Geotechnical Investigation Report

October 2024

Investigation for the Proposed upgrade of the Trenance 3 Reservoir site in Ward 59.

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EGE2024-19: Assessment for the upgrade of the Trenance 3 Reservoir site in Ward 59- KZN Province	
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Executive Summary

EThekwini Municipality appointed Elite Geotech and Enviro Construction Services to conduct a geotechnical investigation for the proposed upgrade of the Trenance 3 Reservoir site located in Ward 59 eThekwini Municipality, KwaZulu-Natal Province.

The proposed development will comprise of the following main infrastructure components:

- A 6 Ml water reservoir;
- A 0.4Ml elevated tanks;
- Pump station; and
- Associated pipelines.

The field investigation was conducted on the 10th to 17th October 2024 and comprised of hand excavation of eight (8 No) test pits to a maximum depth of 1.50m. The fieldwork also included drilling and core logging of seven (7 No) boreholes and collecting representative samples for laboratory testing.

The geotechnical investigation revealed that the profile across the site comprises the following horizons:

- Fill layer;
- Transported Layer;
- Residual Layer; and
- Sandstone Bedrock.

The area is underlain by collapsible and compressible silty sandy material at a depth ranging from 2.50m to about 3.50m, then sandstone to the depth beyond 10.00m.

The residual sandstone and sandstone material underlying the site is **G7** according to the TRH 14 guidelines (CSIR: 1987), therefore it may be suitable for use in the construction of selected subgrade layer material and in moderate stiffness engineered fill. This material may also be used for the construction of an engineered fill of selected subgrade layer material and in low stiffness of engineered fills where it is encountered as **G9.**

The proposed development comprises of light and heavy structures i.e. associated pipelines, a Pump Station, 0.4Ml Elevated Tanks, and a 6Ml Water Reservoir. Of important, the Water Reservoir will exact significant loads onto the ground. The site subsurface conditions are favourable for the proposed developments provided that the recommendations within this report are adhered to.

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1 Introduction

EThekwini Municipality appointed Elite Geotech and Enviro Construction Services to conduct a geotechnical investigation for the proposed upgrade of the water reservoir at the Trenance 3 Reservoir site located in Ward 59 eThekwini Municipality in KwaZulu-Natal.

The proposed development will comprise of the following main infrastructure components:

- A 6 Ml water reservoir;
- 0.4Ml elevated tanks;
- Pump station; and
- Associated pipelines.

The objective of the investigation was to investigate founding conditions at the site (within the limits afforded by the method of investigation) in terms of the following:

- Providing an overview of the geology of the site.
- Describing the soil and rock profiles encountered.
- Evaluating engineering properties of the in-situ materials.
- Assessment of the potential for re-use of materials during construction.
- Presenting findings, geotechnical considerations, and recommendations that may have an influence on the design and construction of the proposed structures.

The geotechnical investigation entailed the following.

- A field investigation comprising surveying, test pits excavations, DCP testing, rotary core drilling with Standard Penetration Test (SPT), soil profiling/core logging, and Point Load Testing (PLT).
- Representative soil samples were collected from test pits excavated on the site and submitted for laboratory testing aimed to classifying the soils and to determine the suitability of the soils.
- The evaluation of the expected bearing capacity, determination of excavatability, and evaluation of the corrosiveness of soil.

To meet the requirements for the investigation, the investigation was conducted as per the South African Institute of Civil Engineering Code of Practice (SAICE, 2010).

2 Available information

At the time of the investigation the following information was available:

- The 1:250 000 scale geological map of the Durban Sheet 2930 (Council for Geoscience, 1986).
- The 1:250 000 scale soil map of the Durban Sheet 2930 (Soil and Research Institute, 1998).
- Aerial photographs, sourced from Google Earth.
- Locality plans which indicate the extent of the investigated section.

3 Site description

3.1 Site Locality

The proposed development is located at Trenance 3 Reservoir in Ward 59 eThekwini Municipality in KwaZulu Natal Province. It is accessible via Jabu Ngcobo Drive (M27), and by taking the Cottonwood Drive off-ramp, turn left to Madrona Drive to get to Trenance 3 Reservoir. The proposed development has a reservoir and associated infrastructure is shown in Figure 1 below.

Figure 1: Showing the investigated area (red boundary) for the proposed development.

3.2 Topography and vegetation

The investigated area for the proposed development is on a hill, gently sloping towards to all directions. The topography and elevation at the site are shown in Figure 2 below. At the time of the investigation, the site was covered by grass and shrubs, and existing structures and access road paving were noted at the site.

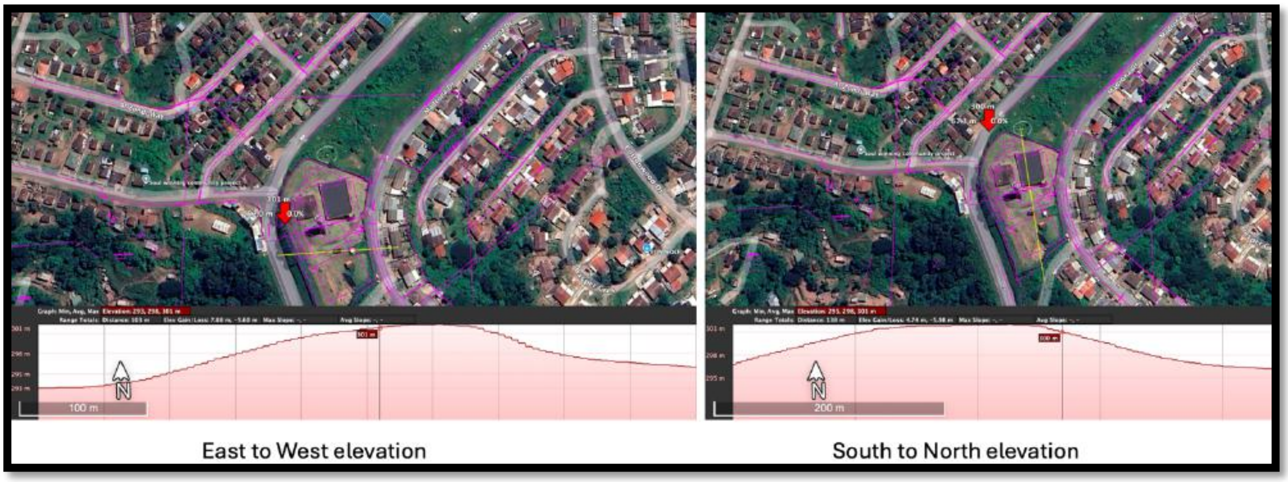

Figure 2: Showing the topography of the investigated site.

