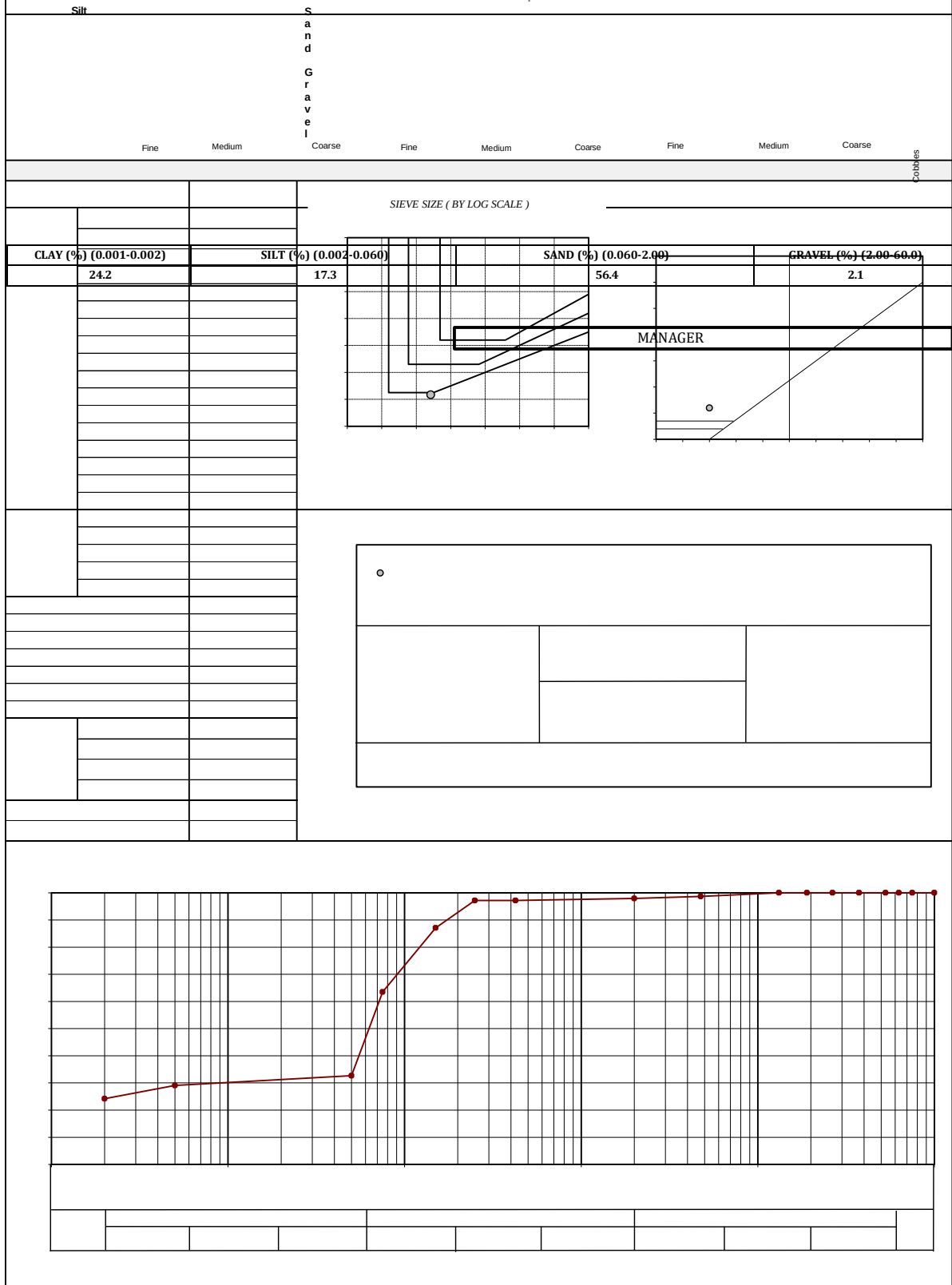


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DATE RECEIVED : 16/09/2024
POSITION : TP 02
LAYER : 0.880 - 3.000
SAMPLE No. : RT 484

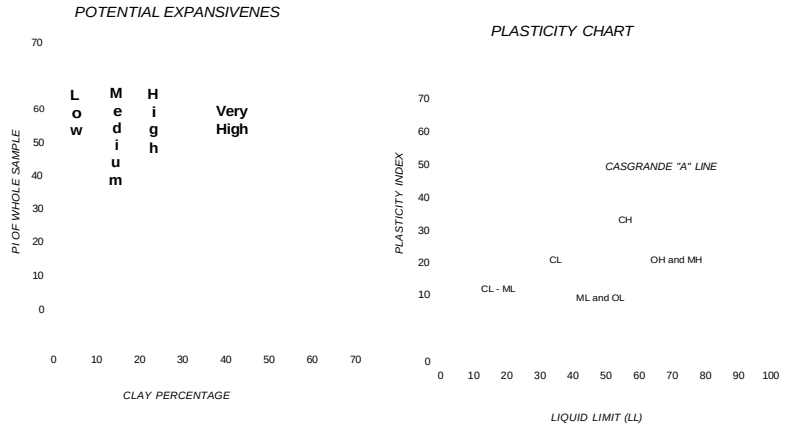
SAMPLE DESCRIPTION : BROWN
Silty Sand

FOUNDATION INDICATOR - (SANS 3001-GR1, SANS 3001-GR10) & (ASTM Method D422)

