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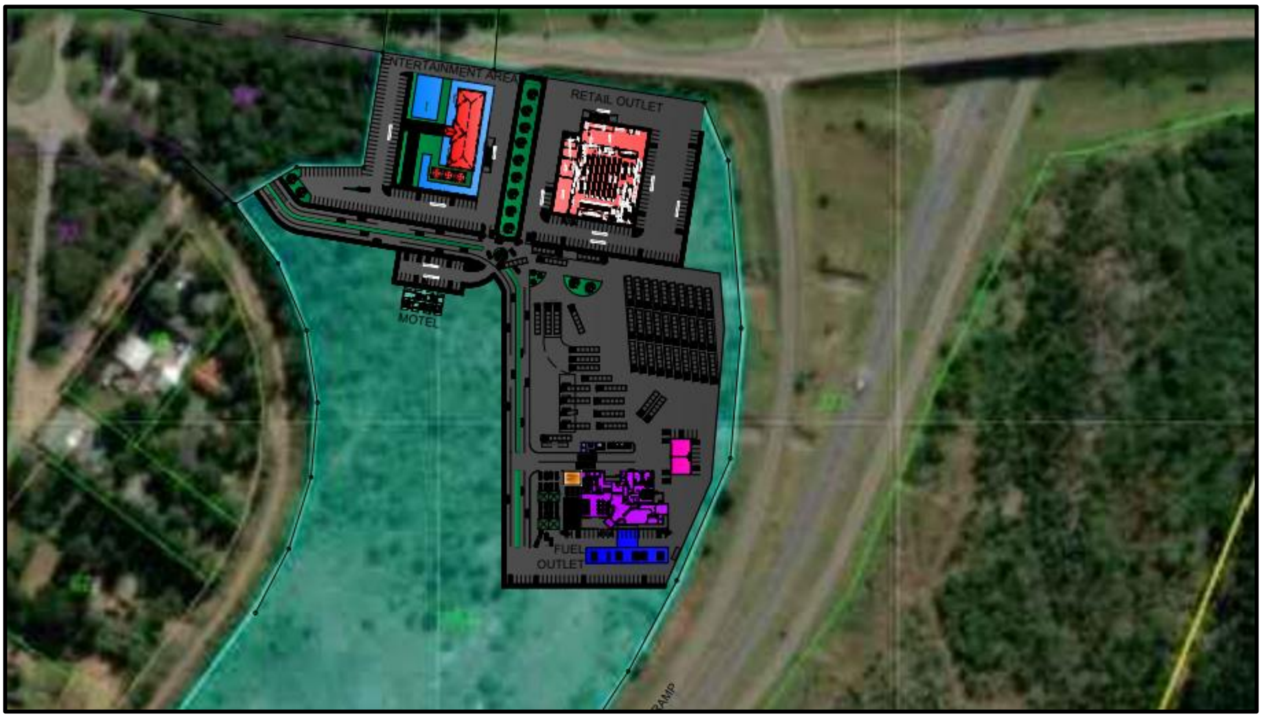
MONDLI CONSULTING (PTY) LTD

GEOTECHNICAL INVESTIGATION REPORT FOR THE PROPOSED NEW SERVICE STATION AND ASSOCIATED INFRASTRUCTURE AT KWAMBONAMBI, KWAZULU-NATAL



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REASON FOR ISSUE: FINAL GEOTECHNICAL INVESTIGATION REPORT

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1. INTRODUCTION

Umhlaba Geotechnical Engineers (hereafter referred to as UGE) was requested by Mondli Consulting (Pty) Ltd to provide a proposal for a scope of work and a cost estimate to undertake a geotechnical site investigation for the proposed new service station and associated infrastructure at KwaMbonambi, KwaZulu-Natal. The cost estimate was accepted and UGE was subsequently appointed to carry out the site investigation as proposed.

1.1 Aims and Scope of Report

The aim of the geotechnical investigation was to ascertain prevailing geological and geotechnical conditions on site for the design and construction of the proposed development.

This geotechnical investigation report summarises the results of the site investigation that was undertaken at the above mentioned site. Estimated settlement and allowable bearing pressures are provided. In addition, recommendations for site drainage, earthworks, material excavatability, material investigation for construction of roads/pavements layerworks, and suitable foundation options are also provided.

2. INFORMATION SUPPLIED

For the purpose of assisting with the investigation, Mondli Consulting provided UGE with a site layout drawing of the proposed development. In addition, the following information was utilised for the purpose of the geotechnical investigation:

- The 1:50 000 Geological Map of KwaMbonambi (2832 CA), compiled by the South African Geological Survey 1988.
- The 1:50 000 Topographical Map of KwaMbonambi (2832 CA).
- Google Earth satellite imagery of the site (2022).

3. DESCRIPTION OF SITE

3.1 Site Locality

The proposed new development is located in KwaMbonambi. The development area may be located using the following central coordinates 28°36'05.28"S and 32°05'39.54"E. KwaMbonambi is a town under King Cetshwayo District Municipality, KwaZulu-Natal.

KwaMBonambi is approximately 30km north of Richards Bay. The proposed development area is to the west of N2 Road. At the time of the site investigation, there was no development on site, with the exception of informal crop farming that was noted on isolated areas of the site. The study area was predominately covered by dense vegetation and trees. The terrain on site was relatively flat, no steep slopes were observed (Figure 1).

Figure 1: General overview of the site.

3.2 Weathering

The type and rate of rock weathering are determined by the climate of an area. Weinert (1980) developed an N-value system, which is used to establish the type of weathering that is likely to occur in an area based on macro-climatic conditions i.e. evaporation and rainfall. Mechanical weathering is likely to occur in regions where N>5, while chemical weathering occurs in regions where N<5. An N-value <2 was determined for the site, using the diagram provided in Figure 2 (TRH4, 1996). This indicates that wet climatic conditions occur on site and, that rock and soil are envisaged to be subject to chemical weathering.

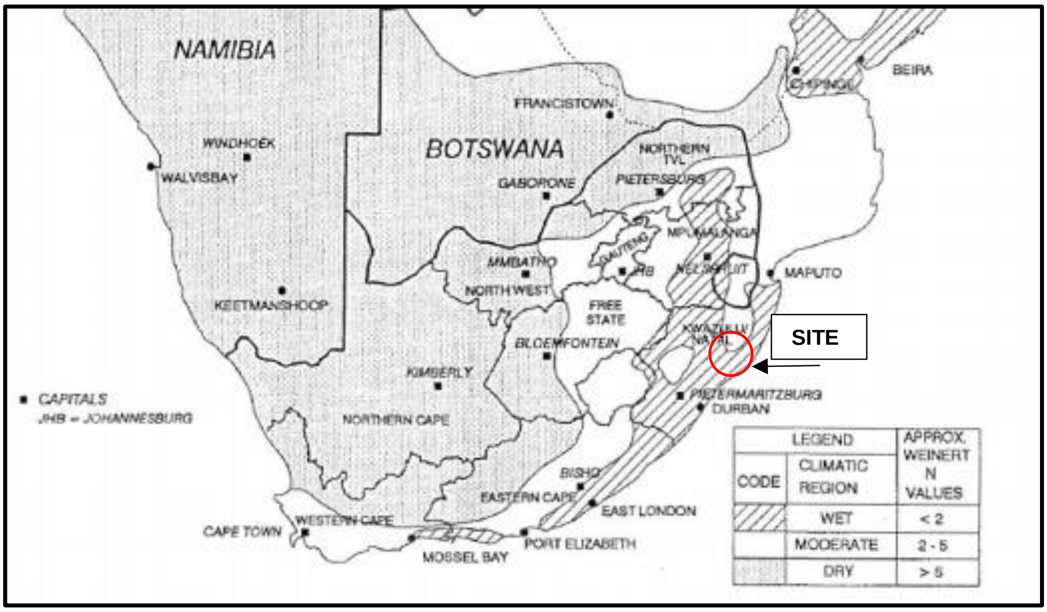


Figure 2: Micro-climatic region of Southern Africa (TRH 4, 1996 adapted from Weinert, 1980).

4. FIELDWORK

The site investigation was undertaken in accordance to the following guidelines and standard codes of practice:

- SAICE: Site Investigation Code of Practice (2010).
- Guidelines for Soil and Rock logging in South Africa (2002).
- OHS Act (1993) & Construction Regulation (2014).

The fieldwork associated with this investigation was carried out on the 23rd of April 2022. All the geotechnical information pertaining to the investigation for the proposed development is included in the appendices of this report. The profiles were logged by a professionally registered Engineering Geologist (SACNASP) using the "Guidelines for Soil and Rock Logging in South Africa (2002)". The Global Positioning System (GPS) coordinates of the tests positions were recorded.

A total of seven (7) test pits, designated TP1- TP7, were excavated using hand tools and hand auger in order to (a) identify the nature of the existing subsoil with depth, (b) retrieve disturbed soil samples for laboratory testing and (c) ascertain shallow groundwater level. The test pits were excavated to depths of below existing ground level (BEGL) using hand tools, and were advanced further by means of hand auger down to a maximum depth of 2.50m BEGL. The subsoils as revealed by the test pits are included in Appendix B of the report. The intercepted horizons were profiled as follows:

- **Soil** – moisture, colour, consistency, structure (for clays), soil type and origin.
- **Rock** – colour, degree of weathering, fabric (grain size, micro-structure spacing and discontinuity spacing), hardness, rock type and stratigraphic unit.

A total of seven (7) Dynamic Probe Light (DPL) tests, designated DPL1-DPL7, were undertaken adjacent to the test pits. The results of the DPL tests were used to evaluate (a) consistency of the underlying material, (b) refusal depth and (c) to estimate allowable bearing pressures of the in-situ material.

5. REGIONAL GEOLOGY

According to the 1:50 000 scale geological map provided by the Council of Geoscience, the site is underlain by sand and aeolianite of KwaMbonambi Formation. A geological map of the area (KwaMbonambi 2832CA 1:50 000) is included in Appendix A: Drawing No. UGE-02-GM-R00.

The upper subsoils on site comprised the following soil profiles:

Aeolian Horizon (Wind Deposit)

Aeolian horizon was noted through the upper profile of the excavated test pits. The material was intercepted from natural ground level down to a maximum depth of 2.50m BEGL. The horizon was profiled as moist/very moist, greyish pale brown/pale brown/dark grey, loose to medium dense with depth, silty fine grained sand/slightly silty fine grained sand with minor rootlets.

Alluvial Horizon (River Deposit)

Alluvial horizon was generally noted to be overlain by aeolian horizon throughout the site. the material was intercepted from a minimum depth of 0.30m down to a maximum excavation depth 2.50m BEGL. The horizon was profiled as moist, reddish orange speckled orange/yellowish orange mottled red, soft/ firm to stiff with depth, intact, slightly silty clay.

Figure 3: (a) Sandy aeolian material (b) clayey alluvial material.

6. GROUNDWATER SEEPAGE

During the site investigation, groundwater seepage was intercepted in **TP1 at a depth of 2.10m** below natural ground level. Furthermore, surfacewater was noted in isolated areas of the development area. The investigation was carried out in April, therefore it must be noted that the month of April in **KwaMbonambi** is generally one the driest months, as it occasionally receives one of the least rainfall. It should be noted that there is a possibility that during periods of prolonged rainfall, particularly during the summer season, groundwater level may ascend towards the surface. Thus, perched watertable is envisaged during and after periods of rainfall.