3.3 Climate

The Verulam area lies about 119.83 m above sea level. It has a warm summer climate. The average temperature in the area is 18.8°C. It normally receives about 766 mm of rain per year. It receives the lowest rainfall (13 mm) in June and the highest (108 mm) in February. February is the warmest month with a midday average of 27.4°C, and July is the coldest month, with midday temperatures averaging 22. °C (Climate-Data.Org: 2024).

The Weinert Climatic N-value for the area (Weinert, 1980) is <5 indicating that the climate is semi-humid and chemical weathering processes are dominant.

3.4 Seismicity Assessment

On the published seismic hazard figure of South Africa (SANS 10160-4:2011) the seismic hazard is defined in terms of peak ground acceleration. In South Africa two seismic zones are apparent: Zone I for natural seismic activity and Zone II for regions of mining-induced and natural seismic activity.

According to the seismic hazard map of SANS 10160-4 (2011), the value for the peak ground acceleration of the investigated site occurs in an area with a value of approximately 0.10 g, with a

10% probability that this value will be exceeded in a 50-year period as shown in Figure 3 below. In accordance with SANS 10160-4:2011, the site does not fall within either Zone I or Zone II and no specific seismic design requirements are therefore required.

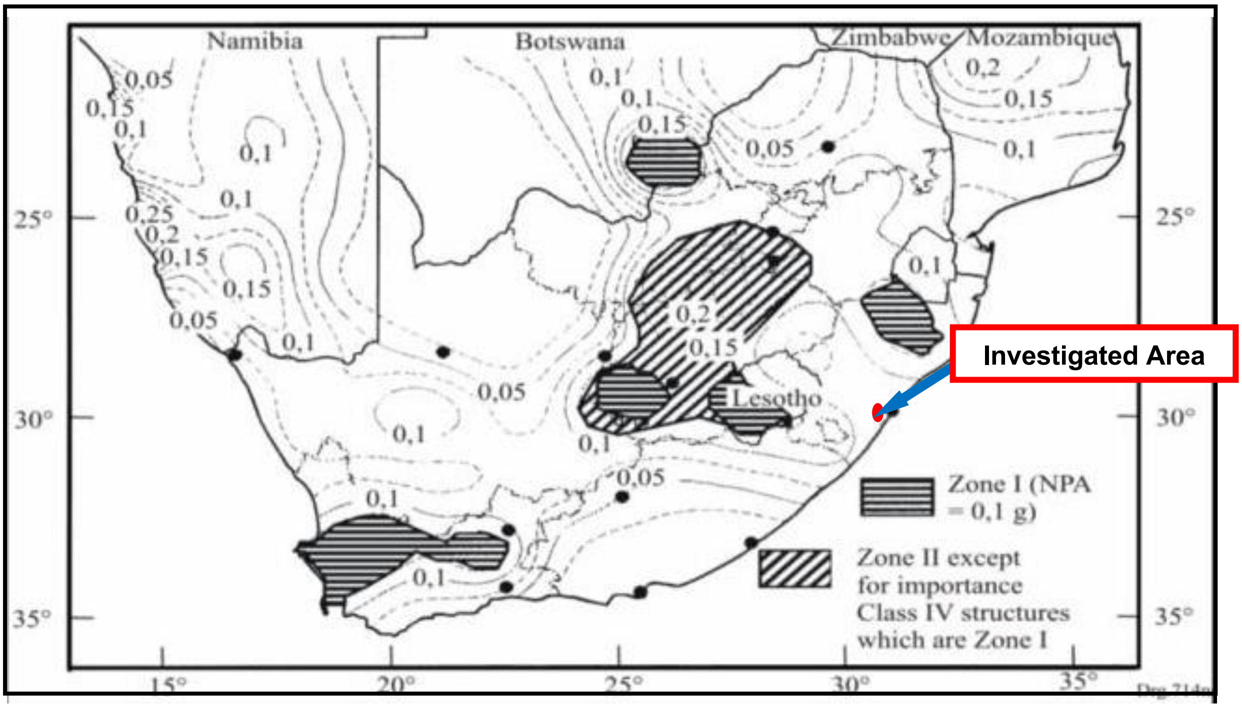

Figure 3: Locality of the site on the seismic hazard map of South Africa.

The peak ground acceleration expresses the seismic hazard and the value of 0.125 m/s^2 may be considered a medium level of seismic hazard. A 10% probability exists that this value will be exceeded in a 50-year period.

4 Geology

4.1 Regional geology

According to the published 1:250 000 geological map of Durban Sheet 2930 (Council for Geoscience, 1986), the site is underlain by the Natal Group (O-Sn) sedimentary rocks, with the lithology consisting of red-brown coarse-grained arkosic to subarkosic sandstone; micaceous sandstone; subordinate siltstone and mudstone. The geological map shows that there is a fault trending NE-SW at a distance from the north-east boundary of the site area. However, it was not encountered during the site investigation. Figure 4 below shows the geological map of the investigated area.

