GEOTECHNICAL INVESTIGATIONS REPORT FOR: SOUTHERN AQUEDUCT WORK PACKAGE 6 DURBAN HEIGHTS TO HIGH WYCOMBE ROAD

FINAL REPORT

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Document Properties

Attribute	Value
Customer Name	Thekwini Metropolitan Municipality
Project Name	Geotechnical Investigation for Southern Aqueduct Work Package 6 Durban Heights to High Wycombe Road
Document Number	EIL_324.3
Document Version	0
Version Date	24 November 2023
Document Status	Final Report
Referenced as	Activities performed report
Authors	Lukhanyo Gqobo
Approval	Basimane Modisane
Access Rights	This document serves to provide information of work undertaken during the geotechnical investigation for the above detailed project. The following role players have access to the document for information and action, as stipulated. • Thekwini Metropolitan Municipality • MMK Group (PTY) LTD

Change History

Version	Revision Date	Revised By	Description
0	-	-	-

Distribution List

Name & Title	Purpose
Thekwini Metropolitan Municipality	Information and Retention
MMK Group (PTY) LTD	Information

Approval

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

Name	Designation	Signature	Date
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1. INTRODUCTION AND TERMS OF REFERENCE

Earthinv Lab was appointed by MMK Group (PTY) LTD to carry out a geotechnical investigation to identify subsurface conditions that will aid in the Construction of Southern Aqueduct Work Package 6 from Durban Heights to High Wycombe Road. The geotechnical data, discussions and recommendations of this report includes a field reconnaissance, data review and field explorations. The investigation comprised a desktop study, site walkover and fieldwork.

The information in this report will inform to the planning, design, and construction precautions to be considered during the implementation of construction works, thus reducing the risk of structural failure and construction damage where adverse conditions may occur.

1.1. Purpose and Scope of Work

This site investigation was carried out to establish the site-specific geotechnical properties of the subsurface material and to provide geotechnical evaluation and recommendations for the detailed design of the proposed project as stated below:

- Review existing geology maps, regional seismic and geological data.
- Assess the groundwater level on site at the time of investigation, if encountered.
- Review available subsurface information in the project vicinity.
- Evaluation of potential construction constraints and development of possible mitigation.
- Advise on appropriate, practicable and cost-effective conceptual planning and design options for the development.
- Excavation conditions.

MMK Group (PTY) LTD are responsible for the design of engineering service for the proposed water trunkline project. This report addresses the required field works needed and provides the parameters to facilitate the design, as well as all applicable South African regulatory requirements, which includes:

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

1.2. Available Information

- Geological Map 2930 Durban 1:250 000
- Locality Plan

2. SITE INFORMATION

2.1. Site Location and Description

The Southern Aqueduct Work Package 6 runs from Durban Heights WTW in Reservoir Hills traversing South West direction crossing M19 Road and running along Aylesbury Road and Amersham Road until High Wycombe Road where it ends. The study area is in the vicinity of Thekwini Metropolitan Municipality in Kwa-Zulu Natal Province. The length of Work Package 6 is approximately 1.6km long and is comprising a combination of a pre-stressed concrete pipe and steel pipe. The proposed study area is flanked by the formal housing throughout the entire length of the proposed area. The longitude and latitude co-ordinates for the start and end of the proposed roads are 29°49'4.00"S & 30°55'11.68"E and 29°48'26.38"S & 30°55'40.56"E respectively. Figure 1 shows the map of the study areas.