7. LABORATORY RESULTS

A total of eight (8) representative soil samples were retrieved from the site in order to assess the engineering characteristics of the material, and were sent to a SANAS accredited soil laboratory for testing and were subjected to:

- (a) **2No. Foundation Indicator testing:** to determine the Atterberg limits, particle size distribution and potential expansiveness.
- (b) **3No. Mod CBR testing:** to determine the suitability of the material to be use as construction material for road layerworks.
- (c) **3No. pH & Electrical Conductivity:** to determine the aggressiveness of the soil to concrete and steel

Table 1: Summary of samples tested

Test Position (TP)	Depth (m)	Foundation Indicator	Mod CBR	pH & Electrical Conductivity
TP1	0.00-2.50		x	x
TP3	0.00-1.60		x	
TP4	0.60-2.40	x		x
TP5	0.30-2.00	x	x	x

7.1 Foundation Indicator Results

Two (2) disturbed soil samples were retrieved on site for laboratory testing and were subjected to foundation indicator testing (as per TMH1 test methods A2 to A4). The samples were considered representative of the material on site. The widely used classification system for geotechnical engineering purposes is the Unified Soil Classification System (USCS).

According to the Unified Soil Classification System (USCS), the soil classifies as SC i.e. clayey sands, sand-clay mixtures. The Weighted PI of the samples is in the order of 6.9 – 11.8%. Figure 4 below shows the potential expansiveness of soil material (mm) per metre as suggested by van der Merwe (1975). As noted in the figure below, that the soil samples plot in the **low region** of the van der Merwe graph. According to van der Merwe (1975), Low potential expansiveness correlates to a heave of **0mm per 1.0m.**

Potential Expansiveness	Heave: mm per m
Very High	> 80
High	40
Medium	20
Low	0

Figure 4: Potential expansiveness (van der Merwe, 1975).

Based on the AASHTO Classification system, the material classifies as A-2-4 (0) and A-6. According to the classification system, **the general rating of the material as subgrade is excellent to good and fair to poor (A-6)**.

Table 2: Summary of Foundation Indicator Test Results

Test Pit	Depth (m)	Description	Particle size (%)				Atterberg Limits %				GM	Potential Expansiveness	Unified Classification	U.S. Highway Classification
			Clay	Silt	Sand	Gravel	LL	PI	LS	WPI				
TP4	0.60-2.40	Light orange pale red blotched yellowish brown, sandy CLAY.	28	6	58	8	29	8	4.3	6.9	0.88	L	SC	A-2-4 (0)
TP5	0.30-2.00	Dark brown, sandy CLAY.	28	12	59	1	32	12	6.3	11.8	0.61	L	SC	A-6 (2)

Where:

SC: Clayey sands, sand-clay mixtures

L: Low Heave Potential.

GM: Grading Modulus.

LL: Liquid Limit.

PI: Plasticity Index

7.2 Compaction Characteristic and Material Suitability Results

Three (3) samples were retrieved on site for MOD CBR testing to determine suitability of the material for construction of roads/pavement. The samples were classified in accordance with TRH14 guidelines.

- TP1 (0.00 – 2.50m) has a CBR strength of 9 at 95% Mod AASHTO density and **classifies as >G10 quality material**.
- TP3 (0.00 – 1.60m) has a CBR strength of 6 at 95% Mod AASHTO density and **classifies as >G10 quality material**.
- TP5 (0.30 – 2.00m) has a CBR strength of 3 at 95% Mod AASHTO density and **classifies as >G10 quality material**.

The material on site comprises **>G10** quality material, therefore it is not suitable for the upper layerworks. It is suggested that the **material should only be used as subgrade material**. The subbase layer requires a material of G5 – G6 quality. The base course material should be at least a G1 – G4 material. The base and subbase material should be obtained from a commercial source.

The design of the layerworks for parking bays and access roads should be as indicated in the table below, furthermore, the thickness of the layerworks should be according to the Engineer's specifications:

Table 3: Layerworks for light traffic flexible pavement

Layer	Material	Compaction/Strength
Surfacing	Asphalt	
Base-course	G1 – G4	86 -88% Apparent Density **(G1)** 100-102% Mod AASHTO **(G2)** 98% Mod AASHTO **(G3 and G4)**
Sub-base	G5-G6	95% Mod AASHTO
Selected layer	G6-G9	93% Mod AASHTO
Sub-grade	G8-G10	Rip and re-compact in-situ material to 93% Mod AASHTO

Table 4: Layerworks for interlocking paving blocks

Layer	Material	Compaction/Strength
Pavers	Interlocking paving blocks on 25mm bedding sand	>25MPa
Sub-base	G5/G6	95% Mod AASHTO
Sub-grade	G7	93% Mod AASHTO

Table 5: Summary of Compaction Test Results

Test Pit	Depth (m)	Description	OMC (%)	Swell (%)	MDD (kg/m³)	CBR at various densities					TRH 14
						90%	93%	95%	98%	100%	
TP1	0.00-2.50	Greyish pale brown, sandy CLAY.	9.1	0.1	1827	4	6	9	15	21	>G10
TP3	0.60-2.30	Light orange yellowish brown, sandy CLAY	102	0.1	1688	1	3	6	14	26	>G10
TP5	0.90-2.30	Light purplish brown, sandy CLAY	12.1	1.2	1877	1.1	1.6	3.0	3.6	5.2	>G10

7.3 Potential Corrosiveness of In-Situ Soil

Corrosion is defined as the degradation of a material or its properties due to a reaction with the environment. Corrosion exists in virtually all materials, but is most often associated with metals. Metallic corrosion is a naturally occurring process in which the surface of a metallic structure is oxidized or reduced to a corrosion product such as rust by chemical or electrochemical reaction with the environment.

Corrosion of metallic elements in soil can vary from relatively rapid material loss to negligible effects. The factors that influence this are:

- The heterogeneity of the soil (long-line currents).
- Aeration (oxygen and moisture to the metallic element).
- Groundwater.
- Electrical conductivity of the soil (reciprocal of resistivity).
- Chemical properties of the soil.
- Bacterial attack.
- Stay currents in the soil, and
- Soils potential to support sulphate reducing bacteria.

Table 6: Guideline Values for Interpretation of Soil Conductivity (Duligal, 1996)

Soil Conductivity (mS/m)	Corrosion Classification
More than 50	Extremely corrosive
25 – 50	Very corrosive
20 - 25	Corrosive
10 - 20	Mildly corrosive
Less than 10	Not generally corrosive

Table 7: Summary of pH and Electrical Conductivity Test

Test Pit	Depth (m)	Description	Electrical Conductivity mS/m	pH
TP1	0.00-2.50	Silty fine grained sand	5.3	5.7
TP4	0.60-2.40	Silty clay	47.2	7.1
TP5	0.30-2.0	Silty clay	30.6	6.6

According to the conductivity values given in the above table, the material retrieved at TP 1 is considered *"not generally corrosive"*. The material retrieved at TP 4 and TP 5 is regarded as being *"very corrosive"* material toward buried metallic objects, corrosion of metallic pipefittings and other construction materials will be negatively influenced by low pH and high conductivity (indicating high concentration of dissolved salts in the soils).

In the light of the foregoing information, we recommend that precautions against corrosion, in the form of piping made of inert materials such as PVC and HDPE be used instead of metallic pipes were feasible, as is frequently common local practice. It should be borne in mind that the soil pH values measured at the site are at average 6.47 (i.e. pH is ranging between 5.7 and 7.1) implying that the soil profile is in general slightly acidic. A pH levels of 5 or below can lead to extreme corrosion rates and premature pitting of metallic objects, a neutral pH of about 7 is most desirable to minimize this potential for damage.

8. SITE CLASSIFICATION

In terms of SANS 10400 – H (2012, Edition 3) clause 4.2.1, Table 1, page 11, the site class designation is **H1/S2.**

9. DEVELOPMENT RECOMMENDATIONS

9.1 Proposed Development

It is understood that a service station and associated infrastructure will be constructed in the development area. The development is anticipated to comprise the following:

- Service Station.
- Retail Outlet.
- Entertainment Area.
- Motel.
- Paved Area.
- Workshop and Battery Centre.
- Male and Female Ablutions.
- Pay Point Building for Trucker.

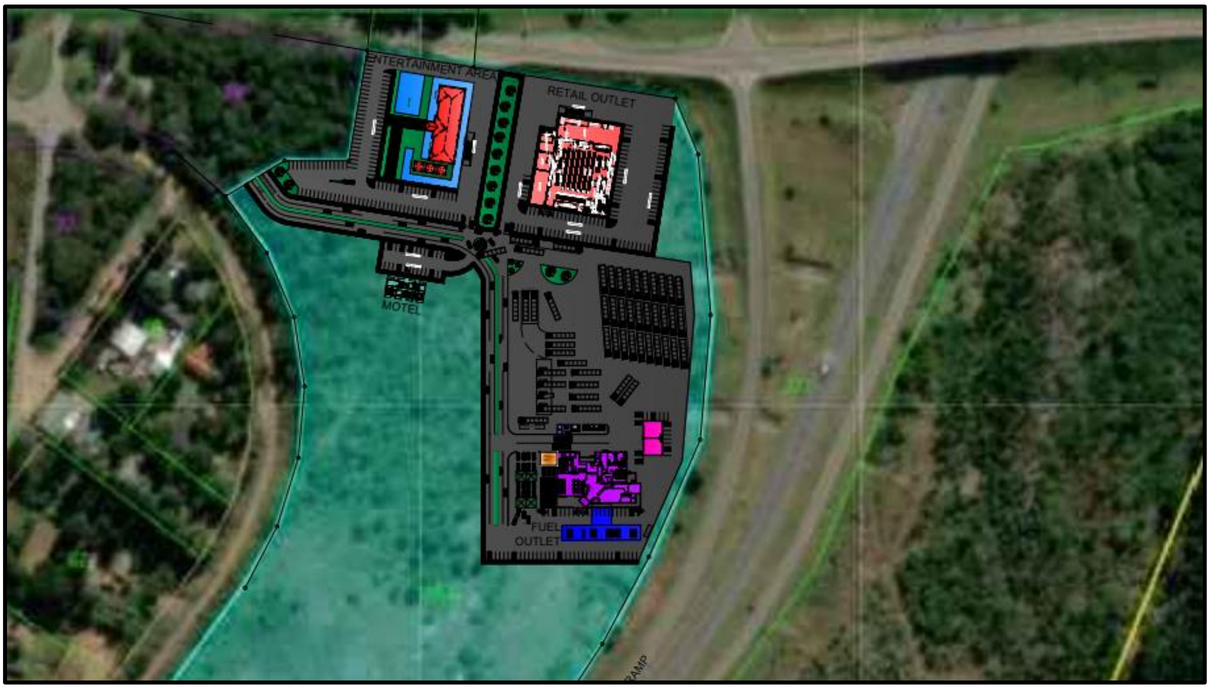


Figure 5: Proposed development on site.

9.2 Excavatability

The excavation characteristics of the soil horizons that underlain the site have been evaluated according to the South African National Standard (SANS 1200D). In terms of SANS 1200D, classification of material for excavation is generally as follows:

- **Soft excavation.**
- **Intermediate excavation.**
- **Hard rock excavation.**

According to SANS 1200D classification, the intercepted material on site classifies as **SOFT**. This implies that, material which can be efficient removed by a back-acting excavator of fly wheel power > 0.10kW for each mm of tined bucket width. During the site investigation, hand tools and an auger were used to excavate the test pits. **It is therefore suggested that a TLB or a better plant should be used for excavation on site**. No intermediate or hard excavation is anticipated for the proposed development on site.