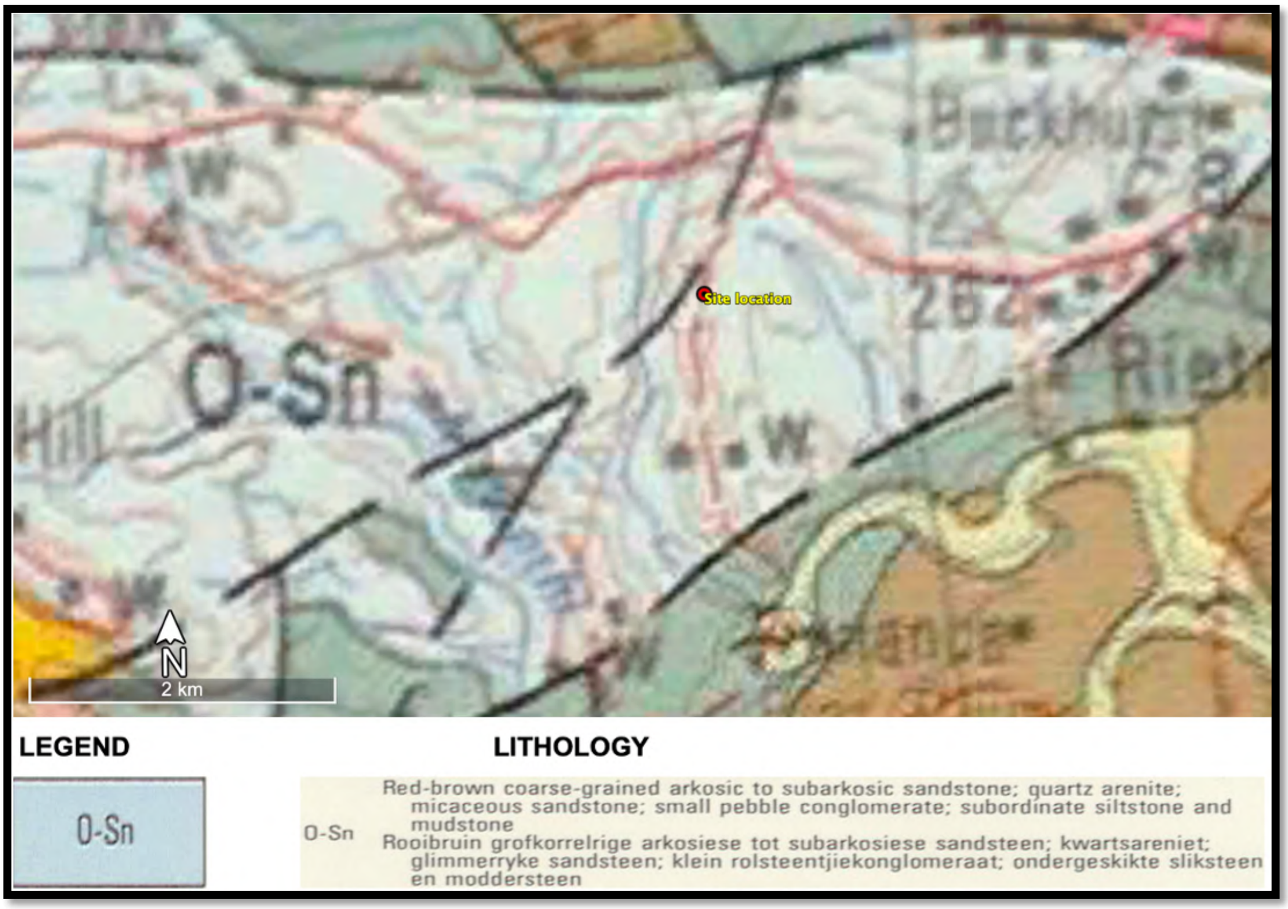

Figure 4: Showing the geological map of the study area; (Geological Survey, printed by the Government Printer, Pretoria, 1986).

4.2 Site geology

The results from the boreholes drilled on site reveal that the area is underlain by the fill layer, followed by the minor patches of the transported layer, residual layer, and then bedrock. The geological North - South cross-section through the site is shown in Figure 6 below, while the line (redline) where the cross section is drawn from is shown in Figure 5 below.

Figure 5: Showing the line where the below geological cross-section was drawn from

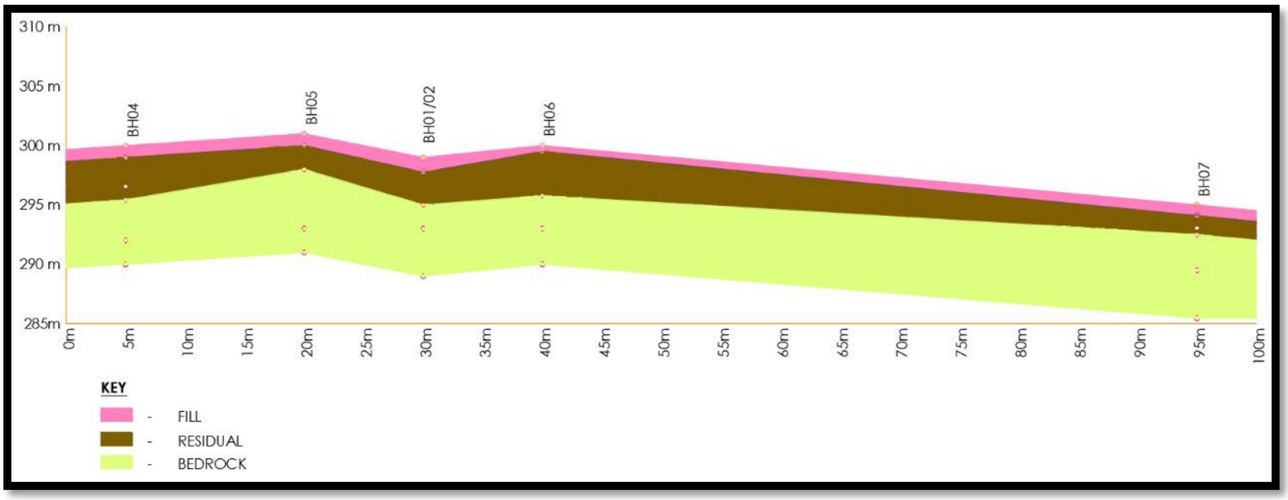

Figure 6: The N-S geological cross section across the site.

5 Investigation Methodology

The geotechnical study was carried out in phases. The first phase was a desktop study, which was followed by a second phase of fieldwork. The desktop study commenced before the fieldwork. Reporting and analysis followed.

5.1 Desktop study

The desk study of available geological information involved the pursuing of aerial images, published geological maps and relevant literature. The purpose of the study was to give technical guidance on the expected geological and geotechnical conditions on site and to define the geotechnical field investigation scope of works.

5.2 Fieldwork

The fieldwork comprised of the following:

- Walk-over survey.
- Marking of borehole positions.
- Borehole drilling and core logging.
- Excavation and profiling of test pits.
- In-situ testing, and
- Collection of representative soil samples for laboratory testing.

5.2.1 Walk over survey

Following the desktop study, a site walkover was undertaken at the proposed pipeline route. This was done to assess the current topographical and geological conditions from the surface without any intrusive work. This was conducted by the Elite geologists and the client's representative.

5.2.2 Marking of borehole positions

The primary objective of marking boreholes is to establish a clear and accurate spatial framework for subsurface exploration. This is essential in ensuring that each borehole is strategically and accurately positioned to yield the most informative data regarding the geologic conditions at the site.

The importance of a professional surveyor marking boreholes on a site, emphasizes both precision and the multifaceted nature of geological investigations. Figure 7 below shows the surveyor marking the borehole positions at the site

Figure 7: Showing the surveyor marking borehole positions at the site.

5.2.3 Borehole drilling

Seven (7 No) rotary core borehole drilling was carried out by a specialist geotechnical drilling contractor, in accordance with accepted South African Standards (CSRA, 1993. Standard Specifications for subsurface Investigations). The borehole was drilled through the sandstone bedrock to a depth of 10m. The borehole was logged in accordance with accepted South African practice (South African National Standard. Profiling, Percussion Borehole, and Core Logging in Southern Africa SANS 633:2012).