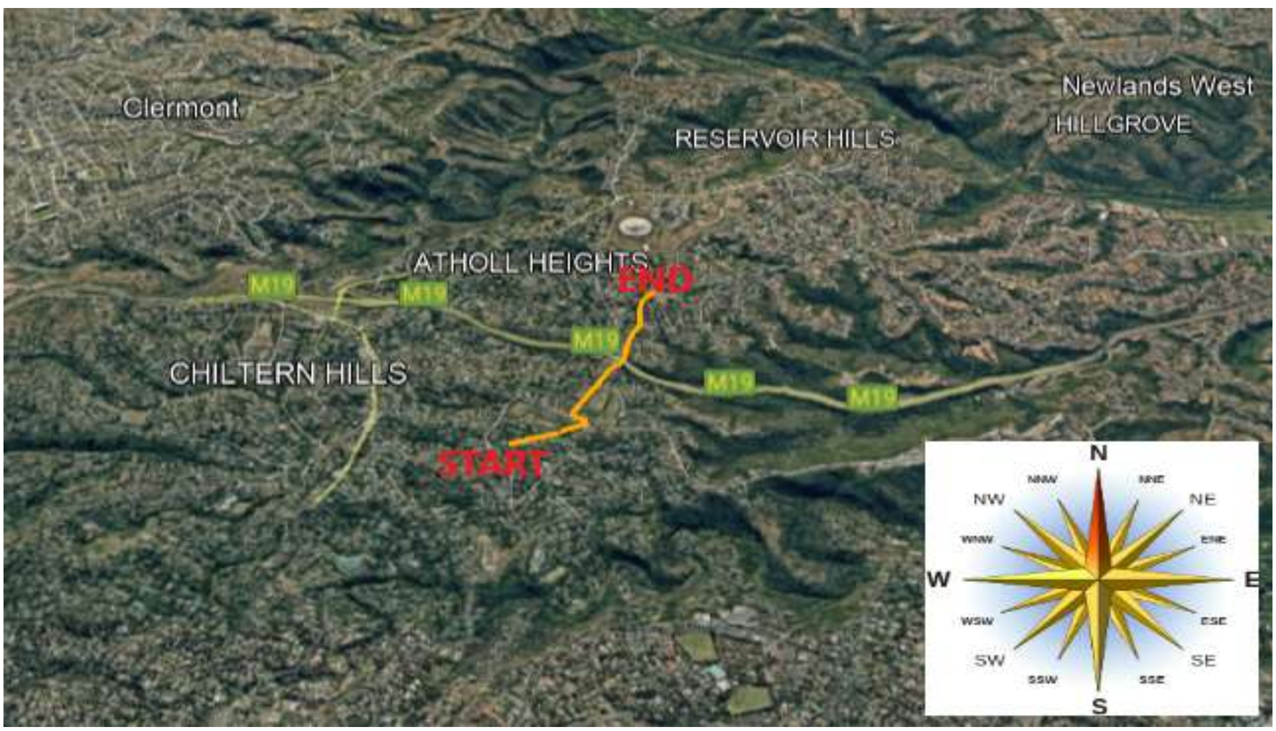

Figure 1 – Site Locality Map

2.2. Topography

The topography of the study area is gently sloping towards south west throughout the entire length of the proposed route with the average slope percentage of 1.1%. The proposed study area is flanked by formal housing. The highest altitude on site is 247m above the sea level at the end of the proposed pipeline route and the lowest is 231m above the sea level at the close to the beginning of the proposed site. See **Annexure H – Topographical Map** for a more detailed topography extracted from a 1:50 000 topographical map 2930DD & 2931CC Durban of South Africa produced by the South Africa Office of the Director-General of Surveys.

2.3. Existing Infrastructure

The proposed sewer outfall line will be located within the servitude of eThekwini Metropolitan Municipality. There were underground services observed during investigations, it is therefore highly recommended that all wayleaves be attained prior to any commencement of works/excavation to avoid any services interruption. *Photos 1-6* show site areas and the surroundings.

	
Photo 1 – Existing Water Chambers on Amersham and High Wycombe Road.	**Photo 2 –** Test Pit Excavation on Amersham Road.
	
