Geotechnical Investigation Report

February 2025

Investigation for the Proposed Watermain from Blackburn Reservoir to Phoenix 1 Reservoir Supply Area

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EGE2025-02: Assessment for the upgrade of the Watermain from Blackburn Reservoir to Phoenix 1 Reservoir Supply Area - KZN Province	
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1 Introduction

EThekwini Municipality appointed Elite Geotech and Enviro Construction Services to conduct a geotechnical investigation to upgrade the proposed Watermain from Blackburn Reservoir to Phoenix 1 Reservoir Supply Area in Ward 102, 35 & 48. The proposed pipeline is approximately 6.00km in length.

The investigation was conducted in accordance with the South African Institute of Civil Engineering Code of Practice (SAICE, 2010) and the Strategic Asset Management Pipeline Design Technical Specification—Doc. No: SAM DOP 00001 TS to meet its requirements. The investigation included a desktop study, fieldwork (test pitting, profiling, and sampling), laboratory testing, and reporting.

The fieldwork was conducted from the 9th to the 11th of January 2025 with the following objectives:

- Describe the investigation procedure.
- Provide an overview of the geology of the site.
- Discuss the soil profiles encountered.
- Comments on the groundwater conditions.
- Characterizes the soil properties based on the results of laboratory testing.
- Comment on the excavatability of the subsoil.
- Identify and discuss potential problematic geotechnical considerations (if any).
- Provide geotechnical recommendations regarding the founding of the pipeline; and
- Presents generic geotechnical related construction recommendations.

This report presents the findings and analysis of the data obtained from the field, including soil profiles, in-situ, and laboratory testing.

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 2626 (Soil and Research Institute, 1998).
- Aerial photographs, sourced from Google Earth.
- Locality plans indicating the extents of the investigated section
- 20157 - Report Cornubia excl Phase 1.

3 Site description

3.1 Site Locality

The proposed pipeline will travel between Mount Edgecombe and Phoenix in Wards 102, 35, and 48 of aThekwini Municipality, KwaZulu-Natal Province. It is accessible via the Phoenix Highway to the southeast and the M41 to the northeast part of the site. The proposed pipeline is approximately 6km long and crosses two main highways, R102 and M41, and streams. The route of the pipeline is shown in Figure 1 below.

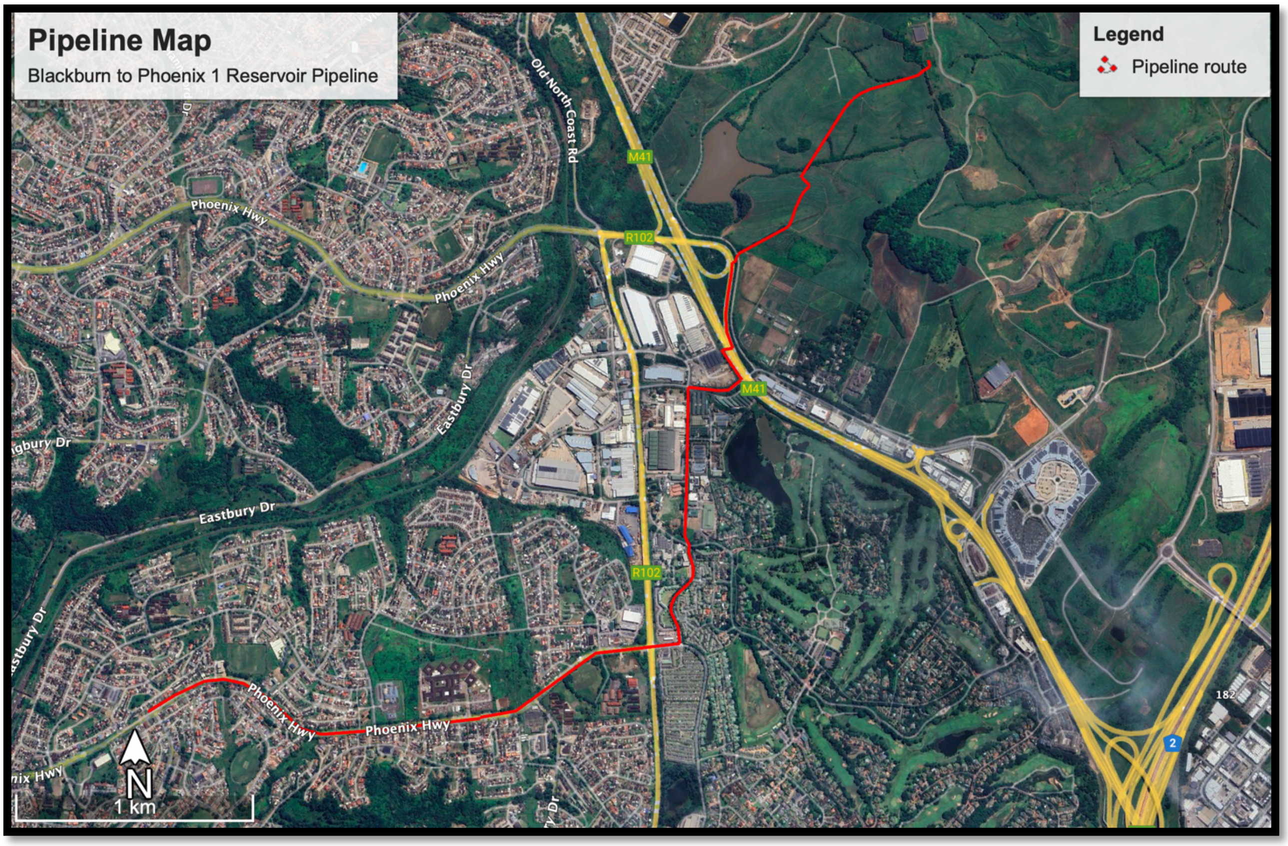

Figure 1: Showing the investigated area.