The position of the borehole is listed below in Table 1 with the detailed borehole log attached in Appendix B. The Figure 8 below shows the borehole drilling equipment during the drilling proposes at the site.

Figure 8: Showing the borehole drilling equipment at the site.

Table 1: Borehole summary

Test Pit No.	Coordinates (WGS84)		Final Depth (m)
	Latitude	Longitude	
BH01	29°39'9.14"S	30°59'46.76"E	10.14
BH02	29°39'8.62"S	30°59'47.73"E	10.00
BH03	29°39'9.30"S	30°59'48.10"E	10.00
BH04	29°39'9.32"S	30°59'47.15"E	10.14
BH05	29°39'9.32"S	30°59'47.51"E	10.00
BH06	29°39'8.73"S	30°59'46.50"E	10.21
BH07	29°39'6.82"S	30°59'46.66"E	9.55

5.2.4 Test Pitting

The fieldwork comprised excavation and profiling of test pits within the footprint of the proposed development. A total of eight (8 No) test pits were hand excavated and augured to the maximum depth of 1.50m or refusal on hard material or until the sidewall stability of a test pit was judged to be unsafe.

A two-person team carried out the test pitting to comply with accepted safety requirements as in the South African Code of Practice and the South African Institution of Civil Engineering – Geotechnical Division (SAICE, 2007). The test pits were set out and profiled by a team of engineering geologists/ geotechnical engineers following accepted South African standards (Jennings, et al, 1973).

The details of the test pits are summarised in Table 2 below, and the detailed test pit soil profiles are attached in Appendix B. Figure 9 below shows the test pit and borehole positions at the site.

Table 2: Test pit summary

Test Pit No.	Coordinates (WGS84)		Final Depth (m)	Soil Temperature (°C)	Remarks
	Latitude	Longitude			
TP1	29°39'9.42"S	30°59'47.06"E	1.50	23.1	No refusal
TP2	29°39'9.09"S	30°59'47.95"E	1.50	23.1	No refusal
TP3	29°39'8.83"S	30°59'46.62"E	1.50	23.2	No refusal
TP4	29°39'8.45"S	30°59'47.97"E	1.50	23.0	No refusal
TP5	29°39'7.72"S	30°59'45.97"E	1.50	23.2	No refusal
TP6	29°39'7.76"S	30°59'48.02"E	1.20	23.0	Refusal on sandstone bedrock
TP7	29°39'6.92"S	30°59'47.65"E	1.50	23.1	No refusal
TP8	29°39'6.96"S	30°59'46.52"E	1.50	23.1	No refusal

Figure 9: Showing the test pit (red dots) and borehole (yellow dots) position at the site.

5.2.5 Sampling

Representative disturbed soil samples from the different soil layers encountered on the sites were taken to a SANAS-accredited laboratory to conduct the material property testing and characterisation of the samples' engineering properties.

5.2.6 In-situ testing

The in-situ field testing was conducted using Standard Penetration Test (SPT) with each borehole and Dynamic Cone Penetrometer (DCP) tests adjacent to each test pit. The SPTs and DCPs were conducted to determine the consistency of the in-situ material to the maximum depth of 3.5m. The N-values from the SPT results are presented in Appendix B and the DCP results are attached in Appendix D of this report. Figure 10 below shows a typical DCP undertaken adjacent to the excavated test pit on site

Figure 10: Showing the DCP testing undertaken at the site.

5.3 Laboratory testing

The collected samples were taken to a SANAS accredited laboratory for soil testing. The following tests conducted were:

- Foundation Indicators tests comprising of sieve and hydrometer grading analyses and Atterberg Limits.
- MOD/CBR
- Chemical tests to determine pH and conductivity.
- Point load tests to determine rock strength.

6 Field Investigation Results

The geotechnical investigation revealed that the profile across the site comprises the following layers:

- Fill layer.
- Transported layer.
- Residual layer.
- Sandstone bedrock.

6.1 Fill layer

The fill layer was encountered in all the excavated test pits at the site, except for test pit 6, this layer was described as moist, light greyish brown and light purplish brown, gravelly silty sand with traces of soft sandstone fragments and root. The layer has a loose to medium-dense consistency.

6.2 Transported layer

The transported layer was removed in most of the test pits due to the previous earthworks activity. It was intercepted overlain by a fill layer at the site and was described as moist greyish brown, silty sand with roots. The consistency was profiled as being loose to medium dense. This layer has a thickness of 0.40m.

6.3 Residual layer

The residual layer, which is present throughout the site, comprises of light purplish brown, silty sand with traces of very soft sandstone fragments. The consistency was profiled as medium dense. This layer has a thickness ranging from 1.35m to 3.72m across the boreholes.

6.4 Sandstone bedrock

The light purplish brown, highly to moderately weathered, soft rock to medium hard rock with depth, and closely jointed to medium jointed rock. The rock horizon extends to depths beyond 10.00m.

The detailed descriptions of the soil profiles encountered in the excavated test pits are presented in Appendix B, while the geological profiles are summarised below for the whole site, based on the soil profiles. The geological profiles as recorded in the test pits and the borehole are summarised in Table 3 and Table 4 below, whilst Figure 11 shows the typical soil profile from the test pit at the site, and Figure 12 shows the typical core from the borehole drilled at the site.

Table 3: Test Pit Profile Summary

Test Pit No	Fill layer (m)	Transported layer (m)	Residual layer (m)	Highly weathered Sandstone bedrock
TP1	0 – 1.40		1.40 – 1.50	
TP2	0 – 1.30		1.30 – 1.50	
TP3	0 – 1.00		1.00 – 1.50	
TP4	0 – 1.35		1.35 – 1.50	
TP5	0 – 1.10		1.10 – 1.50	
TP6		0 – 0.30	0.30 – 1.00	1.00 – 1.20
TP7	0 – 1.00	1.0 – 1.50		
TP8	0 – 1.10		1.10 – 1.50	

Table 4: Borehole Profile Summary

Borehole No	Fill layer (m)	Residual layer (m)	Highly weathered Rock (m)	Slightly weathered rock (m)
BH01	0 – 1.50	1.50 – 3.45	3.45 – 6.00	6.00 – 10.14
BH02	0 – 1.00	1.00 – 4.00	4.00 – 6.00	6.00 – 10.00
BH03	0 – 0.90	0.90 – 3.45	3.45 – 4.77	4.77 – 10.00
BH04	0 – 1.00	1.00 – 4.57	4.57 – 8.00	8.00 – 10.14
BH05	0 – 1.00	1.00 – 3.00	3.00 – 8.00	8.00 – 10.00
BH06	0 – 0.50	0.50 – 4.22	4.22 – 7.00	7.00 – 10.21
BH07	0 – 0.90	0.90 – 2.50	2.50 – 5.50	5.50 – 9.55

Figure 11: Showing the fill, transported, residual material, and sandstone exposed in the test pit at the site.