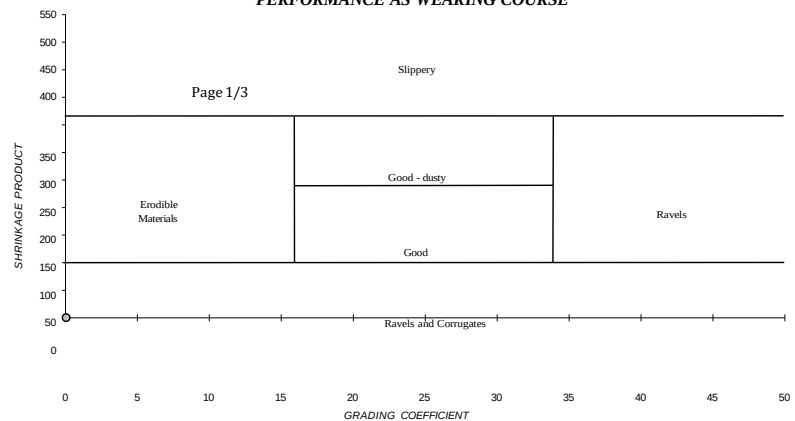
Weighted PI -

Sieve analysis	Cumulative percentage passing	100.0	100
	(mm)	75.0	100
		63.0	100
		50.0	100
		37.5	100
		28.0	100
		20.0	100
		14.0	100
		5.00	100
		2.000	100
		0.425	99
		0.250	99
Soil Mortar Analysis % < 2.00mm		0.150	74
		0.075	53
		50 µm	13
		5 µm	10
		2 µm	6.6
		2.000 - 0.425	1
		0.425 - 0.250	0
		0.250 - 0.150	25
		0.150 - 0.075	21
		< 0.075	53
	Effective size		0.005
	Atterberg Limits	Uniformity Coefficient	
Curvature Coefficient			7.6
Oversize Index			0.0
Shrinkage Product			0
Grading Coefficient			0.1
Grading modulus			0.48
Liquid Limit			-
Plasticity Index			NP
Linear Shrinkage			-
PI < 0.075			-

Unified Soil Classification	#N/A
U.S. Highway Classification	A-4 (0)