3.2 Topography and vegetation

The proposed pipeline route crosses a diverse topographical landscape, which includes moderate and gently sloping areas, and terrains of relatively levelled ground. During the site investigation, the route of the pipeline was traversing adjacent to the road, predominantly covered with grasses, shrubs, and scattered trees. It was also cutting through the streams and roads. A representation of the typical topography highlighting the vegetation is illustrated in **Error! Reference source not found.** and 3 below. Additionally, Figure 4 provides an overview of the site's elevation profile and overall topographical features.

Figure 2: Showing the vegetation cover, stream, and the road within the pipe route.

Figure 3: Showing the vegetation cover within the pipe route.

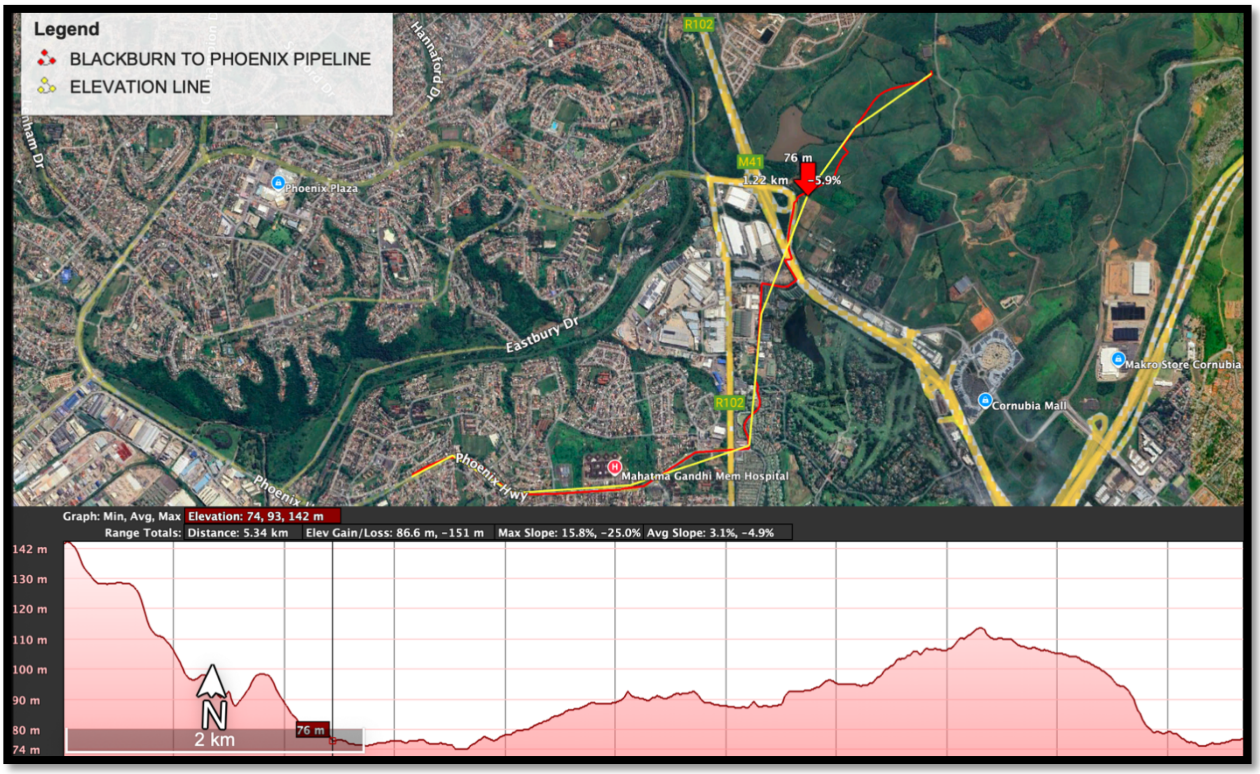

Figure 4: Showing the topography of the investigated site from northeast to southwest.

3.3 Climate

The site area lies approximately 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.

3.1 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 5 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.