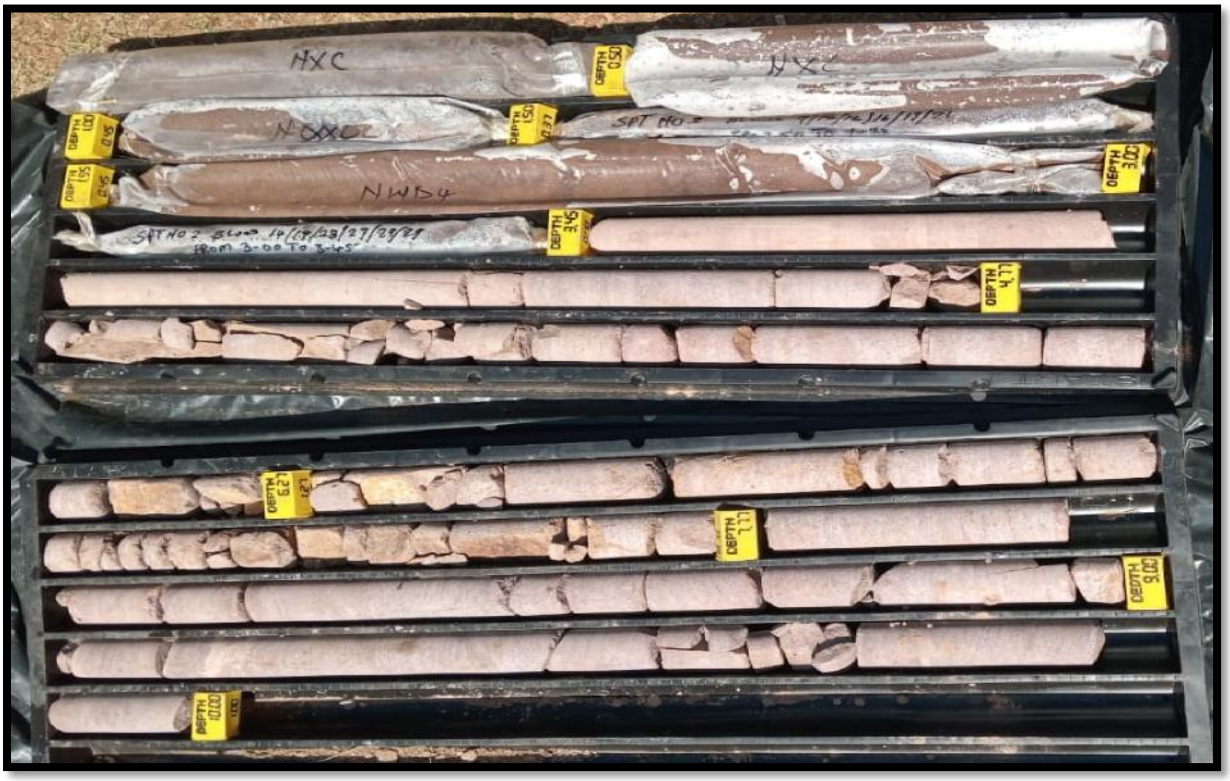

Figure 12: Showing the core from the borehole drilled at the site

6.5 In-situ Testing Results

6.5.1 Dynamic Cone Penetrometer DCP

The results from the DCP tests conducted adjacent to the test pit on site reveal that the Fill, transported, and residual layer is medium dense, refusal was encountered on a hard material at an average depth of 2.0m. The DCP results are attached in Appendix D of this report.

6.5.2 Standard Penetration Tests (SPT)

Standard Penetration Test is conducted by driving a standard 50 mm outside diameter thick-walled sampler into the soil at the bottom of a borehole, using repeated blows of a 63.5 kg hammer falling through 760 mm. The SPT N-value is the number of blows required to achieve a penetration of 300 mm, after an initial seating drive of 150 mm.

Standard Penetration Tests (SPTs) were conducted at 1.5m intervals in each borehole. The test results recorded on the borehole profile descriptions are summarised in Table 5 below:
A guideline for the relationship between the N-values and soil consistency is given in Table 6 below.

Table 5: SPT Results for boreholes drilled at the site

	SPT N-value						
Depth	BH1	BH2	BH3	BH4	BH5	BH6	BH7
1.50	26	R	R	26	R	R	R
3.00	47		R	41			

Table 6: SPT N-value correlation with consistency of soil

Cohesive soils		Non-Cohesive Soils	
N –value	Material description	N –value	Material description
< 2	Very soft	< 5	Very loose
2 – 4	Soft	5 – 10	Loose
4 – 8	Firm	10 – 30	Medium dense
8 – 15	Stiff	30 – 50	Dense
15 – 30	Very stiff	> 50	Very dense
R	Refusal	R	Refusal

6.6 Groundwater conditions

Groundwater seepage was not encountered in the excavated test pits or the drilled boreholes during the site investigation. This absence of seepage can be attributed to the site's elevated position on the hill, which typically results in a lower likelihood of groundwater presence at shallow depths. However, it is important to note that the groundwater table may exhibit seasonal fluctuations, influenced by variations in precipitation and other hydrological factors.

7 Laboratory Test Results

7.1 Foundation Indicators

Representative samples of selected horizons were collected for laboratory testing and submitted for foundation indicator tests. The test results are attached in Appendix C and are summarised in Table 7 below.

Table 7: Summary of section foundation indicator test results

Test Pit	Depth (m)	Activity	Particle size (%)				Atterberg's Limits %			GM	Unified Classification
			Clay	Silt	Sand	Gravel	LL	PI	LS		
Transported Layer											
TP7	1.0 – 1.50	Low	7.7	10.3	77.7	4.3	18	0	0	1.13	SM
Residual Layer											
TP1	1.40 – 1.50	Low	10.3	19.5	67.6	2.7	36	9	4.7	1.03	SM
TP5	1.10 – 1.50	Low	6.2	6.2	50	19.8	19	0	0	1.77	SM

Where:
GM = Grading modulus
LL = Liquid Limit
PI = Weighted Plasticity Index (PI x % passing the 0.425 mm sieve)
LS = Linear Shrinkage
Activity = Expansiveness of the soil according to Van der Merwe's method
SM = Silty sand mixture

Table 7 above indicates that:

The **transported layer** at the site generally consists of the silty sandy mixture (**SM**). The layers have a high (1.13) grading modulus. The fine fractions of this material also exhibit a low liquid limit as well as a very low linear shrinkage. The Plasticity index (PI) of the soil is very low. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **residual layer** at the site generally consists of the silty sandy mixture (**SM**). The layers have a high to very high (1.03-1.77) grading moduli. The fine fractions of this material also exhibit a low to moderate liquid limit as well as a very low to low linear shrinkage. The weighted plasticity index

(WPI) of the soil is very low to low. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

7.2 Compaction Tests

Samples of materials identified as potential sources of construction materials were sampled for laboratory testing. The samples were subjected to compaction tests in which the moisture-density relationship was established, with Californian Bearing Ratio (CBR) tests carried out to determine the suitability of the soils for use in constructing layer works below paved areas. The test results are attached in Appendix C and are summarised in Table 8 below:

Hole no.	Depth (m)	OMC (%)	MDD (kg/m^3)	Swell (%)	CBR at various densities				TRH 14 Class
					90 %	93 %	95 %	98 %	
Residual Sandstone layer									
TP01	1.40 – 1.50	11.7	1875	1.03	6.0	10	13	22	G9
Sandstone									
TP06	1.0 – 1.20	7.80	1928	1.13	20	32	44	65	G-7

Table 8: Summary of section compaction test results

Where:

OMC	=	Optimum moisture content
MDD	=	Maximum dry density (Mod AASHTO)
Swell	=	Soaked at 100% Mod AASHTO compaction

The residual sandstone material underlying the site has a moderate (1875kg/m^3) maximum dry density and moderate (11.7%) optimum moisture content value. The swell is high (1.03%), and the tests yielded low CBR values at densities typically specified in the field (93% to 95%). The material is classifiable according to the TRH 14 (CSIR: 1987) guidelines (**G9**).

The soft sandstone material underlying the site has a moderate (1928kg/m^3) maximum dry density and low (7.80%) optimum moisture content value. The swell is high (1.13%), and the tests yielded very moderate to high CBR values at densities typically specified in the field (93% to 95%). The material is classifiable according to the TRH 14 (CSIR: 1987) guidelines (**G7**).

The sandstone material that is **G7** according to the TRH 14 guidelines (CSIR: 1987), should therefore be suitable for use in the construction of selected subgrade layer material and in moderate stiffness engineered fill.

The residual material may also be used for the construction of an engineered fill of selected subgrade layer material and in low stiffness of engineered fills where it is encountered as **G9.**

7.3 Chemical Tests

Disturbed samples of the various horizons were taken and subjected to chemical tests in accordance with DIN 50929 requirements. The chemical test results are attached in Appendix C and are summarised in Table 9 and Table 10 below. Several environmental factors influence buried metals. These factors are:

- Electrical conductivity of the soil
- Chemical properties of the soil
- Ability of the soil to support sulphide reducing bacteria.
- Heterogeneity of the soil (long-line currents)
- Differential aeration
- Stray currents in the soil, and
- Bacteria attack

The conductivity of the soil has a profound influence on the rate of corrosion of buried metallic objects. Based on significance of soil resistivity on corrosivity, Duligal (1996) provides the following table for evaluation of the conductivity of soil:

Table 9: Guideline values for interpretation of soil conductivity (Duligal, 1996)

Soil conductivity		
Soil conductivity (mS/m)	**Soil resistivity (Ohm.cm)**	**Corrosively classification**
More than 50	0–2000	Extremely corrosive
25–50	2000–4000	Very corrosive
20–25	4000–5000	Corrosive
10–20	5000–10000	Mildly corrosive
Less than 10	>10000	Not generally corrosive

Disturbed samples of the residual material were taken and subjected to chemical (pH and conductivity) tests. The test results are summarised as follows.

Based on Evans guideline (1977), a soil pH less than 6 indicates serious corrosion potential.

Table 10: Chemical test results summary for the pipeline.

Hole no.	Depth (m)	pH	Conductivity (mS/m)
Residual material			
TP1	1.40 – 1.50	6.30	14.1
TP5	1.10 – 1.50	6.20	14.0

According to the soil conductivity guideline values (Table 9) (Duligal, 1996) and the results in Table 10 the residual materials on this site are mildly corrosive due to their pH being closer to 6 and moderate conductivity. Corrosion of buried metallic elements should be considered in the design.

7.4 Rock Strength

The Point Load strength test is an indirect tensile strength test. Samples of core were fractured using a point load machine equipped with conical platens and a load measuring system. The test provided a Point Load Strength (Is) for three samples, two directions of testing from the same borehole that are close to each other. These were then corrected to a standard 50mm size equivalent, the Point Load Strength Index (Is(50)). Two main types of tests were conducted:

- The diametral test was where the sample is loaded normal to its axis and;
- The axial test was where loading is parallel to the core axis

The point load testing was conducted to obtain an indirect quantitative estimate of the UCS of a rock, and to provide a correlation with the inferred rock strength estimated from field index tests.

A total of 7 set point load tests were conducted 21 diametral and 21 axials. Tests were done in each borehole. A factor of 20 was used for the resulting UCS through the PLI testing. Table 11 below summaries the Is50. The sandstone on site is anisotropy with an average of 40 MPa axial and 25 MPa diametral, as shown in Table 12 below. Figure 13 shows the UCS strength of Sandstone (SS) rock determined through site PLI testing and the detailed PLT tests results are presented in Appendix C.

Table 11: Point Load Test results summary for Is50.

Rock Type	Test Type	Is 50 Average (MPa)	Sample Count	Minimum Is 50	Maximum Is 50
Sandstone	Axial	1.99	21	0.56	2.44
Sandstone	Diametral	1.24	21	0.12	2.34

Table 12: Point Load Test results summary for UCS for the site

Rock Type	Test Type	UCS from PLI AVERAGE (MPa)	Sample Count	Minimum UCS	Maximum UCS
Sandstone	Axial	39.86	21	11.26	59.80
Sandstone	Diametral	24.74	21	2.44	46.83

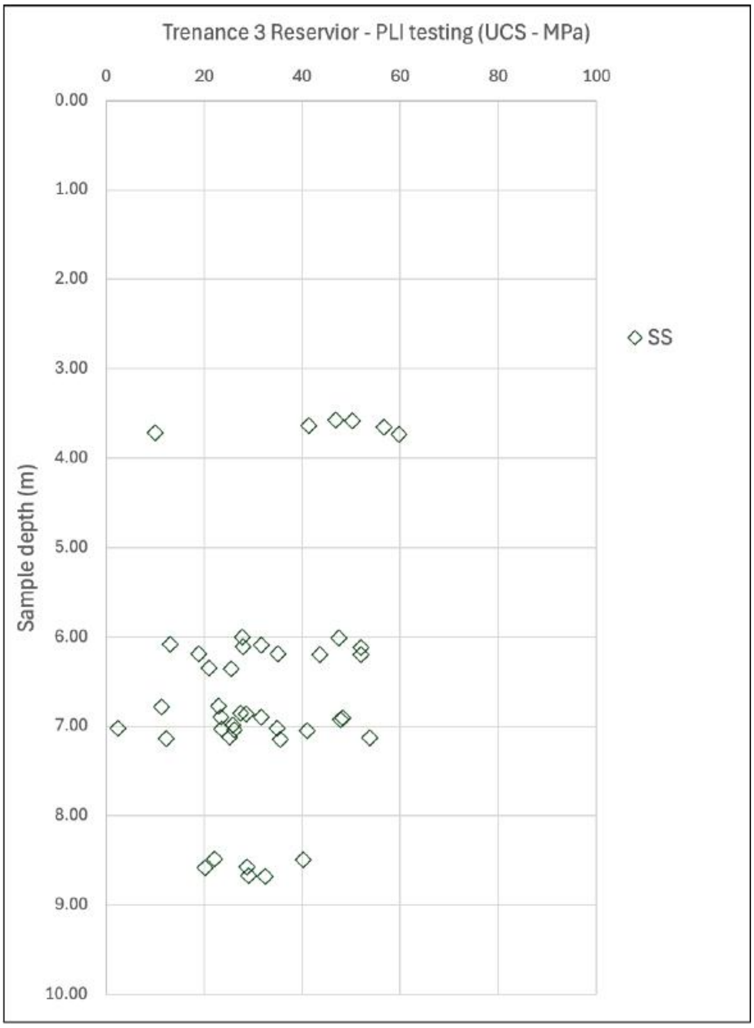

Figure 13: UCS from PLI for all tested Sandstone (SS) samples

8 Geotechnical Considerations

8.1 Collapsible

Soil with a collapsing fabric may be defined as soil that can withstand relatively large, imposed stresses with small settlements at a low in-situ moisture content but will exhibit a decrease in volume and increase in associated settlement with no increase in the applied stress if wetting up occurs, as is aptly described by (Schwartz;1985). Several geotechnical tests are available to determine the collapse potential of a soil material either as a parametric or numerical value. The tests all depend on the availability of an undisturbed sample cut from the soil profile and are relatively expensive. With the transported soils on site being of loose and very loose consistency, it was not possible to extract an undisturbed sample. However, Errerra (1977) researched the properties of the residual and transported soils resulting from siltstone and sandstone tend to be predominantly sandy and defined a grading envelope for collapsing sands. According to his research, it has been found that should the grading curve of a soil material fit into this envelope, the soil can be regarded as being of collapsing nature.

The fill transported and residual materials on site with a consistency that is loose to medium dense which is prone to collapse according to research from Errrera. It is expected that the transported soil material will be collapsible when the moisture conditions change from dry to moist due to rainwater infiltration etc.

8.2 Erodibility of the soil profile

The soil layers encountered along the pipeline route are generally non-cohesive and therefore it is expected to be erodible. Protective measures against erosion must be implemented on the development site.

8.3 Excavatability

The ease at which the soil can be excavated is an important criterion in the selection of a site. The excavation conditions at the site should be categorised as '**soft mechanical excavation**' to average depths of 3.50 m thereafter mechanical means will be required (including excavator) to undertake excavation to the intended depth.

9 Site Considerations

Based on observations made during the geotechnical investigation, the following factors must be taken into consideration:

- At the time of the investigation, the site was fairly levelled on top of the hill.
- The fill layer, transported layer, and residual bedrock material comprise predominantly sandy material, which has a very low potential expansiveness.
- Competent founding material is present at the depth ranging from 2.50m to 4.0 m on the sandstone bedrock.
- The proposed 6 Ml reservoir is a rectangular shape, concrete structure sensitive to total and differential settlements.
- The 0.4Ml elevated tank and pump stations will also be sensitive to settlement.

10 Recommendations

According to the information provided by the client, the proposed development will include associated pipelines, a Pump Station, 0.4Ml Elevated Tanks, and a 6Ml Water Reservoir, some of these structures, especially the Water Reservoir will exact significant loads to the ground.

The results from the excavated test pit reveal that the area is underlain by collapsible and compressible silty sandy material to the depth ranging from 2.50m to 3.50m, then sandstone bedrock to the depth beyond 10.00m.

Based on the findings from the investigation, the following recommendations regarding the construction of Pump Station, Elevated Tanks, the Water Reservoir, and associated connecting pipelines apply:

Foundations for structures should be designed and constructed in accordance with SANS 10400-H or any site-specific specification issued by the structural engineers.

At the time of writing this report, the specific dimensions of the foundation footings were not available. Once these dimensions are provided by the structural engineer, a comprehensive settlement analysis will be determined for the footings that shall be placed on residual material.

It is worth noting that the sandstone bedrock was encountered at an average depth of 3.0m on this site. It would also be favourable to place the footings of the reservoir on this competent bedrock horizon, as the settlement is negligible on sandstone bedrock, typically less than 5 mm.

10.1 Pump Station Founding Recommendations

Option 1: Founding the structures on reinforced deep strips or pad footings.

The positioning of this structure sits close to the position of Borehole 2. The result reveals that the areas are underlain by very dense residual/completely weathered rock (as STP refusal was encountered at 2.0m depth) at an average depth of 2.00m. It is recommended that the proposed structure is founded on the underlying competent material and constructed as stiffened deep strip or spread footings at an average depth of 2.0m below the current platform level, NB the levels may change after the earthworks. The footings should be constructed to the structural engineer's specifications.

Under no circumstances should the foundations be placed in fill unless such fill is engineered for this purpose.

Option 2: Founding the structures on reinforced concrete rafts.

Construction of reinforced concrete rafts incorporated with deeper ground beams on competent founding material. The concrete raft would have to be founded on the sandstone to prevent differential settlements.

NB: It is recommended that an experienced geotechnical engineer inspect and approve all foundation excavations to confirm the depth of founding and bearing pressure.

10.2 Elevated Tanks Founding Recommendations

The positioning of the elevated Tank 1 and Tank 2 sit on the position of Borehole 6 and Borehole 7 respectively, and the result reveals that those areas are underlain by very dense residual/completely weathered rock (as STP refusal on boreholes was encountered at an average depth of 2.0m) at an average depth of 2.00m, it is therefore recommended that both elevated tanks are founded on the pad footings at an average depth of 2.0m below the current platform level, NB the levels may change after the earthworks. An allowable bearing capacity of 150 kPa is deemed appropriate for the residual soil material, based on the geotechnical investigation results. This value is considered suitable for the design of foundations on the residual material. The footings should be constructed to the structural engineer’s specifications.

10.3 Reservoir Founding Recommendations

The following factors will adversely affect the founding of the reservoir:

The proposed 6 Mega litre reservoir is a rectangular, concrete structure sensitive to total and differential settlements.

The stresses anticipated to act on the rectangular reservoir include the following key factors:

Bending Moments: These will manifest in both the horizontal and vertical directions of the reservoir walls. The magnitude and direction of these bending moments will be influenced by the ratio of the wall's length to its height, which will dictate the distribution and orientation of the bending stresses.

Direct Pull: The hydrostatic pressure exerted by the water will induce a direct tensile pull on specific areas of the reservoir walls. It is important to note that, during the course of the site investigation, no groundwater seepage was observed, which may affect the magnitude of such forces.

Bottom Thickness: The thickness of the reservoir base plays a critical role in the overall stress distribution and potential deformation of the structure. A sufficient base thickness is essential to ensure structural stability under the expected loads, particularly in resisting any vertical stresses transmitted from the water above.

Each of these factors must be carefully considered in the design and analysis of the reservoir to ensure its structural integrity under varying operational conditions.

The final design by the structural engineer of this reservoir should consider the bending moments and the other related stresses. The reservoir foundations will induce loads of about 250 kPa to the subsoil.

- Water retaining structures are sensitive to differential ground movements, even if the magnitude of movement is considered tolerable and even minor in terms of structural integrity. Settlement may result in cracking of the floors or damage and/or subsequent leakage at water-stops.
- The floor loads of a reservoir are very high in comparison with typical building floors and because of the size of the footprint of the structure, the influence of the load will affect the soil to a considerable depth, if the foundations were to be placed on ground level.
- The client pointed out that the reservoir will be founded at a depth of about 4.0m below the existing ground level, and the results from BH1 to BH5 reveal that at that depth there is medium hard sandstone.

In view of the above, the following founding measures are recommended:

- Remove the in-situ material to a depth of about 4,0 m below the existing ground level (this will end up in the sandstone).
- A strong excavator can be used to achieve the desired depth, keeping in mind that blasting might be necessary at places.
- Compact the base of the excavation to 90% Mod AASHTO density.
- Backfill two layers (each 150 mm thick) with imported G6 material or better, compacted to 93% Mod AASHTO density. To prevent deterioration of the bedrock, it is recommended that these layers be placed and compacted as soon as possible after completion of the earthworks.
- Backfill to the founding level of the structure with imported G6 material or better, compacted in 150 mm thick layers to 95% Mod AASHTO density.
- The proposed reservoir must be founded on a raft foundation. Based on the geotechnical investigation and subsequent analysis, an allowable bearing capacity of 450 kPa is considered appropriate for the soft bedrock. This value can be utilized for the design of foundations on soft sandstone bedrock, ensuring safety and structural integrity.

To prevent the granular fill from acting as a sump and associated deterioration of the underlying bedrock it is recommended that the site be drained.

10.4 Connecting Pipelines Recommendations

In terms of the SANS1200LB (1983) concerning bedding requirements, buried pipelines require two types of selected material. Those selected materials are termed “Selected Granular Material” and “Selected Fill Material”.

From visual inspection of the materials encountered in the test pits, the following comments and recommendations regarding the suitability and use of in-situ materials can be made:

- Some of the transported materials encountered on site are fine grained silty sand.
- Selected Back Fill Material is defined as “a material with a Plasticity Index (PI) not exceeding 6, free from lumps, vegetation and stones of a diameter exceeding 30mm”.

In general, the “Selected Granular Material” is used as bedding material to support the pipe, while the “Selected Back Fill Material” is used as blanket material over the crown of the pipe. Backfill material is generally placed above the blanket materials, up to ground level.

Some of the in-situ sandy materials (transported layer) along the pipeline routes are considered suitable to be used as Selected Granular Material” or “Selected Back Fill Material”. Should there be inadequate volumes of the transported and residual layer, the materials meeting the SANS1200LB requirements would thus need to be imported to the site.

10.4.1 Selected Fill material

Selected fill material shall be material that has a PI not exceeding 6 and that is free from vegetation and from lumps and stones of diameter exceeding 30 mm.

Based on the results from lab testing, the residual and soft sandstone material found on site is of G9 and G7 quality. Where the material is classified as **G7** according to the TRH 14 guidelines (CSIR: 1987), should be suitable for use in the construction of selected subgrade layer material and in moderate stiffness engineered fill. While the **G9** material may be used for the construction of an engineered fill of selected subgrade layer material and in low stiffness of engineered fills.

Should the sources of Selected Fill Material be not sufficient in terms of volume for the sewer line installation; Selected Fill Material may should be acquired from commercial sources.

10.5 General Recommendations

10.5.1 Trench stability

In general, it is anticipated that vertical sidewall of trench excavations will be stable, however, if any unstable areas are encountered, It is considered that in general trenches not exceeding 1.5m depth can remain open for periods of up to a day without significant collapse provided no significant rainfall and the associated rise in groundwater seepage occurs during this period. Trenches deeper than 1.0m should be battered to a safe angle of 1V:2H or supported laterally. In this respect, it is recommended that no trenches be left open for prolonged periods to prevent sidewall failure.
An experienced geotechnical engineer or an engineering geologist must regularly inspect pipe trenching and sidewall stability.

10.5.2 Excavation Stability

As far as the excavation of service trenches is concerned, trenches less than 1,5 m in depth may be excavated with vertical sidewalls, while deeper temporary excavations and excavations experiencing seepage will require trimming the slope and ensuring that any loose materials in upper soil layers are removed before workers are allowed into the excavations. Slope angles in excavations should not exceed 30 degrees. Shoring is required for excavations extending depths of 3 m below surface level.

10.5.3 Drainage Precautions

The ground surface around the reservoir structure must be sloped away from the structure towards a drainage channel at the toe of the cut slope, at a slope of 5%. The drainage channel must be directed and allowed to drain towards the natural watercourse down the slope east of the reservoir. This will ensure minimal ingress of water to foundations located on improved soil.

All trenches must be properly backfilled to prevent them from acting as French drains. Compaction in these trenches must be performed in 150 mm layers to 90% Mod AASHTO density with a maximum particle diameter of less than 100 mm.

11 Conclusions

The recommendations included in this report relate only to the site that has been investigated. It is recommended that any changes in the proposed structure or future developments, regardless of its proximity to this site, shall be accordingly subjected to geotechnical investigation prior to the construction of any future developments.

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