CLASSIFICATION	DESCRIPTION
Restricted excavation	
Soft	Material which can be efficient removed by a back-acting excavator of fly wheel power >0,10 kW for each mm of tined bucket width.
Intermediate	Material which can be removed by a back-acting excavator having a fly wheel power > 0,10kW for each mm of tined-bucket width or with the use of pneumatic tools before removal by a machine capable of removing soft material.
Hard Rock	Material that cannot be removed without blasting or wedging and splitting.
Non-restricted excavation	
Soft	Material which can be efficiently removed or loaded, without prior ripping, by any of the following plant: a bulldozer or a track type front end loader having an approximate mass of 22 tonne and a fly wheel power of 145 kW. a tractor-scraper unit having an approximate mass of 28 tonne and fly wheel power of 245 kW, pushed during loading by a bulldozer equivalent to that described above.
Intermediate	Material which can be efficiently ripped by a bulldozer having an approximate mass of 35 tonne and a fly wheel power of 220 kW.
Hard Rock	Material that cannot be efficiently ripped by a bulldozer having an approximate mass of 35 tonne and a fly wheel power of 220 kW.
Boulder class A	Material containing more than 40% by volume of boulders of size between 0,03 m^3 and 20m^3, in a matrix of soft material or smaller boulders.
Boulder class B	Material containing 40% or less by volume of boulders of size between 0,03 m^3 and 20m^3, in a matrix of soft material or smaller boulders.

Figure 6: Classification of material for machine excavation (SANS 1200D).

9.3 Inspection of Excavation

The Contractor must (1) ensure that all excavations work is carried out under the supervision of a competent person and (2) evaluate the stability of the ground prior excavation work commences. All excavations must be inspected on daily basis, (a) after unexpected collapse of the ground, (b) after a rainfall, and or (c) after damage to support. The inspection will need to be carried out by a competent person as recommended by OSH Act (1993). It remains the responsibility of the Contractor to comply with the current requirements of the Occupational Health and Safety Act (1993).

9.4 Earthworks

It is recommended that earthworks on site be undertaken in accordance to SANS 1200. Where lateral supports are not constructed, cut slopes should be restricted to maximum batters of **1:1.9 (28˚) on aeolian sand and 1:1.6 (32˚) on alluvial clay,** and to a maximum height of 1.50m. Any excavation that is greater than 1.50m should be battered back to safe slope angle or must be shored to ensure the safety of construction personnel.

Prior to construction of any fill, the natural ground should be cleared of all vegetation and should be compacted with a suitable compaction plant. The fill should be constructed in layers not exceeding 150mm loose thickness and be compacted to at least at least 93% Mod AASHTO maximum dry density prior to the placement of the next layer. The fill should be compacted to within 1 to 2% of optimum moisture content (OMC).

9.5 Drainage and Stormwater

It is recommended that all stormwater from hardened areas (roof and paved areas) should be collected and discharged in a carefully controlled manner. This will need to according to the Engineer's specifications. A detailed stormwater management plan should be produced for the site. Under no circumstances should water be allowed to discharge onto the ground near the foundations. It is further recommended that concrete aprons be constructed around the perimeter of the structures **if there no rigid or flexible pavement**. The material beneath the positions of the concrete aprons should be ripped (~150mm) and re-compacted to at least 93% Mod AASHTO.

In an event where stormwater cannot be discharged into a municipal system, a soakpit system should be considered. However, a percolation test should be undertaken to assess the permeability of the subsoil. The percolation test should be undertaken according to SANS 10400-P:2012. Alternatively, an attenuation tank maybe also be used, and it must be according to Engineer's details.

The soakpit volume should be calculated as follows, $40m^2$ of hardened area of the site equals to $1m^3$ of the soakpit volume. It is important to locate stormwater soakpit on the downslope side of the site and at least 3m from the sides of any building, in order to ensure that there is no flow of subsurface water towards the foundations.

Focused runoff of surfacewater across the site should be avoided in order to limit erosion of the soil. Runoff can be minimised by the use of grass, which will allow part of the stormwater to percolate into the soil.

10. FOUNDATION RECOMMENDATIONS

10.1 Summary of Dynamic Probe Light (DPL) Test Results

As part of the geotechnical investigation, eight (8) test pits were manually excavated. The consistency of the soil material as observed at the time of profiling the test pits and DPL test results were empirically correlated to equivalent "SPT N" values in order to (a) estimate allowable bearing pressures and (b) carry out settlement analysis.

Table 8: Summary of DPL results

Depth (m)	DPL1	DPL2	DPL3	DPL4	DPL5	DPL6	DPL7
0.00	0	0	0	0	0	0	0
0.30	7	4	3	13	10	7	5
0.60	20	16	15	22	15	9	5
0.90	41	22	25	25	21	15	11
1.20	37	33	26	27	19	21	20
1.50	70	47	30	29	22	27	23
1.80	77	49	23	31	34	25	27
2.10	85	53	24	30	30	33	31
2.40	**R**	65	30	30	37	34	35
2.70		73	28	32	39	42	37
3.00		68	26	39	43	45	34

LEGEND

(yellow)	**Aeolian Horizon**
(blue)	**Alluvial Horizon**
(green)	**Refusal**

Table 9: Equivalent SPT N values estimated from DPL blows/300mm

Cohesive Soils			Cohesionless Soils		
Clay consistency	**SPT N**	**DPL blows/300mm**	**Sand consistency**	**SPT N**	**DPL blows/300mm**
Very soft	<2	0-4	Very loose	0-4	0-7
Soft	2-4	4-7	Loose	4-10	8-16
Firm	5-8	7-14	Medium Dense	11-30	17-50
Stiff	9-15	14-25	Dense	31-50	51-84
Very stiff	16-50	25-83	Very dense	>50	>84
Hard	>50	>83			

$\boldsymbol{Equivalent\ SPT\ N = 0.60 * DPL\ blows\ per\ 300mm}$

Aeolian Horizon

According to the DPL test results, the aeolian horizon is generally **loose** between 0.00 – 0.90m, and **medium dense to dense** between 0.90 – 2.10m, and dense between 2.10 – 3.00m. **Refusal was encountered only at TP1 at 2.40m BEGL.**

Alluvial Horizon

According to the DPL test results, the alluvial horizon is generally **firm to stiff** between 0.00 – 1.50m, and **very stiff** between 1.50 – 3.00m. Refusal was not encountered on the alluvial horizon.

10.2 Estimated Allowable Bearing Pressures

Although the DPL test results indicates relatively high "SPT N" values, it should be borne in mind that DPL probes are affected by numerous factors such as type of material in the soil profile, moisture content and other factors. Therefore, significant engineering judgment and understanding as well as knowledge of the specific site are necessary to maximise the information that can be obtained from DPL test results.

As mentioned that the aeolian horizon displayed medium dense to dense consistency between 0.90 to 2.10m. Let **SPT 'N' value be 11** (lower bound of medium dense consistency) for aeolian horizon, thus **the effective friction angle (Ø') is 30˚**. According to the compaction test, the material has an average maximum dry density of 17.97kN/m^3 and an average optimum moisture content of 10.47%. Therefore, the **bulk unit weight is approximately 19.85kN/m^3**.

$$\gamma_b = \gamma_d (1 + w)$$

Where:

γ_b – bulk density (kN/m³)

γ_d – dry density (kN/m³)

W – moisture content (%)

Consistency	Rule of Thumb Field Identification	Approx CPT (MPa)	Approx SPT 'N'	Approx φ'	Typical Dry Density (kN/m³)
Very Loose	Almost no resistance to shovelling	0 to 2	0 to 4	26 to 28	< 14.5
Loose	Easily penetrated with 12mm bar pushed by hand; small resistance to shovelling	2 to 4	4 to 10	28 to 30	14.5 to 16.0
Med Dense	Easily penetrated with 12mm bar driven with 2kg hammer; considerable resistance to shovelling	4 to 9	11 to 30	30 to 35	16.0 to 17.5
Dense	Hard penetration with 12 mm bar to 300mm driven with 2kg hammer; handpick required for excavation	9 to 12.5	31 to 50	35 to 40	17.5 to 19.25
Very Dense	Penetration only up to 75mm with 12mm bar driven with 2kg hammer; power tools required for excavation	> 12.5	> 50	40 to 50	> 19.25

Figure 7: Shear strength parameters for non-cohesive soils (Byrne and Berry, 2008).

As mentioned, the DPL test results indicates that the alluvial horizon is firm to stiff between 0.0 to 1.50m. Let **SPT 'N' value be 4** (lower bound of firm consistency) for alluvial horizon, **therefore the UCS strength is 40kPa (Byrne and Berry, 2008). The undrained cohesion (c') is generally half of the UCS strength, thus the undrained cohesion for the alluvial horizon is 20kPa. At the time of writing this report, dimensions of the buildings were not given, let L=B, where L is the length of the building and B is the width. Therefore, according to Skempton (1951) N_c' = 7.2.**

Table 10: Summary of bearing pressures

Effective friction angle (˚)	Bulk density (kN/m³)	Founding depth (m)	Foundation width (m)	Cohesion (kPa)	Undrained Cohesion (kPa)	N'_c	N'_q	N'_y	Ultimate Bearing Pressure (kPa)	Allowable Bearing Pressure (kPa)
Aeolian Horizon										
30	21	1.00	0.80	0	0	18.9	8.3	4.4	194	65
Alluvial Horizon										
0	18	1.00	0.80	0	20	7.2	-	-	162	54

The ultimate bearing pressures of the in-situ materials were estimated using Terzaghi's modified. **A factor of safety of three (3)** was applied to the ultimate bearing pressure in order to obtain the allowable bearing pressure. The allowable bearing pressure for the aeolian horizon is **65kPa, and 54kPa for the alluvial horizon**. These allowable bearing pressures are **INADEQUATE** to safely carry the proposed new structures. For comparative purposes, a single storey structure generally requires allowable bearing pressures in the order of 75 – 100kPa. Therefore, conventional shallow footing foundations are **NOT** feasible for the proposed structures to be directly supported on the underlying aeolian and alluvial horizon based on the obtained estimated allowable bearing pressures.

10.3 Settlement Analysis

Primary settlement (initial settlement) analysis was carried out using Skempton and Bjerrum (1957) to assess the likely settlement behaviour of a rigid square footing on the aeolian and alluvial horizon (**Table 13**); while consolidation settlement (long term settlement) analysis was undertaken using Terzaghi one dimensional theory (1925) to predict the long term settlement of the alluvial horizon as excess pore pressure dissipates from the clay (**Table 14)**. The results of the settlement analysis for a 0.80x0.80m square footing with a net bearing pressure of **150kPa at 1.00m** below existing ground level are presented below in Table 13 and Table 14.

Modulus of elasticity for the subsoils was estimated using empirically correlated SPT 'N' values. Aeolian horizon comprises of medium dense to dense soil, therefore let SPT 'N' value be **11 and 5** for the alluvial horizon. The modulus of elasticity was estimated using the methods shown below i.e. **approach 1 and approach 2**. The obtained modulus of elasticity estimated from the two methods (approaches) were averaged in order to get a mean value.

Approach 1: $E_s = 300(N + 6)$

Approach 2: $q_c = 400(N)$

$$E_s = 1.5q_c$$

Total settlement is the summation of the primary settlement and consolidation settlement for clay (**alluvial horizon**), while total settlement is equal to the primary settlement for sand material (**aeolian horizon**), therefore the total settlement for a structure with an assumed net bearing pressure of 150kPa is **58.3mm** (28.6mm + 29.7mm) on the alluvial horizon, and **14.9mm** on the aeolian horizon. A maximum total settlement of 25mm is permissible for similar structures. The estimated total settlement on the alluvial horizon is approximately two times the maximum permissible settlement i.e. 58.3mm>>>25mm. Therefore, the alluvial horizon subsurface is inadequate to safely carry the load of the proposed load. As mentioned, the total settlement on the aeolian horizon is **14.9mm**, this value is within permissible settlement i.e. 14.9mm<<<25mm. Therefore, the aeolian horizon subsurface is adequate to safely carry the load of the proposed load within minimal deformation.