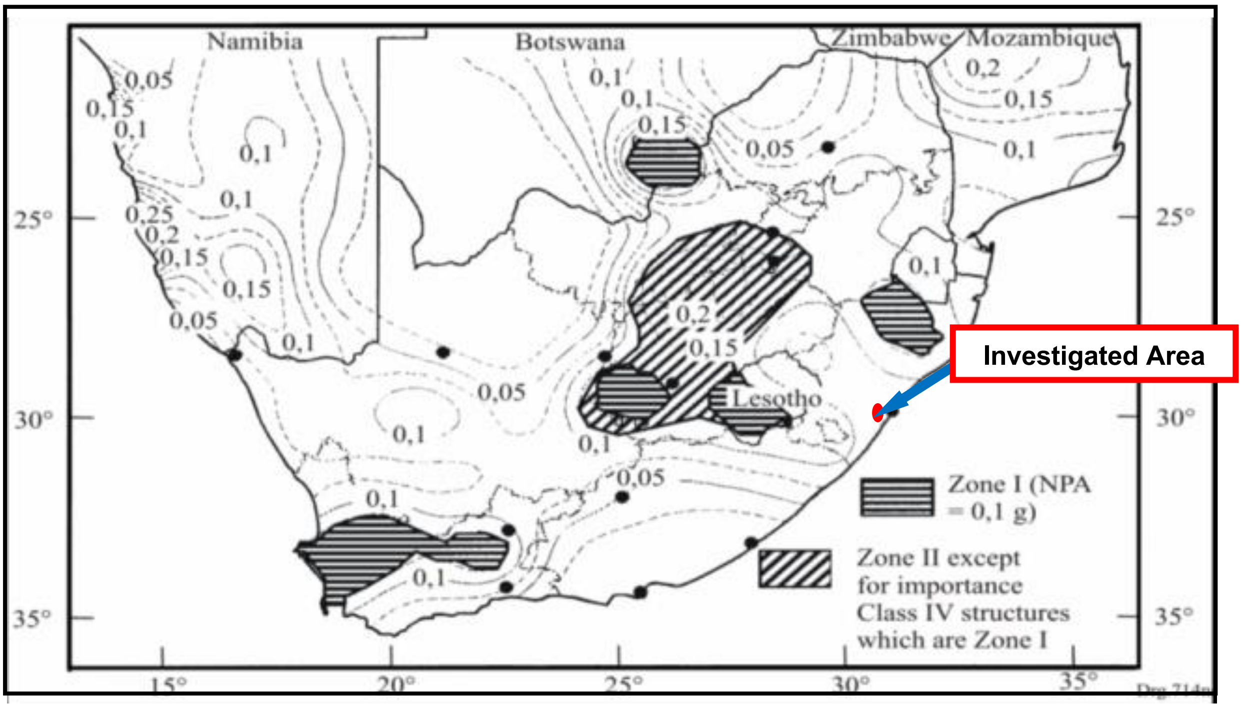

Figure 5: Locality of the site on the seismic hazard map of South Africa.

4 Geology

According to the published 1:250 000 geological map of Durban Sheet 2930 (Council for Geoscience, 1986), the site is underlain by the Quaternary sands of Berea reds formation (**Qb**), with the lithology consisting of red sands, subordinate white, yellow and purple sand, basal conglomerate. Some portions of the site are underlain by Pietermaritzburg Formation (**Pp**), of the Ecca Group, within the Karoo Sequence, with the lithology consisting of dark grey shale, siltstone, subordinate sandstone. Dolerite sills and dykes **(Jd)** are present in the vicinity of the site area as shown in Figure 6 below.

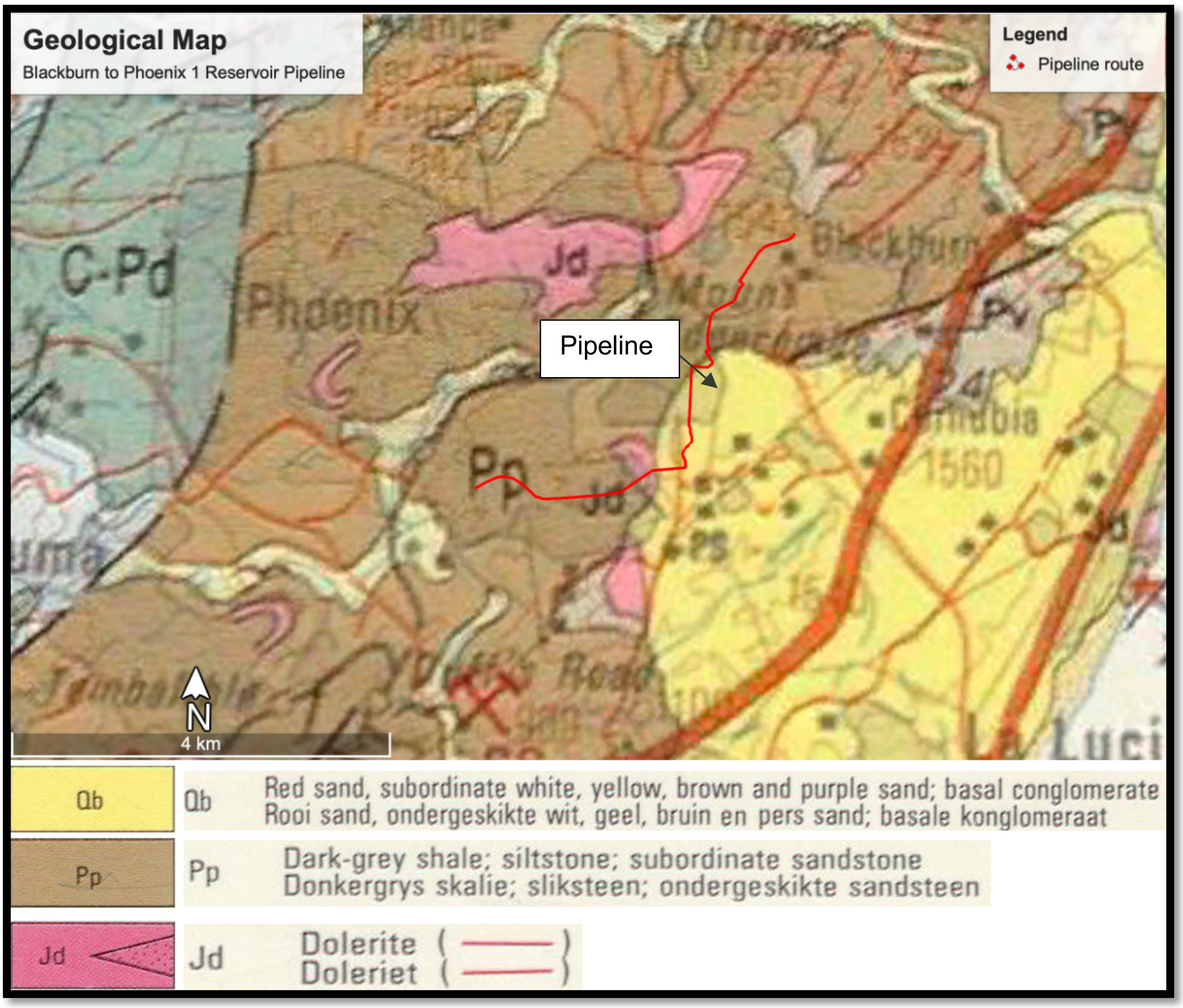

Figure 6: The geological map of the study area is shown (Geological Survey, printed by the Government Printer, Pretoria, 1986).

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.

5.1 Desktop study

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

5.2 Fieldwork

The fieldwork comprised of the following:

- Walk-over survey.
- Excavation and profiling of test pits.
- Collection of representative soil samples for laboratory testing; and
- In-situ testing.