PERFORMANCE AS WEARING COURSE



PARTICLE SIZE DISTRIBUTION



Clay

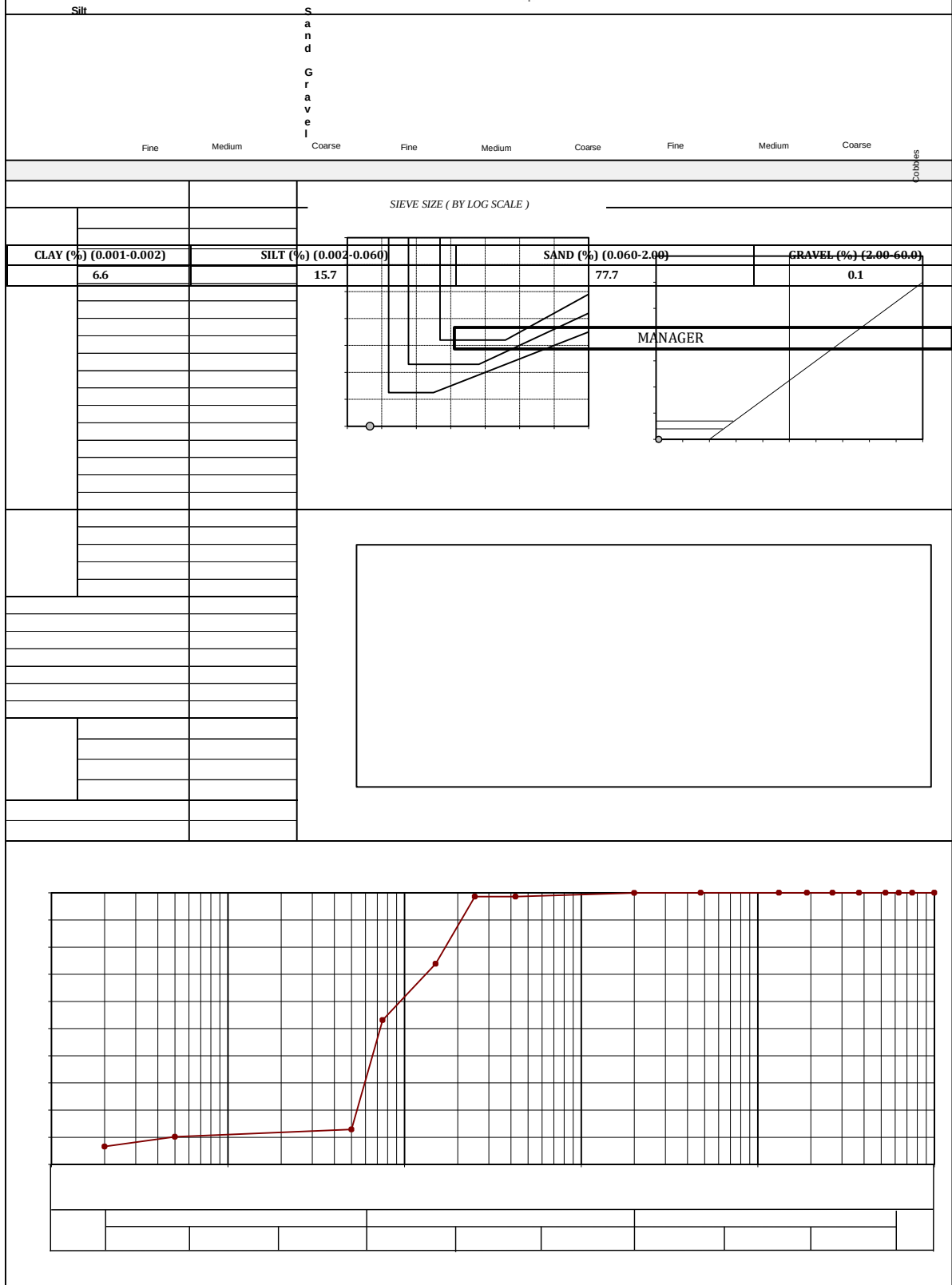


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VAT No: 4670261090

+27 82 609 6118
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DATE RECEIVED : 16/09/2024
POSITION : TP 03
LAYER : 0.0 - 2.800
SAMPLE No. : RT 485

FOUNDATION INDICATOR - (SANS 3001-GR1, SANS 3001-GR10) & (ASTM Method D422)

Unified Soil Classification	#N/A
U.S. Highway Classification	A-6 (4)

Figure 1 is a line graph showing the relationship between Clay Percentage (X-axis) and P of Whole Sample (Y-axis). The X-axis ranges from 0 to 40, and the Y-axis ranges from 0 to 70. The curve starts at (0, 0), rises to a peak of approximately 65 at 15% clay, and then declines to about 55 at 40% clay. The curve is labeled 'Low', 'Medium', 'High', and 'Very High' at different points along the X-axis.

Clay Percentage	P of Whole Sample	Category
0	0	Low
10	55	Low
15	65	Medium
20	60	Medium
25	55	High
30	50	High
35	45	Very High
40	55	Very High

The Plasticity Chart is a graph with Plasticity Index (PI) on the vertical axis (ranging from 0 to 70) and Liquid Limit (LL) on the horizontal axis (ranging from 0 to 100). A solid line, labeled 'CASAGRANDE "A" LINE', starts at (0, 40) and slopes downward to the right. The chart is divided into several regions for soil classification:

- CL - ML**: Located in the lower-left region, below the A-line and to the left of LL=40.
- CL**: Located in the middle-left region, above the CL-ML line and to the left of LL=40.
- ML and OL**: Located in the lower-middle region, below the A-line and between LL=40 and LL=60.
- CH**: Located in the middle-right region, above the A-line and between LL=60 and LL=80.
- OH and MH**: Located in the middle-right region, below the A-line and between LL=60 and LL=80.

Figure 1 is a graph showing the relationship between Shrinkage Product (Y-axis) and Grading Coefficient (X-axis). The Y-axis ranges from 0 to 550, and the X-axis ranges from 0 to 50. The graph is divided into several regions based on these two variables:

- Erodible Materials:** Located in the lower-left region, characterized by low shrinkage product and low grading coefficient.
- Good:** Located in the lower-right region, characterized by low shrinkage product and high grading coefficient.
- Good - dusty:** Located in the upper-middle region, characterized by medium shrinkage product and high grading coefficient.
- Slippery:** Located in the upper-right region, characterized by high shrinkage product and high grading coefficient.
- Ravels:** Located in the middle-right region, characterized by medium shrinkage product and high grading coefficient.