Table 11: Summary of initial settlement for aeolian and alluvial horizon

LAYER	Net Foundation Pressure (kPa)	Foundation dimensions (m)	Poisson's Ratio	Young Modulus (kPa)	Founding Depth (m)	Influence Factor	Predicted Settlement (mm)
Aeolian Horizon	150	0.80x0.80	0.30	5850	1.00	0.8	**14.9**
Alluvial Horizon	150	0.80x0.80	0.25	3150	1.00	0.8	**28.6**

$* \ v \ \ correlated \ from \ Das \ (2017) based \ on \ soil \ consistency.$

Table 12: Summary of consolidation settlement for alluvial horizon

LAYER	Net Foundation Pressure (kPa)	Foundation dimensions (m)	Ave coefficient of volume compressibility (m²/MN) (m_v)	Coefficient which depends on clay type (μ_g)	Founding Depth (m)	Oedometer Settlement (mm) (P_{oed})	Predicted Consolidation Settlement (mm) (P_c)
Alluvium horizon	150	0.80x0.80	0.30	1.0	1.00	29.7	**29.7**

m_v *and* μ_g *correlated fromTomlinson (1980)based on type of clay.*

10.4 Proposed Foundation

A total of three (3) samples were retrieved on site and were subjected to Foundation Indicator testing to determine (a) Atterberg limits, (b) particle size distribution, and (c) heave potential. According to the lab results, the two samples retrieved from the alluvial horizon will not be subjected to heaving. The alluvium material plots in the low heave region of the van der Merwe graph; while the aeolian material which plot in the low region of the van der Merwe graph. This implies that for every 1000mm of the layer of soil, the alluvial material is anticipated to heave by 0mm. Alternatively, for every 300mm of the layer of soil, the material is envisaged to heave by 6mm. According to SANS 10400H, the site class designation of the study area is **H1/S2.**

Based on the bearing capacity and settlement analysis, the subsoils on site do not satisfy the governing factors i.e. bearing capacity and settlement. Furthermore, the alluvial horizon on site will be subjected to medium heave, therefore construction of strip footing and pad footing foundations directly onto the in-situ material is **NOT feasible**. It is therefore suggested that the **single storey structures** be founded on improved soil as follows:

(A) Single Storey Buildings

- **Option 1: Reinforced Raft Foundations**

Reinforced raft foundation may be used for all single storey buildings, where the design bearing pressure is limited to a **maximum of 100kPa, however, the reinforced raft foundation will NEED to be supported on engineered fill as specified below**.

The in-situ material must be removed to a depth of 0.50m below ground level and should be replaced with at least G7 quality material and be compacted in 150mm layers to 93% Mod A.A.S.T.O maximum dry density. The reinforced raft foundations should be according to Engineer's specifications. Guidelines for **reinforced raft foundations** are given in SANS 10400-H:2012.

- **Option 2: Reinforced Strip Footing Foundations**

Reinforced strip footing foundation may be used for all single storey buildings, where the design bearing pressure is limited to a **maximum of 100kPa, however, the reinforced strip footing foundations will NEED to be supported on engineered fill as specified.**

Footings: The in-situ material must be removed from the footprint of the proposed structures. The over-excavated material must then be replaced with at least G7 quality material to a minimum of 1.5B (where B = foundation width) below the foundation footings and to a distance of 0.5B either side of the footings in order to minimise the risk of differential settlement. The inert granular material should be compacted in layers not exceeding150mm to at least 93% Mod A.A.S.T.O maximum dry density. The design bearing pressure should be limited to **100kPa.** The reinforced strip footing foundations must be according to the Engineers specifications. Guidelines for **reinforced strip footing foundations** are given in SANS 10400-H:2012.

Surface bed: 300mm of the subsoil material beneath the floor area should be removed and must be replaced with at least G7 quality material loosely compacted in 100mm layers to at least 93% Mod AASHTO maximum dry density.

It is essential that the concrete is poured into the excavation with a minimum of delay. In this regard, foundation trenches should not remain exposed for periods longer than 3 days to avoid mechanical and chemical deterioration.

(B) <u>Double Storey Buildings</u>

It is understood that a motel forms part of the proposed development on site. At the time of writing this report, it is understood that the proposed motel will be a double storey structure. It is therefore recommended that **any double storey structure** on site be founded on CFA pressure piled foundations with ground beams spanning over the piles.

Table 13: Preliminary Pile Design Summary

Working Load (kN)	Diameter (mm)	Pile Length (m)	Reinforcement (0.8%)
200	250	8	6Y10
300	300	10	6Y12
350	300	11	6Y12
450	350	12	7Y12

The pile lengths in the table above are approximate pile lengths and are measured below the pile caps. **This preliminary pile design summary is based on a maximum shaft stress of 5MPa and a factor of safety of >2 for skin friction and factor of safety of >3 for end bearing.**

The reinforcement for the piles should be at least 0.80% of the required pile area. The size and or quantity of pile reinforcement may need to be adjusted should the piles be subjected to excessive lateral loads. In addition, the diameter of the helical steel should not be less than ¼ of the diameter of compression bar, and the pitch must not be greater than 16 times the diameter of the compression bar. If the piles will be subjected to uplift forces, the skin friction and the self-weight of the pile must be greater than the uplift force. **A detailed pile design should be carried out by the Piling Contractor.**

The following are recommended:

- It is recommended that a detailed pile design should be carried out by the Piling Contractor.
- The piles should be designed by a competent Geotechnical Professional.
- A pile load test should be undertaken on at least one pile to ascertain the pile load carrying capacity. A pile load test is the only dependable means of ascertaining the load capacity of piles.
- A pile integrity should be carried out on all the piles, to check the pile shaft for any structural defects and to ensure the quality of the piles.
- Ground beams should be constructed as the main support structure spanning over the piles.

11. CONCLUSIONS

The report presents the findings of a shallow geotechnical investigation that was undertaken on the 23rd of April 2022 in KwaMbonambi, KwaZulu-Natal.

Based on the information collected at the time of the site investigation the proposed site is feasible for the development provided that the recommendations contained in this report are adhered to.

The subsoil conditions provided in the report refer to the excavated test pits on site, therefore it is possible that conditions may be different elsewhere on site. During the site investigation, an effort was undertaken to identify the prevailing subsoil material, however it is not possible to ensure that varying material may have been missed. The undertaken investigation was in-depth, and site conditions are not anticipated to vary that much compared to those identified in this report. It is therefore essential that Umhlaba Geotechnical Engineers be appointed to undertake inspection during construction.

We trust that this brief summation of our findings and conclusions meet your immediate requirements. Should you require clarity on any part of the report, please do not hesitate in contacting Umhlaba Geotechnical Engineers.

Yours sincerely,
Mr TMH (Pr.Sci.Nat- 124314)
Umhlaba Geotechnical Engineers

APPENDIX A:

TEST POSITIONS *– UGE-01-TP-R00*

REV	DATE	DRAWN	CHECKED	DESCRIPTION

MHLABA

123 Marine View Avenue
Durban North
4051

Cell : 081 328 4756
Email: info@umhlaba.co.za

PROJECT

KWAMBONAMBI SERVICE STATION

DRAWING

TEST POSITIONS

	RESPONSIBLE PERSON		DATE
DRAWN	TS PHEWA		
CHECKED	TS SKOSANA	Pr.Sci.Nat: XXXXXXX	
APPROVED	TS SKOSANA	Pr.Sci.Nat: XXXXXXX	
SCALE	NTS	SHEET SIZE	A3