5.2.1 Walk over survey

After the desktop study, a site walkover was undertaken at the proposed pipeline route, to assess the current topographical and geological conditions from the surface without any intrusive work.

5.2.2 Test Pitting

The fieldwork involved the excavation and profiling of twenty-nine (29) test pits, which were approximately 250 meters apart. In regions featuring critical geological features, such as streams, and road crossings, the spacing of the test pits was reduced to facilitate a more focused analysis.

The test pits were hand-excavated and augured in some areas where necessary, aiming to reach a depth of 1.50 meters below the existing ground level, or until refusal on hard material, or until the sidewall stability of a test pit was considered unsafe. Test pit locations were accurately marked using a hand-held GPS, aligned with the UTM grid and WGS84 datum to ensure precision in spatial referencing.

To adhere to established safety protocols, a two-person team conducted the test pitting in compliance with the Site Investigation Code of Practice (SAICE, 2010). A skilled team of engineering geologists and geotechnical engineers executed the test pit layout and profiling, ensuring alignment with South African Standards (SANS 633:2012).

A summary of the test pit locations and excavated depth is provided in Table 1 below, with detailed soil profiles presented in Appendix B. Figure 7 below further shows the location of the test pit along the pipeline route.

Table 1: Pipeline test pits summary

Test Pit No.	Coordinates (WGS84)		Final Depth (m)	Remarks
	Longitude	(Excavation hardness)		
TP1	31°3'2.12"E	29°41'35.25"S	1.50	No refusal
TP2	31°2'54.85"E	29°41'38.53"S	1.60	No refusal
TP3	31°2'47.30"E	29°41'42.85"S	0.85	Refusal on Shale bedrock
TP4	31°2'42.14"E	29°41'49.76"S	1.00	Refusal on Shale bedrock
TP5	31°2'40.41"E	29°41'52.36"S	1.30	Refusal on dolerite
TP6	31°2'38.26"E	29°41'59.30"S	1.50	No refusal
TP7	31°2'29.85"E	29°42'3.42"S	1.50	No refusal
TP8	31°2'28.73"E	29°42'4.71"S	1.50	No refusal
TP9	31°2'27.65"E	29°42'12.84"S	1.50	No refusal
TP10	31°2'28.67"E	29°42'17.08"S	1.50	No refusal
TP11	31°2'26.87"E	29°42'17.39"S	1.50	No refusal
TP12	31°2'30.06"E	29°42'21.61"S	1.50	No refusal
TP13	31°2'21.49"E	29°42'22.46"S	1.50	No refusal
TP14	31°2'21.17"E	29°42'30.61"S	1.50	No refusal
TP15	31°2'20.90"E	29°42'38.72"S	1.50	No refusal
TP16	31°2'21.42"E	29°42'46.41"S	1.50	No refusal
TP17	31°2'18.33"E	29°42'53.31"S	1.50	No refusal
TP18	31°2'19.68"E	29°42'58.31"S	1.50	No refusal
TP19	31°2'10.50"E	29°42'59.15"S	1.50	No refusal
TP20	31°2'2.49"E	29°43'1.93"S	1.50	No refusal
TP21	31°2'0.06"E	29°43'14.46"S	1.50	No refusal
TP22	31°1'55.13"E	29°43'6.40"S	1.50	No refusal
TP23	31°1'45.87"E	29°43'8.41"S	1.50	No refusal
TP24	31°1'36.53"E	29°43'9.24"S	1.50	No refusal
TP25	31°1'27.27"E	29°43'9.90"S	1.50	No refusal
TP26	31°1'18.45"E	29°43'8.04"S	1.50	No refusal
TP27	31°1'11.33"E	29°43'3.36"S	1.50	No refusal
TP28	31°1'2.30"E	29°43'3.72"S	1.50	No refusal
TP29	31°0'56.05"E	29°43'6.81"S	1.60	No refusal