The boundaries between these regions are defined by horizontal and vertical lines. A horizontal line at approximately 160 separates the 'Good' and 'Good - dusty' regions. A vertical line at approximately 16 separates the 'Erodible Materials' and 'Good' regions. Another vertical line at approximately 34 separates the 'Good' and 'Good - dusty' regions. A horizontal line at approximately 380 separates the 'Good - dusty' and 'Slippery' regions. A horizontal line at approximately 160 separates the 'Good' and 'Ravels' regions.

RLG.s.028.01

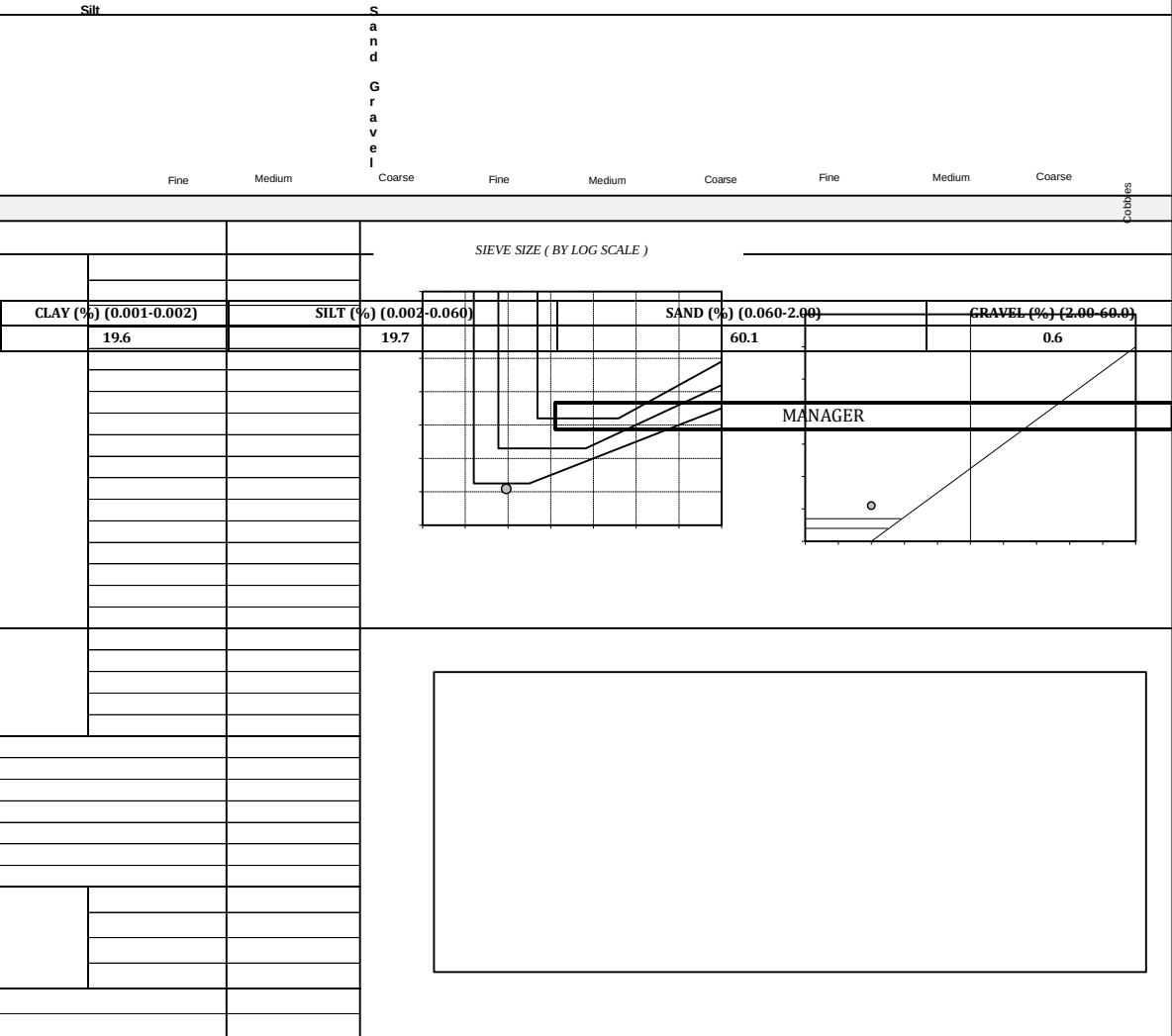


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R61 Misty Mount Off
(Norwood) Mthatha,
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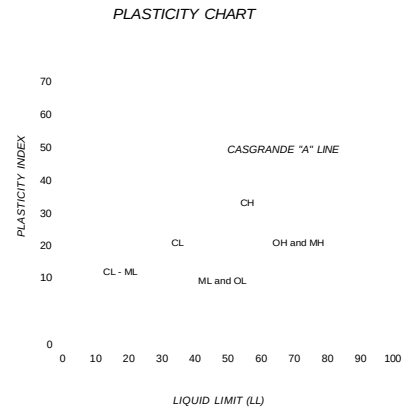
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DATE RECEIVED : 16/09/2024
POSITION : TP 03
LAYER : 2.800 - 3.000
SAMPLE No. : RT 486
LE DESCRIPTION : BROWN WHITE
 Silty Clayey Sand