PROJECT NUMBER DRAWING NUMBER STATUS REVISION

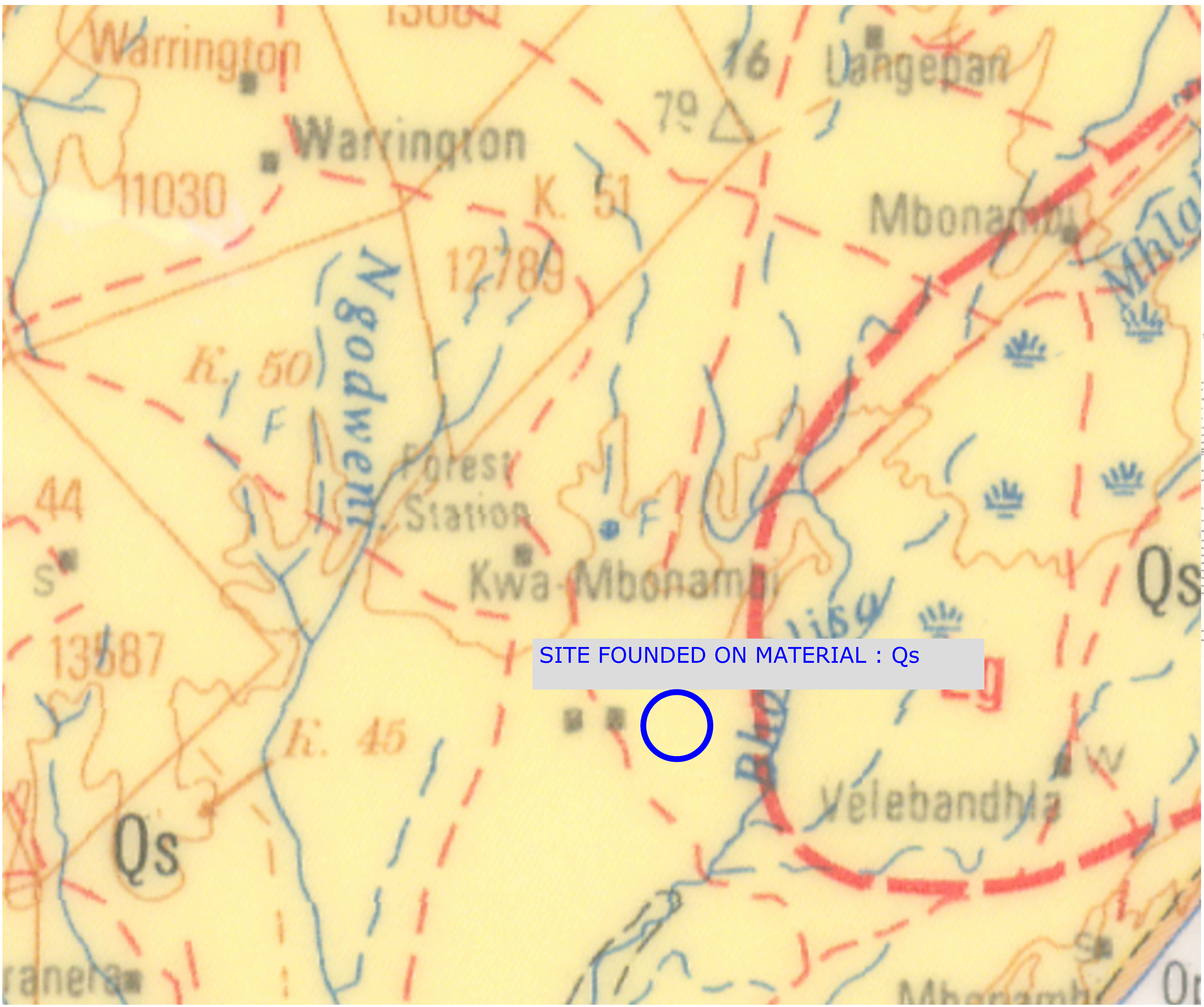
UGE-03-22-TP-R00

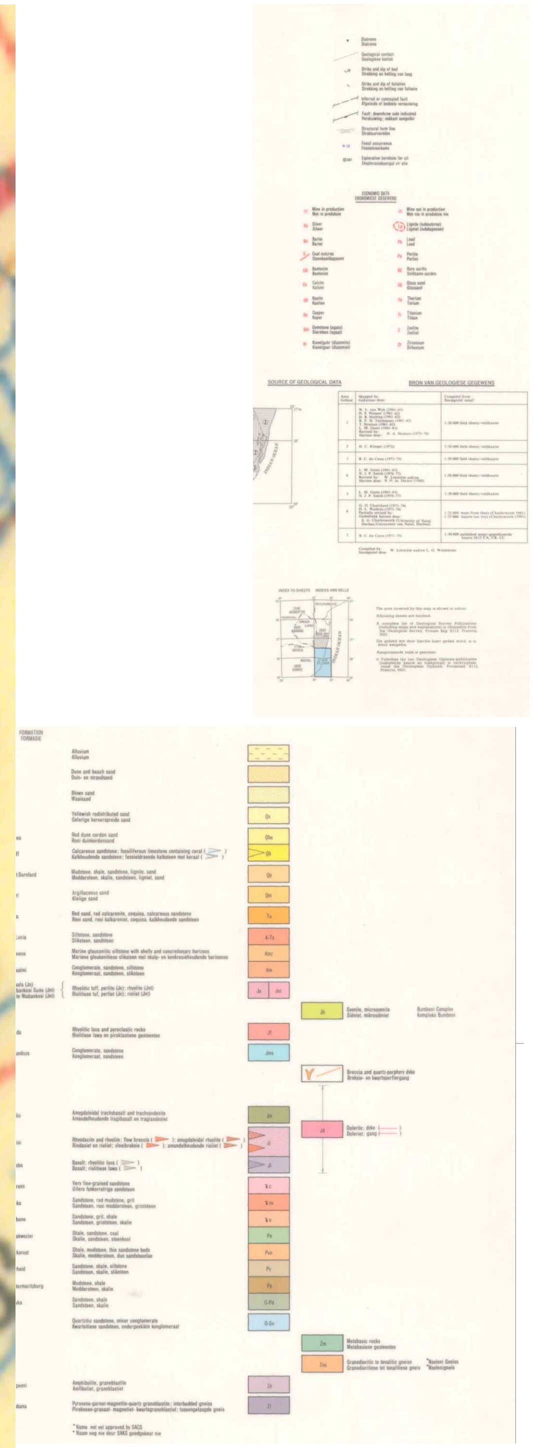
DRAWING STATUS CODES :

R = REPORT
D = DRAFT
T = TENDER
P = PRELIMINARY
C = CONSTRUCTION
A = AS BUILT

APPENDIX B:

GEOLOGICAL MAP *– UGE-02-GM-R00*





GEOMAP			A3
REV	DATE	DRAWN	AMENDMENTS

PROJECT:

KWAMBONAMBI SERVICE STATION

TITLE:

GEOLOGICAL MAP

DRAWN: TS PHEWA

DATE: 11-05-2022

DRAWING

UGE-03-22-GM-R00

UMHLABA
GEOTECHNICAL ENGINEERS

123 MARINE VIEW AVENUE
DURBAN NORTH
4051
TEL: 081 328 4756
EMAIL: info@umhlaba.co.za

APPENDIX C:

TEST PIT LOGS

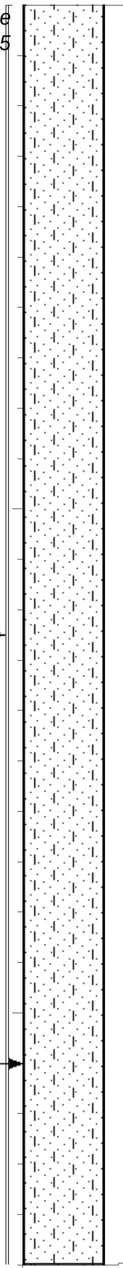
MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

HOLE No: TP 01
Sheet 1 of 1

JOB NUMBER: UGE 03-22

Scale 1:15

0.00



Very moist, greyish pale brown, loose to medium dense with depth, silty fine grained SAND . **Aeolian**.

MOD & PH&EC

2.10m

2.50

NOTES

1) Ground water seepage noted at 2.10m.
2) Side wall collapse at 1.40m.
3) TP advanced with hand auger between 1.50 to 2.50m.
4) PH & EC sample taken between 0.00--2.50m.
5) MOD CBR sample taken between 0.00--2.50m.
6) Surface noted.

CONTRACTOR :	*INCLINATION :*	*ELEVATION :*
MACHINE : HAND&HAND AUGER	*DIAM :*	*X-COORD :* 28° 36’ 09.36"S
DRILLED BY :	*DATE :* 23 APRIL 2022	*Y-COORD :* 32° 05’ 40.12"E
PROFILED BY : TS SKOSANA	*DATE :* 24 APRIL 2022	*HOLE No:* TP 01
TYPE SET BY : TS PHEWA	*DATE : 07/05/2022 09:19*	
SETUP FILE : STANDARD.SET	*TEXT : ..ORTHEREPORT\LOGS\LOG.TXT*	

MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

HOLE No: TP 02
Sheet 1 of 1

JOB NUMBER: UGE 03-22

Scale 1:15

0.00

Very moist, greyish pale brown, loose to medium dense with depth, silty fine grained SAND. **Aeolian**.

2.50

NOTES

1) TP advanced with hand auger between 1.50 to 2.50m.

2) Surface water noted.

CONTRACTOR :
MACHINE : HAND&HAND AUGER
DRILLED BY :
PROFILED BY : TS SKOSANA
TYPE SET BY : TS PHEWA
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE : 23 APRIL 2022
DATE : 24 APRIL 2022
DATE : 07/05/2022 09:19
TEXT : ..ORTHEREPORT\LOGS\LOG.TXT

ELEVATION :
X-COORD : 28° 36' 07.64"S
Y-COORD : 32° 05' 40.67"E

HOLE No: TP 02

MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

HOLE No: TP 03
Sheet 1 of 1

JOB NUMBER: UGE 03-22

Scale 1:15

MOD

Depth (m)	Description
0.00	Moist, pale brown, loose to medium dense with depth, slightly fine grained SAND. **Aeolian**
1.60	Very moist, greyish brown, loose to medium dense, slightly clayey fine grained SAND. **Aeolian**.
2.30	Moist, reddish orange speckled orange, intact, soft, slightly silty CLAY. **Alluvium**.
2.50	

NOTES

1) MOD CBR sample taken between 0.00--1.60m.

2) No groundwater seepage.

CONTRACTOR :
MACHINE : HAND&HAND AUGER
DRILLED BY :
PROFILED BY : TS SKOSANA

TYPE SET BY : TS PHEWA
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE : 23 APRIL 2022
DATE : 24 APRIL 2022

DATE : 07/05/2022 09:20
TEXT : ..ORTHEREPORT\LOGS\LOG.TXT

ELEVATION :
X-COORD : 28° 36' 06.09"S
Y-COORD : 32° 05' 42.70"E

HOLE No: TP 03

MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

HOLE No: TP 04
Sheet 1 of 1

JOB NUMBER: UGE 03-22

Scale 1:15

Depth (m)	Description
0.00	Moist, dark grey, loose, fine grained SAND with rootlets. **Aeolian**.
0.60	Moist, yellowish orange mottled red, firm to stiff with depth, intact. **Alluvium**.
2.40	

PH&EC & FI

NOTES

1) PH & EL sample taken between 0.60--2.40m.

2) FI sample taken between 0.60--2.40m.

CONTRACTOR :
MACHINE : HAND&HAND AUGER
DRILLED BY :
PROFILED BY : TS SKOSANA
TYPE SET BY : TS PHEWA
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE : 23 APRIL 2022
DATE : 24 APRIL 2022
DATE : 07/05/2022 09:20
TEXT : ..ORTHEREPORT\LOGS\LOG.TXT

ELEVATION :
X-COORD : 28° 36’ 01.53"S
Y-COORD : 32° 05’ 41.62"E

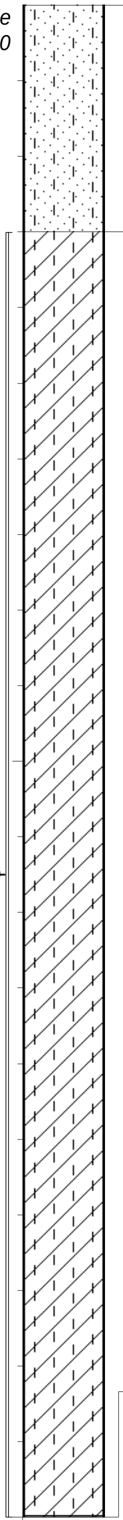
HOLE No: TP 04

MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

HOLE No: TP 05
Sheet 1 of 1

JOB NUMBER: UGE 03-22

Scale 1:10



0.00 Moist, dark grey, loose, silty fine grained SAND. **Aeolian.**

0.30 Moist, yellowish orange speckled red, intact, firm to stiff with depth, silty CLAY. **Alluvium.**

2.00

NOTES

1) PH & EL sample taken between 0.30--2.00m.
2) FI sample taken between 0.30--2.00m.
3) MOD CBRtaken between 0.30--2.00m.
4) Hand auger refused at 2.00m.

CONTRACTOR :
MACHINE : HAND AUGER
DRILLED BY :
PROFILED BY : TS SKOSANA

TYPE SET BY : TS PHEWA
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE : 23 APRIL 2022
DATE : 24 APRIL 2022

DATE : 14/07/2022 20:09
TEXT : ..ORTHEREPORT\LOGS\LOG.TXT

ELEVATION :
X-COORD : 29° 37’ 47.67"S
Y-COORD : 30° 24’ 57.00"E

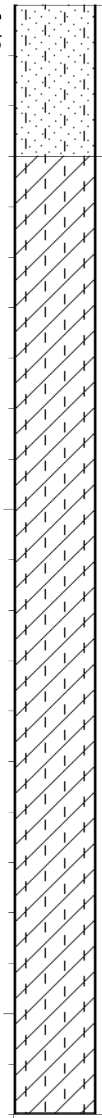
HOLE No: TP 05

MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

HOLE No: TP 06
Sheet 1 of 1

JOB NUMBER: UGE 03-22

Scale 1:15



0.00	Moist, dark grey, loose, silty fine grained SAND . **Aeolian**.
0.30	Moist, yellowish orange speckled red, intact, firm to stiff with depth, silty CLAY. **Alluvium**.
2.20	

NOTES

1) No sample taken.

2) No groundwater seepage noted.

3) Hand auger refused at 2.20m.

CONTRACTOR :
MACHINE : HAND&HAND AUGER
DRILLED BY :
PROFILED BY : TS SKOSANA

TYPE SET BY : TS PHEWA
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE : 23 APRIL 2022
DATE : 24 APRIL 2022

DATE : 07/05/2022 09:20
TEXT : ..ORTHEREPORT\LOGS\LOG.TXT

ELEVATION :
X-COORD : 28° 36’ 03.65"S
Y-COORD : 32° 05’ 39.70"E

HOLE No: TP 06

MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

HOLE No: TP 07
Sheet 1 of 1

JOB NUMBER: UGE 03-22

Scale 1:15

Depth	Description
0.00	Moist, yellowish orange speckled red, intact, firm to stiff with depth, silty CLAY. **Alluvium**.
2.50	

NOTES

1) No sample taken.

2) No groundwater seepage noted.

3) Hand auger refused at 2.20m.