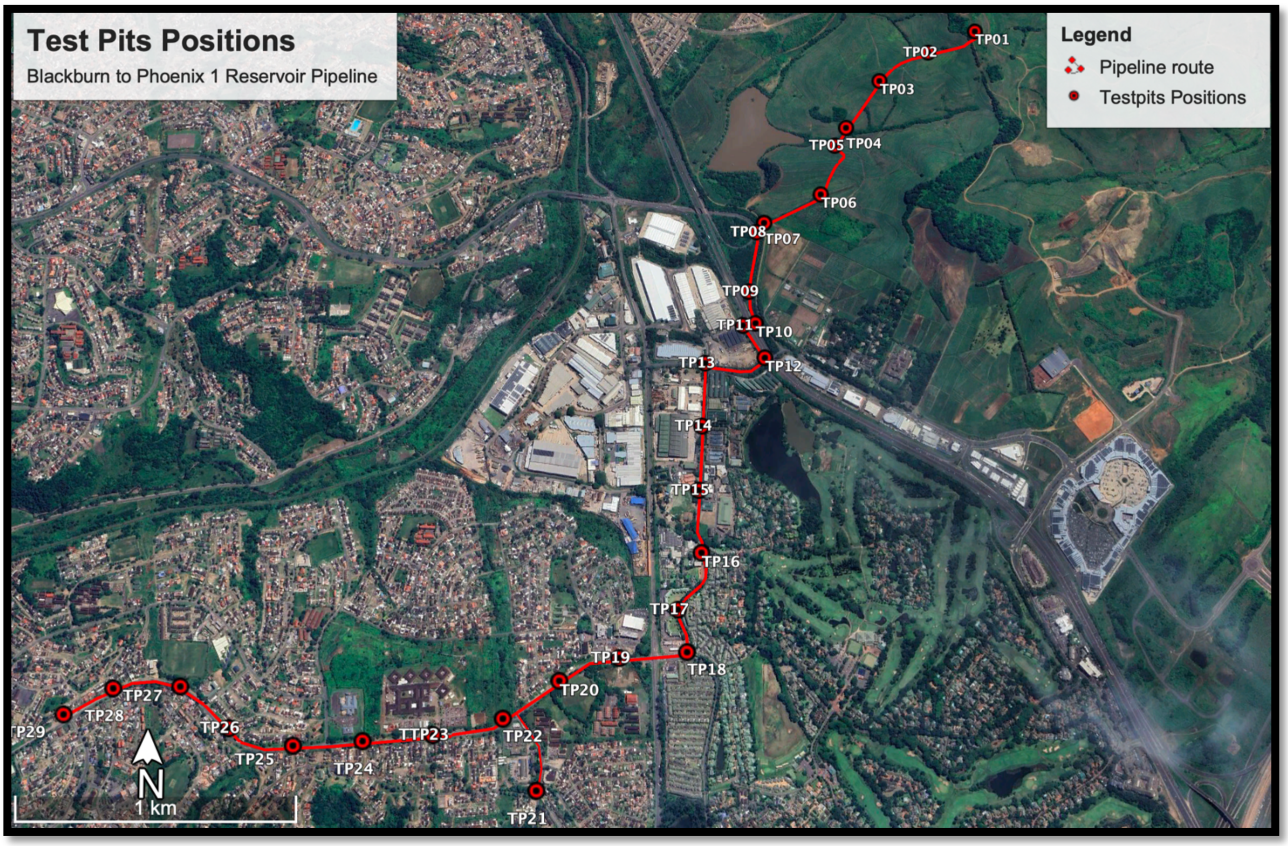

Figure 7: Showing the location of the test pit along the pipeline route.

5.2.3 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.

5.2.4 In-situ testing

The in-situ field testing was conducted using Dynamic Probe Light (DPL) tests. The DPLs were performed adjacent to the test pit along the pipeline to assess the consistency of the in-situ material. The DPL results are included in Appendix C of the report. Figure 4 below illustrates a typical DPL testing conducted next to the excavated test pit on site.

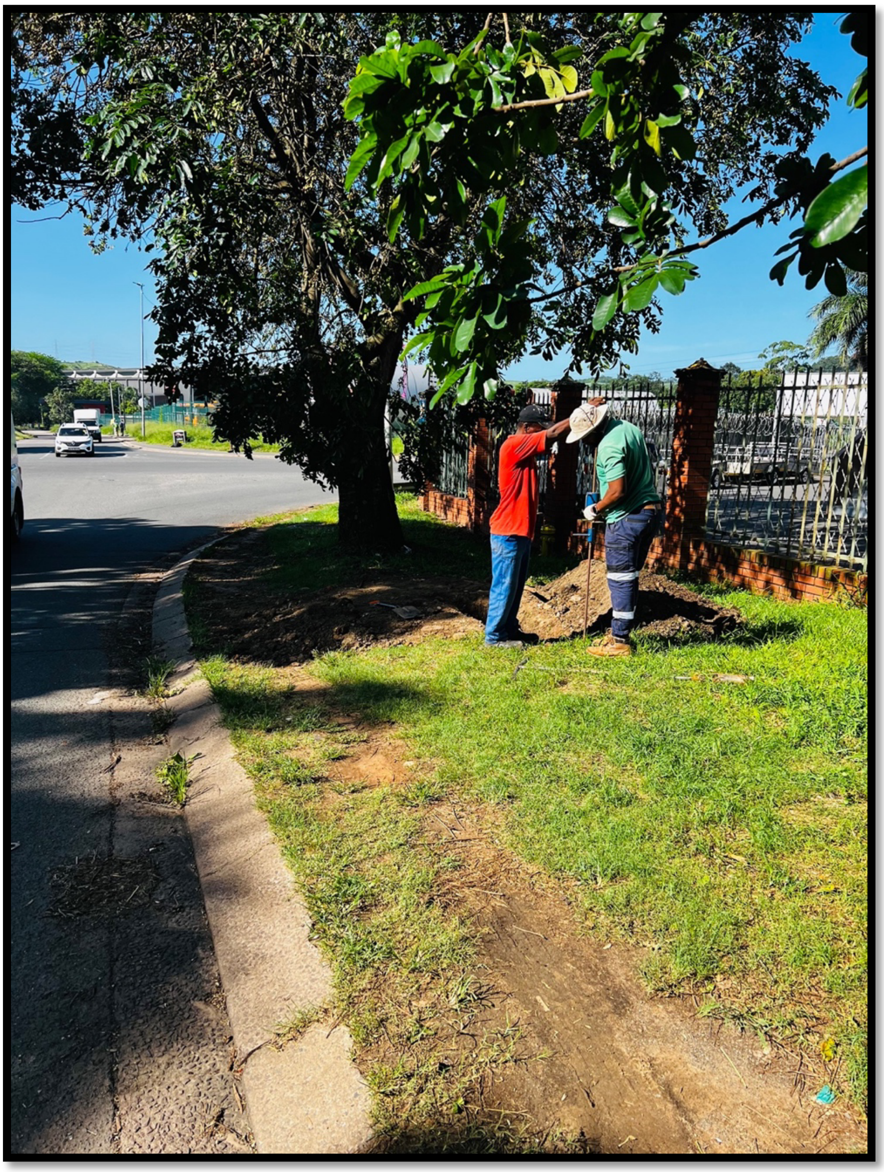

Figure 8: Showing the DCP testing undertaken at the site.

5.3 Laboratory testing

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

- Foundation Indicator tests comprising sieve and hydrometer grading analyses and Atterberg Limits.
- Determination of compaction characteristics (comprising Mods, i.e., maximum dry densities (MDD) and optimum moisture contents (OMC), and CBRs), and
- Determination of pH and conductivity testing.