Weighted PI 9.2

Clay Percentage (%)	P of Low (%)	P of Medium (%)	P of High (%)
0	60	35	5
10	55	55	20
15	50	65	35
20	45	55	25
25	40	45	15
30	35	35	10
40	30	25	5
50	25	15	0



Page 1/3

Grading Coefficient	Shrinkage Product	Material Type
5	250	Erodible Materials
25	320	Good - dusty
25	450	Slippery
25	180	Good
45	250	Ravels
25	50	Ravels and Corrugates

PARTICLE SIZE DISTRIBUTION



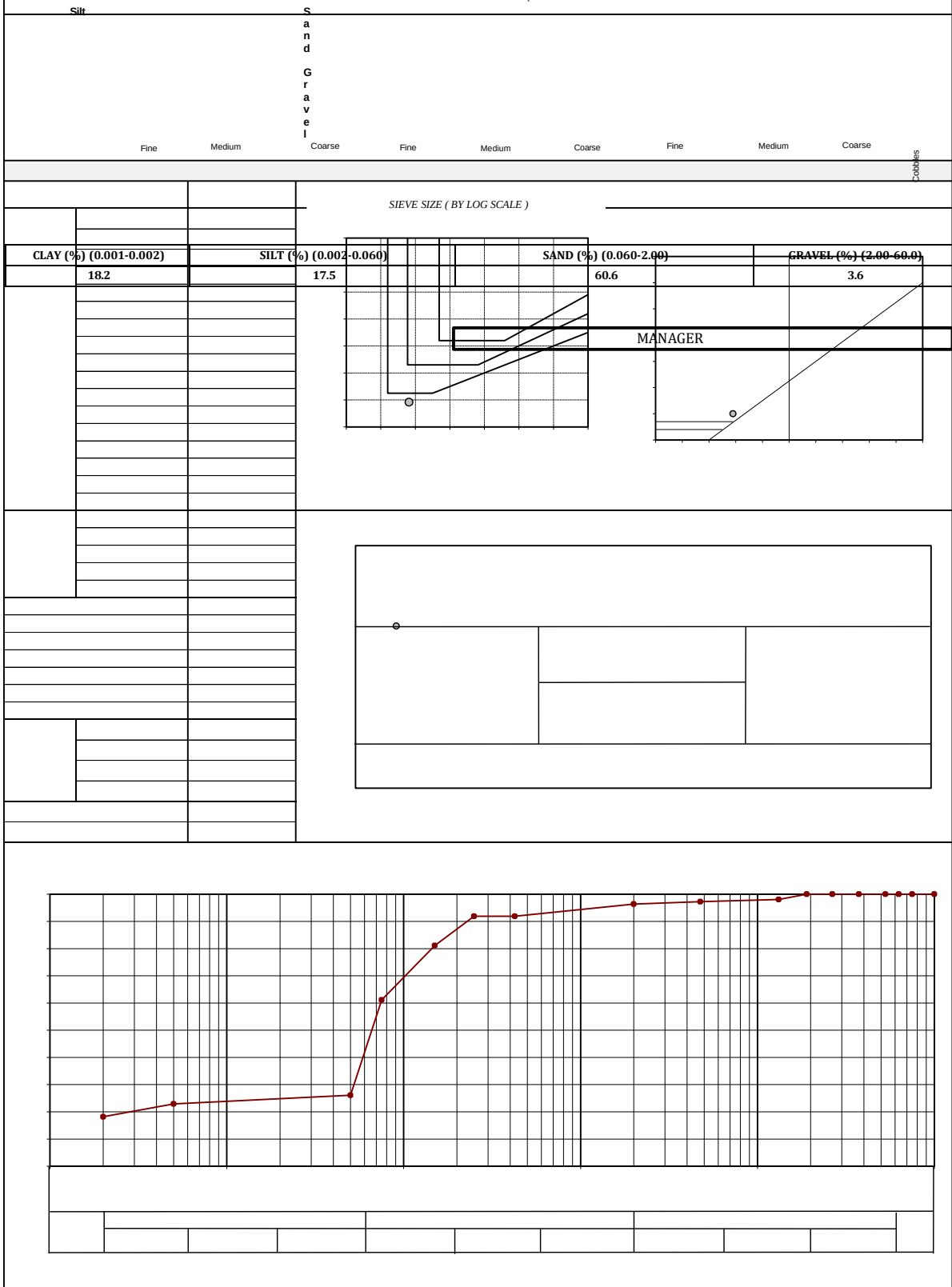


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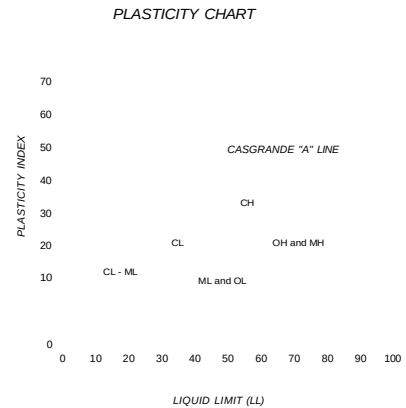
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DATE RECEIVED : 16/09/2024