CONTRACTOR :
MACHINE : HAND&HAND AUGER
DRILLED BY :
PROFILED BY : TS SKOSANA
TYPE SET BY : TS PHEWA
SETUP FILE : STANDARD.SET

INCLINATION :
DIAM :
DATE : 23 APRIL 2022
DATE : 24 APRIL 2022
DATE : 07/05/2022 09:20
TEXT : ..ORTHEREPORT\LOGS\LOG.TXT

ELEVATION :
X-COORD : 28° 36' 05.31"S
Y-COORD : 32° 05' 39.91"E

HOLE No: TP 07

MONDLI CONSULTING
KWAMBONAMBI SERVICE STATION

LEGEND
Sheet 1 of 1

JOB NUMBER: UGE 03-22

	SAND	{SA04}
	SILTY	{SA07}
	CLAY	{SA08}
	CLAYEY	{SA09}
Name	DISTURBED SAMPLE	{SA38}
5.5	WATER SEEPAGE/water strike	{CH50}

CONTRACTOR :
MACHINE :
DRILLED BY :
PROFILED BY :

INCLINATION :
DIAM :
DATE :
DATE :

ELEVATION :
X-COORD :
Y-COORD :

TYPE SET BY : TS PHEWA
SETUP FILE : STANDARD.SET

DATE : 07/05/2022 09:20
TEXT : ..ORTHEREPORT\LOGS\LOG.TXT

LEGEND
SUMMARY OF SYMBOLS

APPENDIX D:

LIGHT DYNAMIC PENETROMETER PROBE (DPL) TEST RESULTS

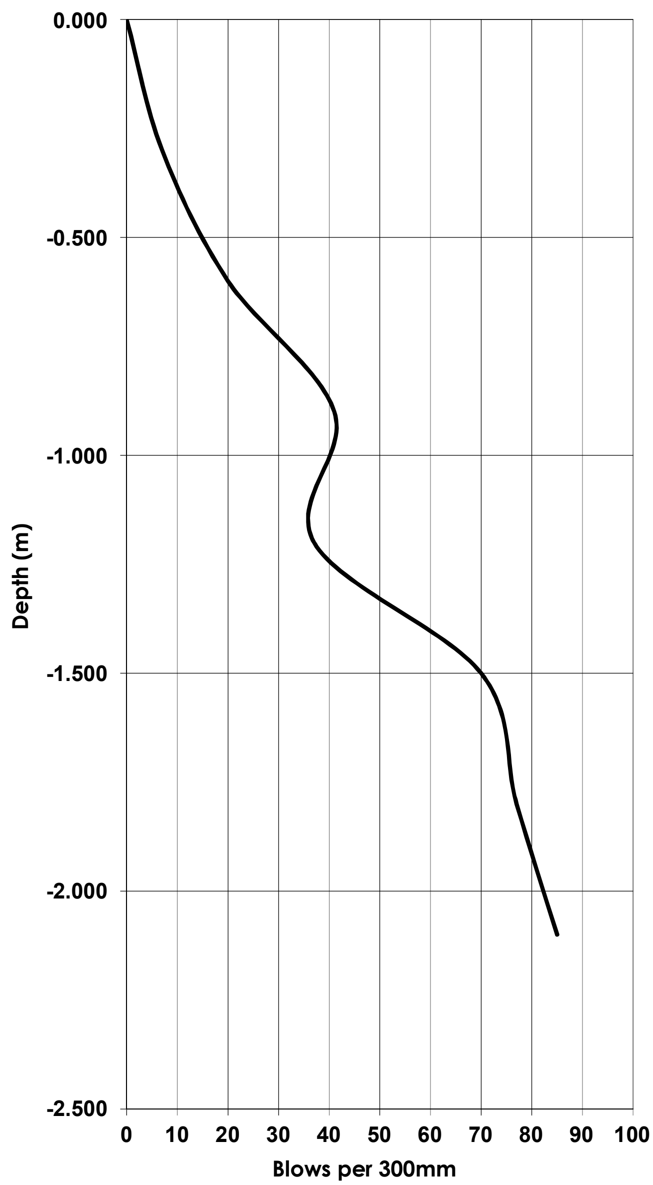
Client:	MONDLI CONSULTING	Ref.No.:	UGE-03-22
Project:	KWAMBONAMBI SERVICE STATION	Date:	23-Apr-22
Section:	RICHARDS BAY	Operator:	T.S SKOSANA

LIGHT DYNAMIC PENETROMETER PROBE (DPL) ***TEST No.: DPL 1***

THE INSITU STRENGTH DEPENDS ON SOIL MOISTURE CONTENT AND GRAIN STRUCTURE WHICH HAVE NOT BEEN ASSESSED AND MAY CHANGE. THE VALUES GIVEN ARE THEREFORE INDICATIVE ONLY AND SHOULD BE VERIFIED BY TEST OR OBSERVATION

Depth metres	Blows/ 300mm	Inferred Consistency	Insitu Shear Strength
0	0		
0.3	7	Loose	<30 deg
0.6	20	Med.Dense	32 deg
0.9	41	Dense	36 deg
1.2	37	Med.Dense	35 deg
1.5	70	Dense	37 deg
1.8	77	Dense	37 deg
2.1	85	Dense	38 deg
2.4	R		
2.7			
3			
3.3			
3.6			
3.9			
4.2			
4.5			
4.8			
5.1			
5.4			
5.7			
6			
6.3			
6.6			
6.9			
7.2			
7.5			
7.8			
8.1			
8.4			
8.7			
9			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12			

Hammer: 10kg falling 550mm
Cone: 25mm diameter with 60 degree apex angel
Rods: 16mm diameter, 22mm diameter couplings



UMHLABA GEOTECHNICAL ENGINEERS
123 Marine View Avenue
Durban North
Phone - 081 328 4756
info@umhlabageotech.co.za

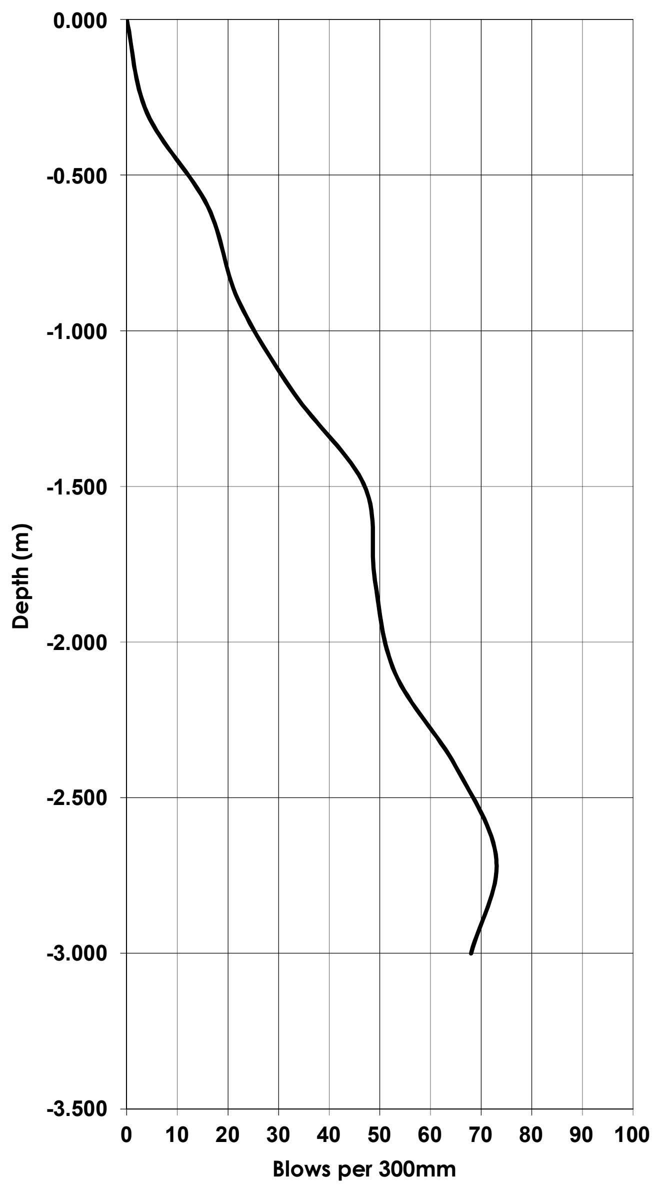
Client:	MONDLI CONSULTING	Ref.No.:	UGE-03-22
Project:	KWAMBONAMBI SERVICE STATION	Date:	23-Apr-22
Section:	RICHARDS BAY	Operator:	T.S SKOSANA

LIGHT DYNAMIC PENETROMETER PROBE (DPL) — ***TEST No.: DPL 2***

THE INSITU STRENGTH DEPENDS ON SOIL MOISTURE CONTENT AND GRAIN STRUCTURE WHICH HAVE NOT BEEN ASSESSED AND MAY CHANGE. THE VALUES GIVEN ARE THEREFORE INDICATIVE ONLY AND SHOULD BE VERIFIED BY TEST OR OBSERVATION

Depth metres	Blows/ 300mm	Inferred Consistency	Insitu Shear Strength
0	0		
0.3	4	Very Loose	<29 deg
0.6	16	Med.Dense	30 deg
0.9	22	Med.Dense	33 deg
1.2	33	Med.Dense	35 deg
1.5	47	Dense	36 deg
1.8	49	Dense	36 deg
2.1	53	Dense	37 deg
2.4	65	Dense	37 deg
2.7	73	Dense	37 deg
3	68	Dense	37 deg
3.3			
3.6			
3.9			
4.2			
4.5			
4.8			
5.1			
5.4			
5.7			
6			
6.3			
6.6			
6.9			
7.2			
7.5			
7.8			
8.1			
8.4			
8.7			
9			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12			

Hammer: 10kg falling 550mm
Cone: 25mm diameter with 60 degree apex angel
Rods: 16mm diameter, 22mm diameter couplings



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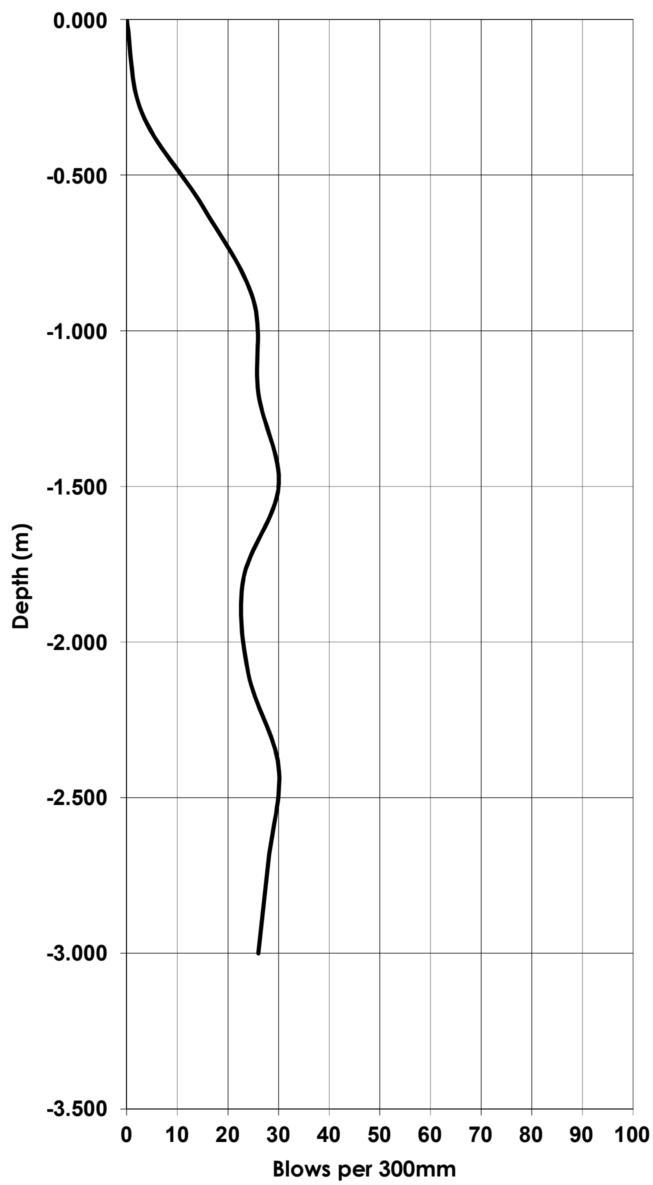
Client:	**MONDLI CONSULTING**	**Ref.No.:**	UGE-03-22
Project:	**KWAMBONAMBI SERVICE STATION**	**Date:**	23-Apr-22
Section:	**RICHARDS BAY**	**Operator:**	T.S SKOSANA