6 Field Investigation Results

The area is heavily saturated with a variety of underground utilities, including water pipelines, electrical cables, and fibre optic lines, particularly between TP7 and TP29. This significantly complicates excavation activities, requiring careful attention due to the sensitivity of these existing services.

A comprehensive description of the soil profiles encountered during the testing process is provided in Appendix B. A summary of the key soil profiles observed from the excavated test pits on-site is outlined below. The layers encountered are as follows:

- Fill layer.
- Transported layer.
- Residual dolerite layer.
- Residual Mudrock layer.
- Mudrock bedrock, and
- Dolerite bedrock.

6.1 Fill layer

The fill material was encountered as the uppermost layer from test pit 12 to test pit 29 excavated along the pipeline route, except for test pit 21. This is due to the concentration of services, such as cables and water pipes, in this area. This layer was described as slightly moist to moist, light greyish brown, gravely silty sand with traces of cobbles and roots. The layer has a medium dense average consistency.

6.2 Transported layer

The transported layer was encountered as the uppermost layer in test pits 01 to test pit 11 along the pipeline route. It was encountered as slightly moist, dark grey, brown, loose, intact, slightly clayey sand with roots. The layer had a loose to medium dense consistency.

6.3 Residual dolerite layer

The residual dolerite layer was encountered overlying dolerite bedrock in the excavated test pit. It intercepted in test pits 5 and 6 and described slightly moist, dark greyish brown mottled reddish brown, stiff, gravelly sandy clay with very soft rock fragments, gravel and cobbles.

6.4 Residual Mudrock

The residual mudrock intercepted on site was described as moist to wet, light greyish brown, clayey silty sand with gravel cobbles and traces of shale fragments. This layer occurred as medium dense in consistency at the site. It was encountered in most test pits excavated at the site.

6.5 Mudrock bedrock

The mudrock bedrock was encountered at the shallow depth in the base of test pit 3 and test pit 4 and underlies the transported layer at the site. It occurs as light greyish brown, mottled orangey brown, thinly bedded close-jointed, soft rock. This layer occurred as moderately weathered.

6.6 Dolerite bedrock

The dolerite bedrock was encountered at the base of test pit 05 and test pit 06 and underlies the transported and residual dolerite layer at the site. It occurs as dark greyish brown, highly fractured, soft rock. This layer occurred as highly weathered.

NB: It was difficult to excavate between TP7 and TP29 at the site due to the underground services, including water pipelines, electrical cables, and fibre optic lines, Figure 9 below shows the exposed cables inside the test pit and underground water seepage. Test pit 6 showing the stable test pit walls excavated at the site. The summary of test pit lithology is given in Table 2 bellow.

Figure 9: Showing the exposed cables inside the test pit and underground water seepage conservatively.

Figure 10: showing the stable test pit walls excavated at the site.

Table 2: Summary of test pit lithology

Test Pit No	Fill layer (m)	Transported layer (m)	Residual Shale/ Mudrock (m)	Residual Dolerite (m)	Shale Bedrock (m)	Dolerite (m)
TP1		0 – 0.90	0.90 – 1.50			
TP2		0 – 1.00	1.00 – 1.50			
TP3		0 – 0.50			0.50 – 0.85	
TP4		0 – 0.20			0.20 – 1.00	
TP5		0 – 0.50		0.50 – 1.10		1.10 – 1.30
TP6		0 – 0.40		0.40 – 1.10		1.10 – 1.50
TP7		0 – 1.10	1.10 – 1.50			
TP8		0 – 0.90	0.90 – 1.50			
TP9		0 – 1.00	1.00 – 1.50			
TP10		0 – 0.90	0.90 – 1.50			
TP11		0 – 1.50	1.00 – 1.50			
TP12	0 – 0.50	0.50 – 1.00	1.00 – 1.50			
TP13	0 – 0.90		0.90 – 1.50			
TP14	0 – 1.00		1.00 – 1.50			
TP15	0 – 0.90		0.90 – 1.50			

Test Pit No	Fill layer (m)	Transported layer (m)	Residual Shale/ Mudrock (m)	Residual Dolerite (m)	Shale Bedrock (m)	Dolerite (m)
TP16	0 – 1.00		1.00 – 1.50			
TP17	0 – 0.50	0.50 – 0.90	0.90 – 1.50			
TP18	0 – 0.40	0.40 – 1.00	1.00 – 1.50			
TP19	0 – 0.65		0.65 – 1.50			
TP20	0 – 0.60		0.60 – 1.50			
TP21		0 – 1.50				
TP22	0 – 0.75		0.75 – 1.50			
TP23	0 – 0.90		0.90 – 1.50			
TP24	0 – 0.90		0.90 – 1.50			
TP25	0 – 1.00		1.00 – 1.50			
TP26	0 – 0.90		0.90 – 1.50			
TP27	0 – 0.95		0.95 – 1.50			
TP28	0 – 0.80		0.80 – 1.50			
TP29	0 – 1.00		1.00 – 1.50			

7 Groundwater conditions

Groundwater seepage was intercepted in some test pits excavated at the site, namely TP4, TP5, TP7, and TP29. There are also stream crossings between test pits 7 and 8, test pits 4 to 5, and between test pit 6 and 7, the pipeline travels adjacent to the stream. Therefore, water problems are anticipated at places along the pipeline

8 Laboratory Test Results

8.1 Foundation Indicators

Representative samples of selected horizons were collected for laboratory testing and submitted for foundation indicator tests. The detailed test results are attached in Appendix C and summarised in Table 3,

Table 4 and Table 6 below.