POSITION : TP 05
LAYER : 0.0 - 0.600
SAMPLE No. : RT 487
LE DESCRIPTION : DARK BROWN
Silty Clayey Sand

Weighted PI	6.0
-------------	-----

Category	Clay Percentage (%)	P of Whole Sample (%)
Low	10	60
Medium	20	40
High	25	55
Very High	40	60



PERFORMANCE AS WEARING COURSE

Page 1/3

SHRINKAGE PRODUCT

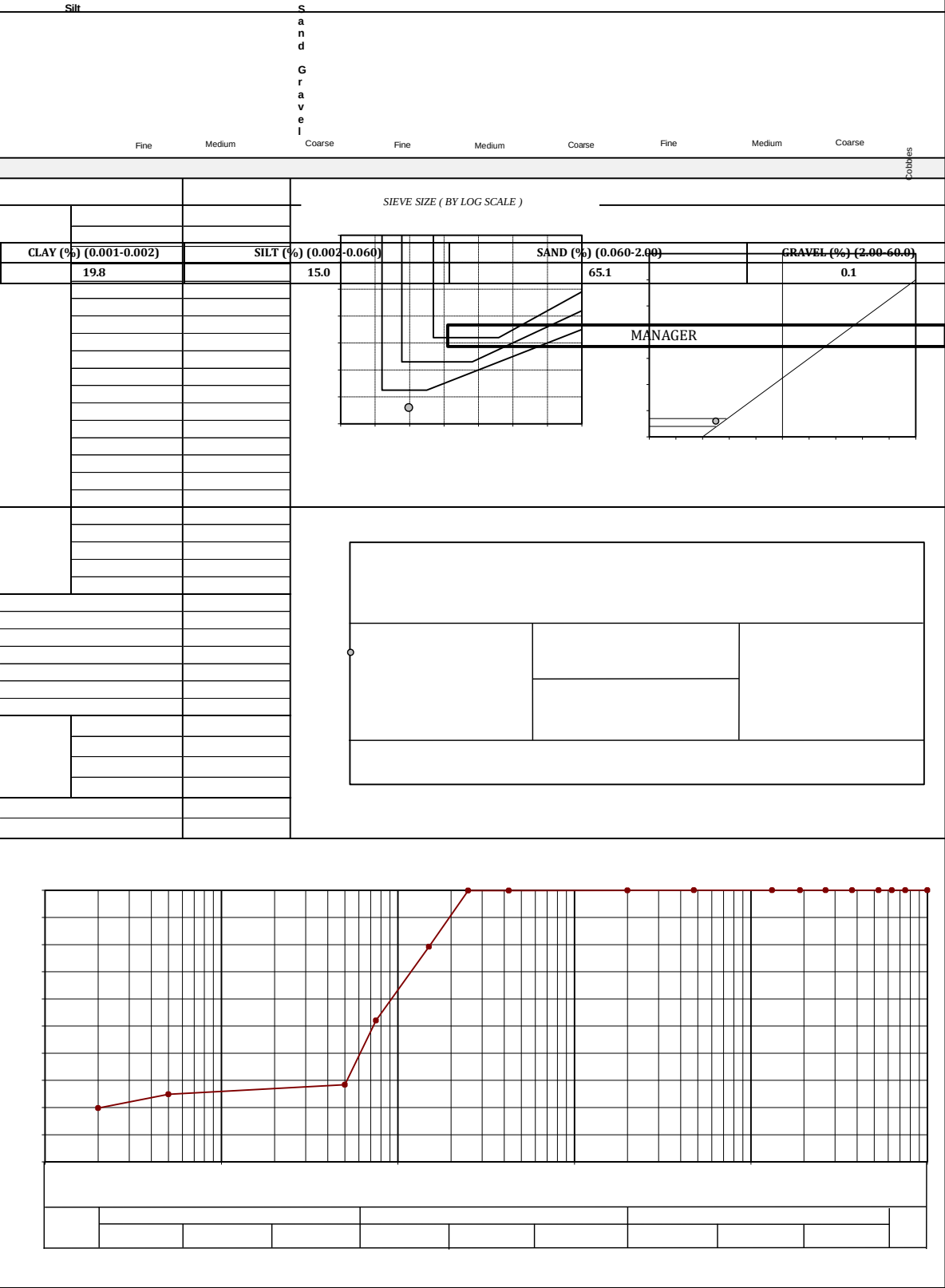
GRADING COEFFICIENT

Grading Coefficient	Shrinkage Product	Condition
5	250	Erodible Materials
25	450	Slippery
25	300	Good - dusty
25	180	Good
45	250	Ravels
25	50	Ravels and Corrugates

PARTICLE SIZE DISTRIBUTION



RLG.s.028.01



Appendix D

Moisture / Density Relationships and Compaction Test Results



Job Request No.: MTR 00527
 AFROTEAM CONSULTANT
 2 Merriman Lusiti Street -Mbuqe
 Mthatha
 5099
 Attention : Madam/Sir

Client Ref.No.: N/S

Date Reported : 25/09/2024

Project : ALIWAL NORTH LANDFILL SITE

MOD,CBR,IND SANS3001-GR1-GR10-GR20-GR30-GR40-GR41

SAMPLE INFORMATION AND PROPERTIES

SAMPLE NO.	RT 483		
HOLE NO./ Km / CHAINAGE	TP 1		
ROAD NO./ NAME Line 1	N/S		
ROAD NO./ NAME Line 2	N/S		
LAYER TESTED/SAMPLED	0.620 - 1.700		
SAMPLE DEPTH	1080		
DATE SAMPLED	17/09/2024		
COLOUR OF SAMPLE	BROWN		
TYPE OF SAMPLE	SILTY SAND		

SIEVE ANALYSIS - % PASSING SIEVES *(SANS 3001-GR1:2010, SANS 3001-GR2:2010)

SIEVE ANALYSIS (GR 1) % PASSING	100.0 mm			
	75.0 mm			
	63.0 mm			
	50.0 mm			
	37.5 mm			
	28.0 mm			
	20.0 mm			
	14.0 mm	100		
	5.0 mm	99		
	2.0 mm	98		
GM %	0.425 mm	98		
	0.075 mm	64		
		0.40		

ATTERBERG LIMITS ANALYSIS - *(SANS 3001-GR10:2010)

ATTERBERG LIMITS (%) SANS GR10,GR11	LIQUID LIMIT	20		
	PLASTICITY INDEX	12		
	LINEAR SHRINKAGE	5.0		
CLASSIFICATION	H.R.B.	A-6(7)		
	COLTO	-		
	TRH 14	-		

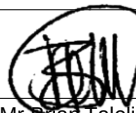
CALIFORNIA BEARING RATIO - *(SANS 3001-GR30:2010, SANS 3001-GR40:2010)

SANS GR30 MAX. DRY DENSITY	OMC %	14.7		
	MDD (kg/m³)	1864		
	COMP MC %	14.7		
SWELL % @	MOD NRB PRO	0.72 0.79 0.87		
	100 %	3		
	98 %	2		
C.B.R. SANS GR40	97 %	2		
	95 %	2		
	93 %	2		
	90 %	1		
STABILISER IN LAB		-		
TEST TYPE		MOD/CBR/IND		
SAMPLING METHOD		TMH5		
WEATHER WHEN SAMPLED		N/S		

Deviation from Test Method : No Deviation

Remarks and Notes : sampled by client

Opinions and interpretations are not included in our schedule of accreditation.
 The samples were subjected to analysis according to (SANS)(TMH5)(DOT)(ASTM)
 The test results reported relate to the samples tested.
 Further use of the above information is not the responsibility or liability of Roadlab
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 Report compiled by : Dineo Lethasa


 Mr. Brian Taleli
 Manager

Job Request No.: MTR 00527
 AFROTEAM CONSULTANT
 2 Merriman Lusiti Street -Mbuqe
 Mthatha
 5099
 Attention : Madam/Sir

Client Ref.No.: N/S

Date Reported : 25/09/2024

Project : ALIWAL NORTH LANDFILL SITE

MOD,CBR,IND SANS3001-GR1-GR10-GR20-GR30-GR40-GR41

SAMPLE INFORMATION AND PROPERTIES

SAMPLE NO.	RT : 051		
HOLE NO./ Km / CHAINAGE	TP 1		
ROAD NO./ NAME Line 1	N/S		
ROAD NO./ NAME Line 2	N/S		
LAYER TESTED/SAMPLED	0.0 0.620		
SAMPLE DEPTH			
DATE SAMPLED	17/09/2024		
COLOUR OF SAMPLE	BROWN W		
TYPE OF SAMPLE	SILTY CLAY		

SIEVE ANALYSIS - % PASSING SIEVES *(SANS 3001-GR1:2010, SANS 3001-GR2:2010)

SIEVE ANALYSIS (GR 1) % PASSING	100.0 mm			
	75.0 mm			
	63.0 mm			
	50.0 mm			
	37.5 mm			
	28.0 mm			
	20.0 mm			
	14.0 mm			
	5.0 mm			
	2.0 mm	100		
GM %	0.425 mm	99		
	0.075 mm	69		
		0.30		

ATTERBERG LIMITS ANALYSIS - *(SANS 3001-GR10:2010)

ATTERBERG LIMITS (%) SANS GR10,GR11	LIQUID LIMIT	30		
	PLASTICITY INDEX	6		
	LINEAR SHRINKAGE	3.0		
CLASSIFICATION	H.R.B.	A-6(7)		
	COLTO	-		
	TRH 14	-		

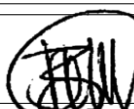
CALIFORNIA BEARING RATIO - *(SANS 3001-GR30:2010, SANS 3001-GR40:2010)

SANS GR30 MAX. DRY DENSITY	OMC %	14.5		
	MDD (kg/m³)	1569		
	COMP MC %	14.2		
SWELL % @	MOD NRB PRO	1.65 1.67 1.68		
	100 %	2		
	98 %	2		
C.B.R. SANS GR40	97 %	2		
	95 %	2		
	93 %	2		
	90 %	1		
STABILISER IN LAB		-		
TEST TYPE		MOD/CBR/IND		
SAMPLING METHOD		TMH5		
WEATHER WHEN SAMPLED		N/S		

Deviation from Test Method : No Deviation

Remarks and Notes : sampled by client

Opinions and interpretations are not included in our schedule of accreditation.
 The samples were subjected to analysis according to (SANS)(TMH5)(DOT)(ASTM)
 The test results reported relate to the samples tested.
 Further use of the above information is not the responsibility or liability of Roadlab
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 Report compiled by : Dineo Lethasa


 Mr. Brian Taleli
 Manager

Job Request No.: MTR 00527
AFROTEAM CONSULTANT
2 Merriman Lusiti Street -Mbuqe
Mthatha
5099
Attention : Madam/Sir

Client Ref.No.: N/S

Date Reported : 25/09/2024

Project : ALIWAL NORTH LANDFILL SITE

MOD,CBR,IND SANS3001-GR1-GR10-GR20-GR30-GR40-GR41

SAMPLE INFORMATION AND PROPERTIES

SAMPLE NO.	RT : 485		
HOLE NO./ Km / CHAINAGE	TP 3		
ROAD NO./ NAME Line 1	N/S		
ROAD NO./ NAME Line 2	N/S		
LAYER TESTED/SAMPLED	0.0 - 2.800		
SAMPLE DEPTH	2800		
DATE SAMPLED	17/09/2024		
COLOUR OF SAMPLE	BROWN		
TYPE OF SAMPLE	SILTY SAND		

SIEVE ANALYSIS - % PASSING SIEVES *(SANS 3001-GR1:2010, SANS 3001-GR2:2010)

SIEVE ANALYSIS (GR 1) % PASSING	100.0 mm			
	75.0 mm			
	63.0 mm			
	50.0 mm			
	37.5 mm			
	28.0 mm			
	20.0 mm			
	14.0 mm	100		
	5.0 mm	99		
	2.0 mm	99		
GM %	0.425 mm	98		
	0.075 mm	69		
		0.30		

ATTERBERG LIMITS ANALYSIS - *(SANS 3001-GR10:2010)

ATTERBERG LIMITS (%) SANS GR10,GR11	LIQUID LIMIT	20		
	PLASTICITY INDEX	11		
	LINEAR SHRINKAGE	6.0		
CLASSIFICATION	H.R.B.	A-6(7)		
	COLTO	-		
	TRH 14	-		

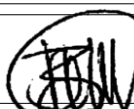
CALIFORNIA BEARING RATIO - *(SANS 3001-GR30:2010, SANS 3001-GR40:2010)

SANS GR30 MAX. DRY DENSITY	OMC %	15.3		
	MDD (kg/m³)	1769		
	COMP MC %	15.2		
SWELL % @	MOD NRB PRO	1.83 1.83 1.84		
	100 %	6		
	98 %	5		
C.B.R. SANS GR40	97 %	4		
	95 %	3		
	93 %	3		
	90 %	2		
STABILISER IN LAB		-		
TEST TYPE		MOD/CBR/IND		
SAMPLING METHOD		TMH5		
WEATHER WHEN SAMPLED		N/S		

Deviation from Test Method : No Deviation

Remarks and Notes : sampled by client

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Mr. Brian Taleli
Manager