Photo 4: Water Seepage on the excavated test pit (TP2) that suggests an existing water pipe leakage.	**Photo 5:** Sidewall Profile on site on Aylesbury Road (TP5)

	
Photo 6 – Existing Formal Housing closely adjacent to the proposed route.	**Photo 8** – Existing overlying electrical cables along the proposed site area.

2.4. Climate

According to Koppen-Geiger, Durban has a hot dry-summer subtropical climate that is mild with moderate seasonality. Summers are dry and hot due to the domination of subtropical high pressure systems while winters experience moderate temperatures and changeable, rainy weather due to the polar front. These climates usually occur on the western sides of continents between the latitudes of 30° and 45°. Vegetation is adapted to the dry summers and is fragrant and oily making it susceptible to fire. The typical Mediterranean climate average monthly temperatures in excess of 22.0 °C in its warmest month and an average in the coldest month between 18 to -3 °C with at least four months above 10 °C. The average temperature is 20.5 degrees Celsius. Total annual Precipitation averages 828 mm.

Climate determines the mode of weathering and rate of weathering. The effect of climate on the weathering process (i.e., soil information) is determined by the climatic N-value defined by Weinert (H.H. Weinert 1980).

Table 1: Weinert N Value

Climate Zone	Arid	Semi-Arid	Sub Humid	Humid
Weinert N-Value	>10	5-10	2-5	<2
Mean Annual Rainfall (mm)	< 250	250-500	500-800	800

The N-value for the study area is <2, which implies a humid climate, and is an indication that chemical decomposition is the dominant mode of weathering which may change the original rock forming minerals into secondary minerals within the zone of weathering.

Climate Data indicates that construction would be better suited between the months of May and August, as there is less rain which can hamper a construction program. The months of June and July are favorable as there is minimal rain during this period.

3. METHOD OF INVESTIGATION AND OBSERVATIONS

3.1. Field Investigations

The site investigation was conducted on the 02nd of May 2023 to 03rd of November 2023, where seven (7) test pits and seven (7) DCPs were conducted. The test pits were excavated within the area of the proposed development. Co-ordinates of the test pits were determined using a hand-held Garmin etrex 10 on the South African grid, while excavations were done using TLB (CAT). Safety procedures as set out in the SAICE Code of Practice (2003, updated 2007), as well as the Occupational Health and Safety Act No. 85 of 1993 were following during excavation and sampling. Soil profiles were described by an engineering geologist based on the standard method proposed by Jennings et al. (1973), table 2 below gives a summary of test pit profiles, while profile logs are attached as **Annexure E**. The map showing test pits positions are enclosed as **Annexure C** while **Annexure D** contains the Site Photos.

Table 2: **Test pits summary information**

Test Pit No	Sample Depth (m)	Total Depth (m)	GPS Coordinates		Remarks
			X	Y	
1	1.3-2.7	2.7	30°55'12.05"E	29°49'4.00"S	Groundwater seepage, no refusal met
2	-	1.3	30°55'18.95"E	29°49'1.48"S	Groundwater seepage, no refusal met
3	-	2.2	30°55'27.21"E	29°48'58.25"S	No Groundwater seepage, no refusal met
4	0.7-2.7	2.7	30°55'28.09"E	29°48'52.24"S	No Groundwater seepage, no refusal met
5	0.3-2.0	2.4	30°55'35.26"E	29°48'39.67"S	Groundwater seepage, no refusal met
6	2.2-2.5	2.5	30°55'37.39"E	29°48'34.46"S	No Groundwater seepage, no refusal met

7	1.2-2.4	2.4	30°55'40.63"E	29°48'26.23"S	Groundwater seepage, no refusal met

4. LABORATORY TESTING

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

- Maximum Dry Density (MDD)
- California Bearing Ratio (CBR)
- Foundation Indicators
- Grading Analysis
- pH and Conductivity

5. GEOLOGY

The Geological Map Series, sheet number 2930 Durban, published at a scale of 1:250 000 by Council of Geoscience from the Geological Survey indicate that the area is underlain by the Red-brown coarse grained arkosic to subarkosic Sandstone, small pebble Conglomerate, surbodinate Siltstone and Mudstone. This lithology was formed during Ordovician- Silurian Age. **The Geological Map Extract is attached on Annexure B for ease of reference.**

6. SEISMIC ACTIVITY

According to Fernandez and Guzman (1979), the area investigated is classified as having a seismic intensity of not more than VI on the modified Mercalli scale (MMS) with a 10% probability of being exceeded atleast once during a 50-year recurrence period. An earthquake with an intensity of VI on the MMS is described as follows:

- All people, in and outdoors feel it.
- Windows, dishes, and glassware are broken.
- Pictures and books fall off walls and shelves.
- Furniture is moved and overturned.
- Weak plaster and poorly constructed mansory structures crack.

The expected peak ground acceleration associated with this magnitude of earthquake are:

- Horizontal acceleration; 56 cm/s^2
- Vertical Acceleration; 18 cm/s^2

The peak ground acceleration indicates a low intensity of seismic activity. Furthermore, the peak horizontal ground acceleration may be 50 to 100 cm/s^2 at least once in a period of 50 years.

With the above in mind, it is expected that serious damage to well-built mansory structures constructed from good quality materials and of good workmanship will not occur. There is therefore no need for special measures to resist natural seismic events.

7. GROUNDWATER

Groundwater seepage was encountered at test pits 2 & 5 during the excavations on site. The encountered groundwater seepage suggested a perched water table on test pit 5 and an existing pipe leakage on test pit 2; however, it is important that the design of the storm water management system allow for the drainage of accumulated surface water from the platforms into the stormwater system or natural drainage lines.

7.1. Test Pits

A total of seven (7) test pits were excavated and profiled by an engineering geologist, with detailed analysis carried out based on the field works.

The site profiles show that the top material is an imported/fill material. This horizon was encountered at all test pits with the dark brown, dark yellow, light brown and dark orange with the pinholed voided and intact structure and the texture of the silty Clay, silty Sand and clayey Sand. The material is loose to medium dense and soft to firm in consistency and is the fill material that has an average thickness of 1.6m. The material that appeared below the fill is the colluvium material. This material is the naturally transported material from the hillwash by slope and water as mode of transport. This is a dark grey horizon that is intact, soft to firm and medium dense, slightly moist to moist with the average thickness of 0.9m and it appeared on test pit 6 & 7.

The horizon that appears at the bottom is the residual material. These are the soil materials that originate from the weathering of the parent bedrock due to weathering resulting from earth movements. Their classification depends on the degree of weathering i.e., highly weathered, highly weathered to intermediately weathered. The residual material on site is an intermediately to highly weathered Sandstone and Shale which are described as silty Clay, clayey Sand and sand Gravel with the structure resembling the mentioned parent rock. They have medium dense and soft consistency with yellowish brown, dark yellow and light grey colour. Residual material on site has an average thickness of 1.1m and it was intersected at most of the test pits. This horizon extends down to the bottom of the test pit until the required depth or the refusal. **Refer to annexure E for the test pits soil profiles.**

The maximum dry density that can be obtained by compaction is dependent upon the soil type and arrangement of various particles making up the soil structure with relative movement of soil particles being affected by the water content. Each soil type has its own optimum moisture content for certain compacted effort and the effect of increasing the compactive effort is to increase the maximum dry density and decrease the optimum moisture content. Table 4 shows the maximum and minimum values obtained for maximum dry density and OMC for samples obtained from seven (7) excavated test pits. The variation in maximum dry density and OMC shown on the table are due to the variation in particle size and number of fines in the soil mass.

Table 3: Laboratory Test Results for Foundation Indicators

Test Position	Depth (m)	Material description	Atterberg limit			PERCENTAGE FINER THAN (mm)			GM	CLASSIFICATION		
			LL	PI	LS	0.075	0.425	2.00		HRB	UNIFIED	TRH14
1	1.3-2.7	Light grey silty Clay	55	18	8.9	53	78	99	0.69	A-5-7	Ci	-
2	0.7-2.7	Dark yellow clayey Sand	37	12	5.9	60	90	95	0.55	A-5-6	CL	G10
5	0.3-2.0	Dark brown silty Sand	37	9	3.4	27	57	85	13.1	A-2-4	CL	G5
6	2.2-2.5	Yellowish brown sandy Gravel	-	NP	0.0	7	32	91	1.71	A-2-4	SM	G9
7	1.2-2.4	Dark yellow clayey Sand	33	5	3.0	35	64	79	1.21	A-4	CL	G10

GM-Grading Modulus

LL-Liquid Limit

PI-Plasticity Index

LS-Linear Shrinkage

CBR test indirectly measures the shearing resistance of a soil under controlled moisture and density conditions. The variation in CBR values are the results of differences in size and shape of the particles, the prevailing moisture within the soil, grading of the particles, how loose the grains are, presence of hard rock fragments and fines. CBR test was conducted on recovered samples from profiled test pits to characterize the strength and bearing capacity of the obtained samples and these were used as a guide complementing CBR obtained from DCP testing. Laboratory results shows low to medium values values for obtained CBRs @ 93%, 95% and 98%. According to TRH14, material sampled from excavated test pits on site have been classified as G5 to less than a G10 class. Parameters obtained indicate that material have poor to good subgrade property according to the HRB classification of soil. **See Annexure G: Laboratory Test Results for ease of reference.**

Table 4: Compaction Properties

Tests	Maximum	Minimum
MDD (kg/m³)	2014	1806
OMC	12.6	6.5
CBR @93%	15	2
CBR @95%	22	2
CBR @98%	38	2

Compaction properties for samples obtained from excavated test pits on site show the MDD, OMC, and CBR values. The MDD for all the samples varies from 1806kg/m3 to 2014kg/m3 and the OMC of 6.5% to 12.6%. The CBR for the sampled material is ranging from low to medium at 93 %, 95 %, and 97% compaction with the values of 2-38. **Laboratory test results are enclosed on this report as Annexure G.**

7.2. Hydrometer Analysis

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

Table 5: Hydrometer Test Results

Test Pit	Depth of Sample (m)	Overall, PI	Activity
1	1.3-2.7	14.38	Medium
4	0.7-2.7	10.37	Low
5	0.3-2.0	5.19	Low
6	2.2-2.5	NP	Low
7	1.2-2.4	3.31	Low

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

7.3. pH & Conductivity

From the journal of Applied Sciences Research, vol. 8, no. 3, pp. 1739-1747, 2012 ("Relationship between Soil Properties and Corrosion of Carbon Steel") by M. N. Norhazilan and Y. Nordin, resistance of an electrolytic solution, as represented by the saturated soil paste or water, is that property of the solution which opposes the flow of an electric current. The resistance is dependent on the amount and nature of dissolved ion content (TMH 1 second addition).

The potential of hydrogen (pH) 5.6-6.8 was measured, which is slightly acidic and generally acceptable corrosive effect. The tested value for conductivity is 0.028-0.037S/m indicating that the materials are generally, not corrosive. The tabulated results in **Table 6**, conforms to the guidelines as specified in **Table 7.**

Table 6: Measured pH & Conductivity

TP	Depth (m)	pH Value	Conductivity (S/m)
2	1.3-2.7	5.6	0.028
3	0.7-2.7	6.2	0.037
6	1.2-2.4	6.8	0.034

Table 7: Conductivity & Corrosiveness of the Soil

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

7.4. Dynamic Cone Penetration (DCP) Tests and Analysis

The 1m dynamic cone penetrometer (DCP) tests were carried out during the field investigations adjacent to each excavated test pits. This method of testing was used to evaluate and analyze the bearing capacity of the in-situ subgrade layers. The dynamic cone penetration (DCP) test is conducted by driving a 20 mm diameter, 60° cone into the ground by an 8 kg hammer. The hammer is lifted by hand and dropped for a distance of 575 mm, and the results are expressed as the penetration rate (PR) in mm per blow. It is important to note that the obtained DCP derived CBR is used as a guide to complement material testing.

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

If DN > 2 mm/blow CBR = 410 x DN-1.27

If DN < 2 mm/blow CBR = (66.66 x DN2) - (330 x DN) + 563.33

DN (the rate of cone penetration)

The results are tabulated on the graph indicating a stable founding material with an average CBR of 37.63 % and a UCS of 383.42 KPa at all test pits.

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

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

The average bearing capacity has been calculated at 412.27 KPa.

Table 8: DCP Average CBR, UCS & Bearing Capacity

DCP	Co-ordinates	Ave CBR %	Ave UCS KPa	Ave Bearing capacity KPa
1	30°55'12.05"E, 29°49'4.00"S	37	517.5	692.6
2	30°55'18.95"E, 29°49'1.48"S	11.8	135	198.3
3	30°55'27.21"E, 29°48'58.25"S	23.2	407.83	338.6
4	30°55'28.09"E, 29°48'52.24"S	62.5	569.8	150.96
5	30°55'35.26"E, 29°48'39.67"S	32.06	275.8	383.83
6	30°55'37.39"E, 29°48'34.46"S	36.08	359	375
7	30°55'40.63"E, 29°48'26.23"S	60.74	419	746.6
		37.63	383.42	412.27

General: In general, conditions of soft to intermediate excavation must be anticipated and the trench sidewalls proved to be quite stable during the field explorations. Furthermore, the DCP probes were conducted on site and confirm compressible to collapsible soils as the top layer.

Refer to Annexure F – DCP Probes and Analysis

8. EXCAVATABILITY

The excavation characteristics of the different soil horizons encountered have been evaluated according to the South African Bureau of Standards; standardized excavation classification for earthworks (SABS-1200D) and earthworks (small works-SABS-1200DA). The soil on site can be classified as soft to intermediate excavation and can be achieved by a back-acting Excavator. It is unlikely to envisage hard excavation requiring any blasting in this project.

Table 8: Excavation Classification

Excavation Class	Description
Soft	Excavation in material that can be efficiently removed by a back-acting excavator of flywheel power approximately 0.1 Kw per millimeter of tined bucket width, without the use of pneumatic tools such as paving breakers.
Intermediate	Excavation in material that requires a back-acting excavator of flywheel power excavating 0.1 kW per millimeter of tined-bucket width or the use of pneumatic tools such as paving breakers.
Hard	Hard rock excavation shall be excavation in material (excluding boulder excavation) that cannot be efficiently removed without blasting or wedging and splitting.
Boulder	Excavation in material containing more than 40% by volume of boulders of size in the range of 0.03-20m$^{3.}$ In a matrix of soft or smaller boulders.

9. STABILITY OF TRENCHES

The test pits excavations were all vertical and there was no evidence of side wall collapse during excavation of the test pits; therefore, any trenches excavated to within the limits 1.4m depths are expected to be stable. However, in cases where water ingress is encountered, or the slopes are left open for an extended period, there could be instability problems. In such case(s), the excavated trenches would have to be battered to stable angles or shored to avoid sidewall collapse.

The Standard Engineering Specifications for Earthworks for Pipe trenches, Part DB, states that “trenches must be excavated in narrow sidewall conditions with vertical sides necessitating the use of adequate shoring methods to prevent erosion and consequent slope instability along any section of the proposed route”. Safety regulations require that any trench greater than 1.2 metres in depth must be appropriately shored.

Where the trenches traverse valley lines or even unclearly defined streams and the subsoils may be saturated, even the cohesive clays may have reduced sidewall stability and additional shoring or trimming back of batters may need to be considered. As a general guideline, side slopes of temporary trench excavations should be restricted to the following:

- Colluvium and residual soils - 1:1 (Vertical: Horizontal)
- Loosely bedded siltstone and shale - 1:1
- Highly to moderately weathered bedrock - 2:1
- Competent tightly jointed/bedded bedrock - 4:1.

10. PIPELINE BEDDING MATERIAL

The typical silty sands materials encountered along the pipe route show clearly that the in-situ materials are fine grained and plastic to be used as pipe bedding in the trenches. SANS 1200LB provides for one class of bedding for flexible pipes. Thin wall steel pipes are usually considered as flexible pipes and therefore, the bedding requirements should be based upon the following requirement:

- Selected Granular Material - Non-cohesive, singularly graded between 0,6mm and 19,0mm, having a compatibility Factor not exceeding 0.4.
- Selected Fill Material - Material with a plasticity index not exceeding 6 and free of vegetation and lumps or stones exceeding 30mm.

11. DESIGNS ANALYSIS AND RECOMMENDATIONS

The test pits were not perfectly backfilled and any structural construction over such areas might lead to some degree of settlement if not properly designed for. It is recommended that such areas be identified and recompacted. Layer works should be designed following an analysis of the expected volume of traffic and water that will be channelled.

11.1. Stormwater Drainage Proposed

Where necessary the excavation of the trenches for subsurface drains shall comply with the requirements specified. The trench shall be backfilled with approved impermeable material preferably obtained from the excavations, in layers not exceeding 300 mm and compacted to 90% of modified AASHTO density, unless otherwise specified by the Engineer.

11.2. Site Clearance and Earthworks

Site Clearance

Normally borrow areas and the portions of the site on which excavations are to be made shall be cleared and grubbed as per the Engineers specification.

Earthworks

Prior to starting any excavations, construction-bed preparations, or fill construction, the contractor shall obtain instructions from the Engineer regarding any stripping of topsoil or any clearing and grubbing that might be required.

It is recommended that all earthworks be carried out in accordance with SANS 1200 (latest version). Excavations shall be backfilled with approved material in horizontal layers not exceeding 150 mm in depth after compaction, to the level of design drawings. Each layer shall meet the specified optimum moisture content for the material and be compacted to a density of not less than 90% of modified AASHTO density.

After backfilling of the trench, the surrounding ground surface must be levelled out to ensure free surface run-off of stormwater and prevent ponding of run-off along or near the trench as this could lead to softening of the backfill or even, in extreme conditions on steep slopes, the inducing of localised slope stability challenges by the excessive ingress of moisture into the backfilled subsoils.

12. CONCLUSIONS

This report contains the findings from the geotechnical investigation carried out along the proposed site with detailed assessment of the engineering properties and classification of the material down to maximum depth of 3m. The comments and recommendations contained within this report are based solely on the exposed sections of the test pits excavated during the geotechnical field work. Cognisance should be taken of the relevant legal and environmental requirements.

It is strongly recommended that inspection and monitoring during construction works is carried out to track and record any deviation in founding conditions as predicted from the original ground investigation and also confirm that the findings in the geotechnical report represents the ground conditions of the proposed site.

It is recommended that all excavations be inspected by a competent person prior placing of bedding and pipe laying as per SABS 1200D specification.

Regular checks on the quality and compaction of the bedding, blanketing and backfill to the trenches should be made. Pressure tests should also be undertaken, in sections, to check for potential leakages.

During geotechnical site investigations there were underground services observed adjacent to the proposed area that could be of hindrance during construction. It is recommended that all wayleaves or service detection be performed prior to any commencement of works/excavation, to avoid services interruption.

The geotechnical investigation carried out and discussed in this report is for a Southern Aqueduct Work Package 6 that will be constructed on the proposed route and site. Should this not be the case, further geotechnical investigations may have to be conducted.

13. REFERENCES

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