Table 3: Summary of section foundation indicator tests results

Hole no.	Depth (m)	Soil composition				GM	Atterberg limits			Activity	Unified soil classification
		Clay (%)	Silt (%)	Sand (%)	Gravel & Cobble (%)		LL (%)	WPI (%)	LS (%)		
Transported layer											
TP01	0.0 – 090	12.4	36.9	24.7	26.0	1.10	23	10	5	LOW	CL
TP10	0.0 – 0.45	10.8	32.0	19.2	38.0	1.30	29	12	6.5	LOW	SC
TP18	0.40 – 1.00	15.1	15.7	25.2	44.0	1.60	32	13	6.5	LOW	SC
Residual Shale											
TP21	0.40 – 1.50	2.1	14.9	37.0	46.0	1.90	33	14	7.5	LOW	SC
TP29	1.0 – 1.60	2.6	20.9	32.5	44.0	1.60	32	13	7.0	LOW	SC

Where:
GM = Grading modulus
LL = Liquid Limit
WPI = 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
SC = Clayey sand
CL = Lean clays

Table 3 above indicates that:

The **transported layer** at the site generally consists of lean clays (**CL**) and clayey sand (**SC**). The layers have a high to very high (1.0-1.60) grading moduli. The fine fractions of this material also exhibit a low to moderate liquid limit as well as a very low to moderate linear shrinkage. The weighted plasticity index (WPI) of the soil is low to moderate. The material has a low to medium potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **residual layer** at the site generally consists of the clayey sand (**SC**). The layers have a very high (1.6-1.90) grading moduli. The fine fractions of this material also exhibit a moderate liquid limit as well as a moderate linear shrinkage. The weighted plasticity index (WPI) of the soil is moderate. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

8.2 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 4 below:

Table 4: Summary of section compaction test results

Hole no.	Depth (m)	OMC (%)	MDD (kg/m^3)	Swell (%)	CBR at various densities				TRH 14 Class
					90 %	93 %	95 %	98 %	
Residual Shale									
TP29	0.2–0.45	10.4	2097	1.05	3.0	5.0	6	8	G9
Residual Dolerite									
TP05	0.50–1.10	11.9	1936	1.00	2.6	4.0	4.9	6	G10

Where:

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

The residual shale and material underlying the site has a moderate to high (1936 – 2097kg/m^3) maximum dry density and moderate (10.4 – 11.9%) optimum moisture content value. The swell is moderate to high (1.00 - 1.05%) and the tests yielded very low to moderate 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 and G10**).

The material that is **G9 and G10** according to the TRH 14 guidelines (CSIR: 1987), should therefore be suitable for use in the construction of selected subgrade layer material and in low stiffness engineered fill.

8.3 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 5 and Table 6 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 5: 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 transported and 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 6: Chemical test results summary for the pipeline.

Hole no.	Depth (m)	pH	Conductivity (mS/m)
Transported and residual material			
TP1	0.0 – 0.90	8.10	136
TP10	0.0–0.45	8.00	135
TP21	1.00–1.60	8.20	137

According to the soil conductivity guideline values (Table 5) (Duligal, 1996) and the results in Table 6, the transported and residual materials on this site are aggressive due to their pH being >6 and conductivity. The high conductivity indicates that Corrosion of buried metallic elements can be expected in places.

9 Geotechnical Considerations

9.1 Shallow seepage/groundwater level

Groundwater seepage and the water table were intercepted at TP4, TP5, TP7, and TP29. Additionally, there are stream crossings between test pits 7 and test pits 8, 4 and 5, and between test pits 6 and 7, where the pipeline runs adjacent to the stream. Consequently, water issues are expected at various locations along the pipeline, particularly during the rainy season. Figure 11 below shows the water in a stream intercepted along the pipeline as well as the groundwater in test pit 29.

Figure 11: Showing the water in a stream intercepted along the pipeline as well as the groundwater seepage in a test pit.

9.2 Collapsible / Compressible soil profile

The transported and residual material underlying the site consists of loose, slightly moist, granular, and cohesive soils. It is expected that the materials will be compressible and collapsible when the moisture conditions and loads change due to water infiltration and load applied.

Problems related to compressibility and collapsibility are expected at the site due to the nature of the granular and cohesiveness content encountered in the transported and residual materials. It is expected that these materials will be compressible and collapsible when the moisture and load conditions change.

9.3 Erodibility of the soil profile

The site generally features gentle slopes, with areas of flat terrain interspersed with cohesive and non-cohesive soil layers along the pipeline route. No signs of water scouring have been observed along the pipe route. However, it is crucial to implement effective erosion control measures, particularly during the construction phase, to protect the integrity of the terrain and prevent soil erosion. These protective measures should be strategically planned and executed to minimize erosion risk, ensuring long-term stability and environmental compliance throughout the project lifecycle.

9.4 Corrosivity

The chemical test results indicated that the transported and residual layers may be aggressive and corrosive. Cathodic designs must take this into consideration to ensure the pipe is not corroded by these materials.

9.5 Excavatability

9.5.1 Geotechnical Excavation Zoning for Pipeline Route

The selection of the pipeline route necessitates understanding of soil excavatability, as the ease of excavation plays an important role in both project cost and schedule. To understand the soil's excavability characteristics, a detailed geotechnical investigation was conducted using Hand Excavation and visual observations from the test pits.

The findings revealed that some portion of the site is underlain by bedrock formations, including shale and dolerite. Some portion of the route features deeper transported and residual materials. The strata encountered during the investigation suggest that a minor portion of the site requires hard mechanical excavation due to the presence of bedrock. Therefore, the depth of hard excavation zones has been categorized along the pipeline route as follows:

The excavatability of material can be grouped into the following categories, according to SANS 1200D, namely Zone A, B, and C, as described in Table 7 below.

Table 7: Excavatability zonation with descriptions

Excavatability Zonation and Description		
Excavatability Zone	**Depth Ranges (m)**	**Description**
A	>1.50m	This zone comprises of deeper transported and residual material with an average consistency of medium dense and can be hand excavatable to the depth greater than 1.50m.
B	1.0 – 1.50	This zone comprises of transported and residual material underlain by bedrock (shale & dolerite). Hand excavation refusal was encountered at the depth ranging from 1.0 - 1.50m.
C	0 – 1.00	This zone is underlain by sallow bedrock (shale, and dolerite). Hand excavation refusal was encountered at the depth ranging from 0 - 1.0m.