LIGHT DYNAMIC PENETROMETER PROBE (DPL) ***TEST No.: DPL 3***

THE INSITU STRENGTH DEPENDS ON SOIL MOISTURE CONTENT AND GRAIN STRUCTURE WHICH HAVE NOT BEEN ASSESSED AND MAY CHANGE. THE VALUES GIVEN ARE THEREFORE INDICATIVE ONLY AND SHOULD BE VERIFIED BY TEST OR OBSERVATION

Depth metres	Blows/ 300mm	Inferred Consistency	Insitu Shear Strength
0	0		
0.3	3	Very Loose	<29 deg
0.6	15	Loose	<30 deg
0.9	25	Med.Dense	33 deg
1.2	26	Med.Dense	34 deg
1.5	30	Med.Dense	34 deg
1.8	23	Med.Dense	33 deg
2.1	24	Med.Dense	33 deg
2.4	30	Med.Dense	34 deg
2.7	28	Med.Dense	34 deg
3	26	Med.Dense	34 deg
3.3			
3.6			
3.9			
4.2			
4.5			
4.8			
5.1			
5.4			
5.7			
6			
6.3			
6.6			
6.9			
7.2			
7.5			
7.8			
8.1			
8.4			
8.7			
9			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12			

Hammer: 10kg falling 550mm
Cone: 25mm diameter with 60 degree apex angel
Rods: 16mm diameter, 22mm diameter couplings



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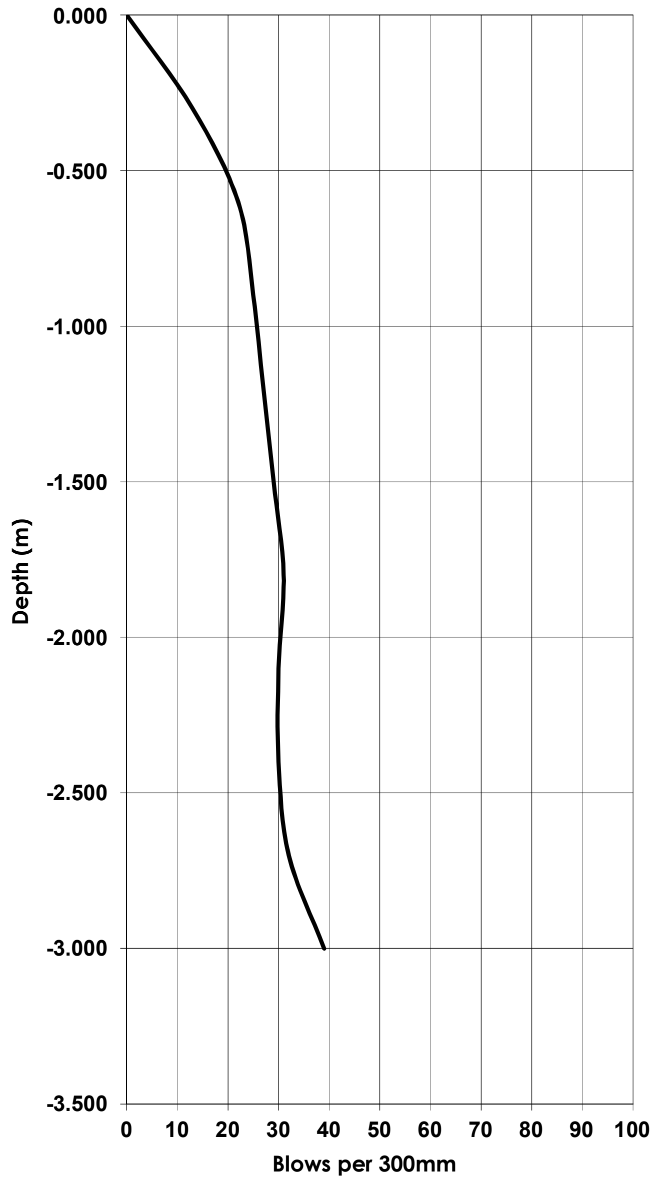
Client:	MONDLI CONSULTING	Ref.No.:	UGE-03-22
Project:	KWAMBONAMBI SERVICE STATION	Date:	23-Apr-22
Section:	RICHARDS BAY	Operator:	T.S SKOSANA

LIGHT DYNAMIC PENETROMETER PROBE (DPL) — ***TEST No.: DPL 4***

THE INSITU STRENGTH DEPENDS ON SOIL MOISTURE CONTENT AND GRAIN STRUCTURE WHICH HAVE NOT BEEN ASSESSED AND MAY CHANGE. THE VALUES GIVEN ARE THEREFORE INDICATIVE ONLY AND SHOULD BE VERIFIED BY TEST OR OBSERVATION

Depth metres	Blows/ 300mm	Inferred Consistency	Insitu Shear Strength
0	0		
0.3	13	Loose	<30 deg
0.6	22	Med.Dense	33 deg
0.9	25	Med.Dense	33 deg
1.2	27	Med.Dense	34 deg
1.5	29	Med.Dense	34 deg
1.8	31	Med.Dense	35 deg
2.1	30	Med.Dense	34 deg
2.4	30	Med.Dense	34 deg
2.7	32	Med.Dense	35 deg
3	39	Med.Dense	36 deg
3.3			
3.6			
3.9			
4.2			
4.5			
4.8			
5.1			
5.4			
5.7			
6			
6.3			
6.6			
6.9			
7.2			
7.5			
7.8			
8.1			
8.4			
8.7			
9			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12			

Hammer: 10kg falling 550mm
Cone: 25mm diameter with 60 degree apex angel
Rods: 16mm diameter, 22mm diameter couplings



UMHLABA GEOTECHNICAL ENGINEERS
123 Marine View Avenue
Durban North
Phone - 081 328 4756
info@umhlabageotech.co.za

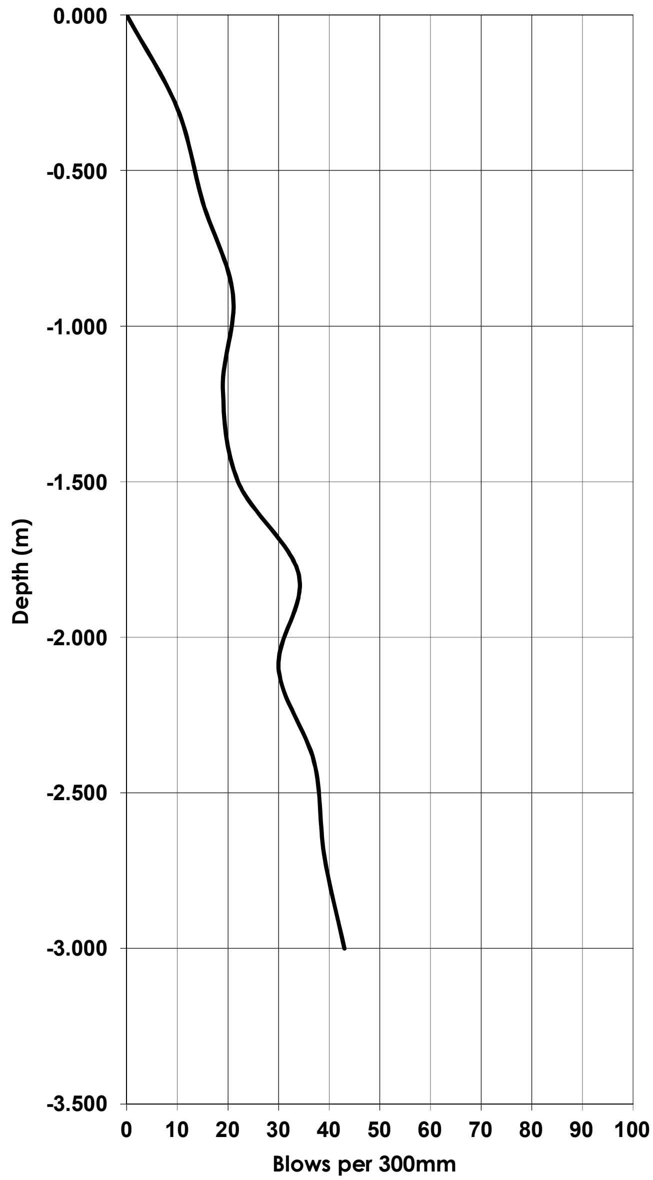
Client:	MONDLI CONSULTING	Ref.No.:	UGE-03-22
Project:	KWAMBONAMBI SERVICE STATION	Date:	23-Apr-22
Section:	RICHARDS BAY	Operator:	T.S SKOSANA

LIGHT DYNAMIC PENETROMETER PROBE (DPL) ***TEST No.: DPL 5***

THE INSITU STRENGTH DEPENDS ON SOIL MOISTURE CONTENT AND GRAIN STRUCTURE WHICH HAVE NOT BEEN ASSESSED AND MAY CHANGE. THE VALUES GIVEN ARE THEREFORE INDICATIVE ONLY AND SHOULD BE VERIFIED BY TEST OR OBSERVATION

Depth metres	Blows/ 300mm	Inferred Consistency	Insitu Shear Strength
0	0		
0.3	10	Loose	<30 deg
0.6	15	Loose	<30 deg
0.9	21	Med.Dense	32 deg
1.2	19	Med.Dense	32 deg
1.5	22	Med.Dense	33 deg
1.8	34	Med.Dense	35 deg
2.1	30	Med.Dense	34 deg
2.4	37	Med.Dense	35 deg
2.7	39	Med.Dense	36 deg
3	43	Dense	36 deg
3.3			
3.6			
3.9			
4.2			
4.5			
4.8			
5.1			
5.4			
5.7			
6			
6.3			
6.6			
6.9			
7.2			
7.5			
7.8			
8.1			
8.4			
8.7			
9			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12			

Hammer: 10kg falling 550mm
Cone: 25mm diameter with 60 degree apex angel
Rods: 16mm diameter, 22mm diameter couplings



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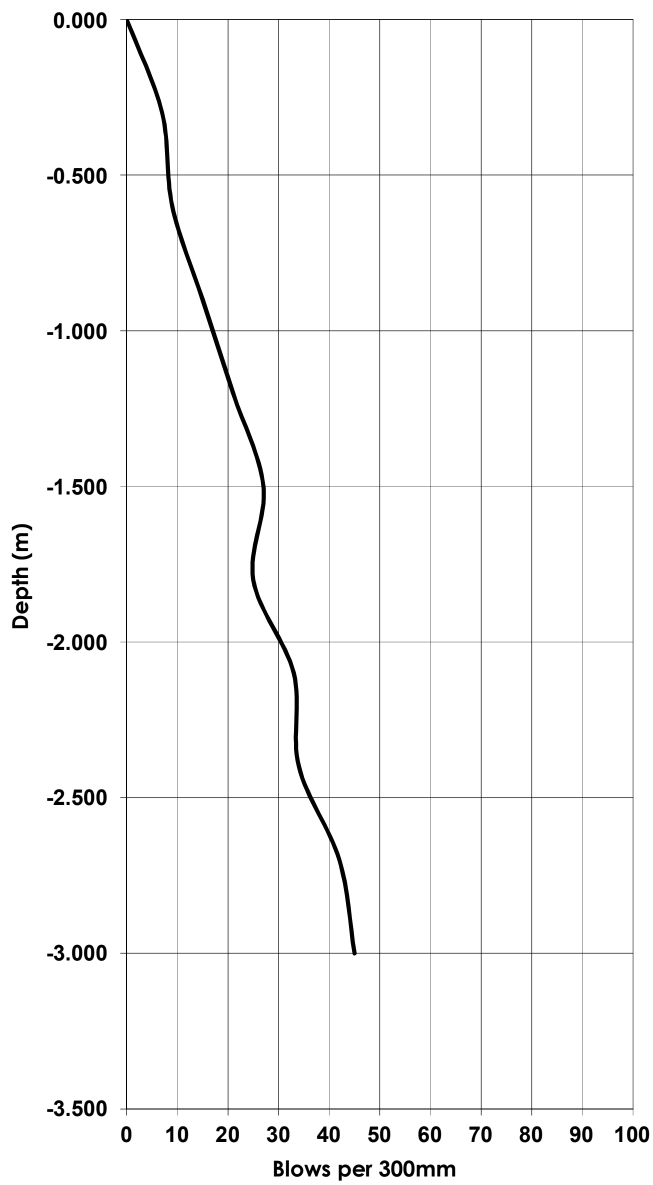
Client:	MONDLI CONSULTING	Ref.No.:	UGE-03-22
Project:	KWAMBONAMBI SERVICE STATION	Date:	23-Apr-22
Section:	RICHARDS BAY	Operator:	T.S SKOSANA