9.5.1.1 **Zone A: Depth > 1.50m**

This zone is characterized by deeper transported and residual materials of medium dense consistency. These materials can be efficiently hand excavated to depths exceeding 1.50 meters. The excavability in this zone is generally favorable.

9.5.1.2 **Zone B: Depth 1.0m – 1.50m**

This zone comprises transported and residual materials overlying bedrock formations, including shale, and dolerite. The hand excavation reached refusal at depths ranging from 1.0 to 1.30 meters, marking it challenging excavation conditions.

9.5.1.3 **Zone C: Depth 0 – 1.00m**

This zone is underlain by shallow bedrock, including shale, and dolerite. Refusal was recorded at depths ranging from 0 to 1.00 meters, signifying the presence of rock layers close to the surface. Excavation within this zone presents the greatest challenge, requiring the use of more advanced excavation machinery or blasting, which may further impact project costs and timelines.

9.5.2 Implications for Pipeline Construction

The excavation characteristics across the three zones should guide both the construction methodology and scheduling. For **Zone A**, the excavation can proceed relatively smoothly with standard equipment, while **Zones B and C** will likely require more specialized techniques, such as rock excavation, blasting, or the use of heavy-duty machinery, all of which may influence the project's timeline and cost structure. This must be factored into both planning and budgeting to mitigate any unforeseen delays and cost overruns.

Table 8: Summary of Excavability Characteristics by Zone

Test Pit No	Excavatability Zonation	Depth Range (m)	Comments
TP1		>1.50	No refusal
TP2		>1.50	No refusal
TP3		0 – 1.00	Refusal on Shale bedrock
TP4		0 – 1.00	Refusal on Shale bedrock
TP5		0 – 1.30	Refusal on dolerite
TP6		0 – 1.50	Refusal on dolerite
TP7		>1.50	No refusal
TP8		>1.50	No refusal
TP9		>1.50	No refusal
TP10		>1.50	No refusal
TP11		>1.50	No refusal
TP12		>1.50	No refusal
TP13		>1.50	No refusal
TP14		>1.50	No refusal
TP15		>1.50	No refusal
TP16		>1.50	No refusal
TP17		>1.50	No refusal
TP18		>1.50	No refusal
TP19		>1.50	No refusal
TP20		>1.50	No refusal
TP21		>1.50	No refusal
TP22		>1.50	No refusal

TP23		>1.50	No refusal
TP24		>1.50	No refusal
TP25		>1.50	No refusal
TP26		>1.50	No refusal
TP27		>1.50	No refusal
TP28		>1.50	No refusal
TP29		>1.50	No refusal

10 Recommendations

10.1 Trench stability

In general, it is expected that the vertical sidewalls of trench excavations will remain stable under typical conditions. However, should any areas of instability arise during excavation, it is important to address these promptly to maintain safety and structural integrity. For trenches with a depth not exceeding 1.0 m, they can generally remain open for periods up to 24 hours without significant risk of collapse, assuming there are no significant rainfall events or associated increases in groundwater seepage during this time frame.

For trenches exceeding a depth of 1.0 m, it is imperative that the sidewalls be properly supported or battered to a safe angle. A typical safety slope of **1V:2H** (vertical to horizontal) should be maintained to ensure the stability of the trench. This will minimize the risk of sidewall failure and prevent potential safety hazards or delays in the construction process.

It is strongly recommended that no trench be left open for extended periods, particularly for deeper excavations, to reduce the risk of sidewall instability.

To ensure the continued safety of trenching operations, an experienced geotechnical engineer or engineering geologist must conduct regular inspections of the trenching activities, assessing the stability of the sidewalls and identifying any potential hazards early. Regular monitoring and immediate corrective actions will contribute to maintaining a safe working environment and the overall success of the project.

10.2 Reuse of materials

The material that will be utilized on this project is selected fill and imported bedding material/ padding material. Comment on the existing material's suitability for potential applications is, provided below.

10.2.1 Bedding Material

In the context of pipeline construction, the selection of appropriate bedding material is paramount to ensuring both the immediate stability and long-term durability of the pipeline. Bedding material plays an integral role in supporting the pipe and protecting it from external loads and environmental conditions, making it a crucial component in pipeline design. The geotechnical requirements for bedding material are clearly outlined in **SANS 1200 LB (1983),** which specifies the use of two types of selected materials: **Selected Granular Material** and **Selected Fill Material**.

According to the SANS 1200 LB (1983) specifications for buried pipelines, Selected Granular Material is typically utilized as bedding material to provide direct support to the pipeline, while Selected Backfill Material serves as the blanket material placed over the crown of the pipe. Backfill material is usually placed above the blanket material and extends up to ground level.

Upon conducting a visual inspection of the materials encountered in the inspection pits along the pipeline route, several observations and recommendations regarding the suitability of in-situ materials were made:

- Most of the material present on-site consists of cohesive soils, with occurrences of shallow bedrock in certain areas, accompanied by minimal silty sand.

- As per the **SANS 1200 LB (1983)** guidelines, **Selected Backfill Material** is defined as material with a Plasticity Index (PI) not exceeding 6, free from lumps, vegetation, and stones larger than 30mm in diameter.

Based on the results from the site investigation, it was determined that the in-situ materials encountered along the pipeline route are generally unsuitable for use as either Selected Granular Material or Selected Backfill Material. Consequently, it will be necessary to import suitable materials to the site to meet the required standards and ensure that the pipeline's installation adheres to both geotechnical and engineering specifications.

In conclusion, the need to import specific materials highlights the importance of adhering to the prescribed standards to achieve optimal pipeline performance and stability, especially in terms of support, protection, and durability under varying environmental conditions.

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