LIGHT DYNAMIC PENETROMETER PROBE (DPL) ***TEST No.: DPL 6***

THE INSITU STRENGTH DEPENDS ON SOIL MOISTURE CONTENT AND GRAIN STRUCTURE WHICH HAVE NOT BEEN ASSESSED AND MAY CHANGE.
THE VALUES GIVEN ARE THEREFORE INDICATIVE ONLY AND SHOULD BE VERIFIED BY TEST OR OBSERVATION

Depth metres	Blows/ 300mm	Inferred Consistency	Insitu Shear Strength
0	0		
0.3	7	Loose	<30 deg
0.6	9	Loose	<30 deg
0.9	15	Loose	<30 deg
1.2	21	Med.Dense	32 deg
1.5	27	Med.Dense	34 deg
1.8	25	Med.Dense	33 deg
2.1	33	Med.Dense	35 deg
2.4	34	Med.Dense	35 deg
2.7	42	Dense	36 deg
3	45	Dense	36 deg
3.3			
3.6			
3.9			
4.2			
4.5			
4.8			
5.1			
5.4			
5.7			
6			
6.3			
6.6			
6.9			
7.2			
7.5			
7.8			
8.1			
8.4			
8.7			
9			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12			

Hammer: 10kg falling 550mm
Cone: 25mm diameter with 60 degree apex angel
Rods: 16mm diameter, 22mm diameter couplings



UMHLABA GEOTECHNICAL ENGINEERS
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Durban North
Phone - 081 328 4756
info@umhlabageotech.co.za

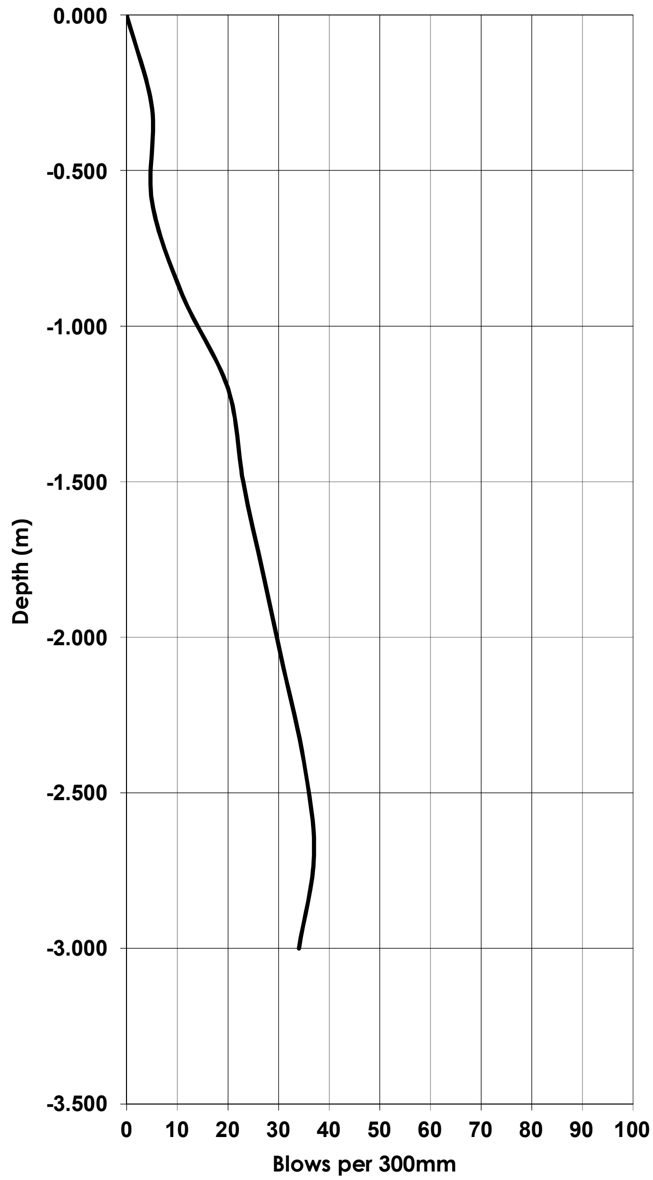
Client:	MONDLI CONSULTING	Ref.No.:	UGE-03-22
Project:	KWAMBONAMBI SERVICE STATION	Date:	23-Apr-22
Section:	RICHARDS BAY	Operator:	T.S SKOSANA

LIGHT DYNAMIC PENETROMETER PROBE (DPL) — *TEST No.: DPL 7*

THE INSITU STRENGTH DEPENDS ON SOIL MOISTURE CONTENT AND GRAIN STRUCTURE WHICH HAVE NOT BEEN ASSESSED AND MAY CHANGE. THE VALUES GIVEN ARE THEREFORE INDICATIVE ONLY AND SHOULD BE VERIFIED BY TEST OR OBSERVATION

Depth metres	Blows/ 300mm	Inferred Consistency	Insitu Shear Strength
0	0		
0.3	5	Very Loose	<29 deg
0.6	5	Very Loose	<29 deg
0.9	11	Loose	<30 deg
1.2	20	Med.Dense	32 deg
1.5	23	Med.Dense	33 deg
1.8	27	Med.Dense	34 deg
2.1	31	Med.Dense	35 deg
2.4	35	Med.Dense	35 deg
2.7	37	Med.Dense	35 deg
3	34	Med.Dense	35 deg
3.3			
3.6			
3.9			
4.2			
4.5			
4.8			
5.1			
5.4			
5.7			
6			
6.3			
6.6			
6.9			
7.2			
7.5			
7.8			
8.1			
8.4			
8.7			
9			
9.3			
9.6			
9.9			
10.2			
10.5			
10.8			
11.1			
11.4			
11.7			
12			

Hammer: 10kg falling 550mm
Cone: 25mm diameter with 60 degree apex angel
Rods: 16mm diameter, 22mm diameter couplings



UMHLABA GEOTECHNICAL ENGINEERS
123 Marine View Avenue
Durban North
Phone - 081 328 4756
info@umhlabageotech.co.za

APPENDIX E:

LABORATORY RESULTS

SNALAB CIVIL ENGINEERING LABORATORY

SNA CIVIL AND STRUCTURAL ENGINEERS (PTY) LTD
13 PESETA PARADE, ARBORETUM RICHARDS BAY
PO Box 10191, Meerensee 3901
Tel :(035) 789 7105/6/7
E-MAIL : rb@sna.co.za
REG No. 2005/006128/07

TEST REPORT

REPORT NUMBER
91651/2022

Client : UMHLABA GEOTECHNICAL ENGINEERS

Address: 123 Marine View Avenue
Durban North
4051

Cell : 078 378 0327
Tel: No Information Provided
Fax: No Information Provided

ATTENTION: Mr T.S Phewa

Project/Order: Kwambonambi Service Station / Cash

Brief : **Design:** - Determine 2 No. Foundation Indicator. 3 No. MDD/OMC, CBR, pH+ for the samples provided.

Date requested	17/06/2022
Date sampled	SAMPLED BY CLIENT
Date received	17/06/2022
Date tested	Refer attached report/s
Location of sampling	SAMPLED BY CLIENT
Sampling method/methods	SAMPLED BY CLIENT
Sampling plan	SAMPLED BY CLIENT
Sampled by	SAMPLED BY CLIENT
Sample number	Refer attached report/s
Sample description	Refer attached report/s
Sample condition	Slightly moist, uncontaminated
Sample classification	Refer attached report/s
Sampling Environmental condition	SAMPLED BY CLIENT
Test Method/Methods used	Refer attached report/s
Test done at	SNALAB R/Bay
Deviation to Test Method's :	**Will be noted on test report sheets/s, as applicable.**

Test report/reports shall not be reproduced except in full, without the written approval of the Laboratory.
This test report relates only to samples received.
If the report is referred to as an INTERIM REPORT it is not fit for publication.
Terms and conditions of business are contained on the reverse side of the report.
Information above noted as "**Supplied by Client/Sampled by Client**" may effect the validity of the test results.

Christiaan Swart
Laboratory Manager

04/07/2022
DATE:

SOIL TEST REPORT

SNALAB CIVIL ENGINEERING LABORATORY			
Client:	UMHLABA GEOTECHNICAL ENGINEERS	Lab No:	REFER BELOW
Project Name/No:	KWAMBONAMBI SERVICE STATION	Client No :	91651/2022
Section from:	REFER BELOW	Date :	29/06/2022

Criteria REFER PROJ SPEC

Laboratory & Field Data		**RESULTS**			
Laboratory No		88996	88997	88998	88999
Field No		**TP1**	**TP3**	**TP4**	**TP5**
Source		INSITU	INSITU	INSITU	INSITU
Depth	**m**	0.00 - 2.50	0.00 - 1.60	0.60 - 2.40	0.30 - 2.00
Description					
Sampled by		CLIENT	CLIENT	CLIENT	CLIENT
SIEVE + HYDRO.ANALYSIS (SANS 3001-GR1) / ASTM D422	% P				
63 mm					
53 mm					
37.5 mm					
26.5 mm					
19 mm				100	
13.2 mm				97	
4.75 mm				94	100
2.00 mm				92	99
0.600 mm				89	99
0.425 mm				86	98
0.075 mm				34	42
0.060 mm				34	39
0.050 mm				33	39
0.026 mm				31	37
0.015 mm				31	35
0.010 mm				31	35
0.0050 mm				31	31
0.0020 mm				28	28
0.0015 mm				28	28
Grading Modulus (SANS 3001 - PR5)				0.88	0.61
SOIL TYPE DISTRIBUTION (M.I.T)	**%**				
Gravel (> 2.0mm)				8	1
Coarse Sand (2.0 - 0.6mm)				3	0
Medium Sand (0.6 - 0.2mm)				26	20
Fine Sand (0.2 - 0.06mm)				29	39
Silt (0.06 - 0.002mm)				6	12
Clay (<0.002mm)				28	28
SOIL CONSTANTS (SANS 3001 - GR10)					
Liquid Limit	**%**			29	32
Plasticity Index	-			8	12
Linear Shrinkage	**%**			4.3	6.3
MDD/OMC (SANS 3001 - GR30)					
Maximum Dry Density		1827	1688		1874
Opt. Moisture Content		9.1	10.2		12.1
CBR (SANS GR40)					
CBR at : 100% Compaction		21	26		5.2
: 98% Compaction		15	14		3.6
: 95% Compaction		9	6		3.0
: 93% Compaction		6	3		1.6
: 90% Compaction		4	1		1.1
Swell @ 100% Compaction		0.1	0.1		1.2
OTHER					
pH **(SANS 3001 GR57)**	**%**	5.7		7.1	6.6
Electric Conductivity **(TMH1 A21T)**	**S/m**	0.0053		0.0472	0.0306
CLASSIFICATION					
AASHTO SCS & Group Index	-			A-2-4 (0)	A-6 (2)
	-			UNSUITABLE	UNSUITABLE

Notes :
This report relates only to the specific sample(s) tested herein.
% P denotes "percentage passing"

Remarks:

Date: 04/07/2022
Signature: