TRANSNET CAPITAL PROJECTS

PORT OF SALDANHA

Geotechnical Investigation Report for Stormwater Management System, Port of Saldanha

Prepared by	N. Magigaba	Project number	1226551
Reviewed by	Z. Pienaar	File number	W(F)317/1/114/40
Date of Issue	12 May 2017	Document number	1226551-H-RPT-01

PORT OF SALDANHA

GEOTECHNICAL INVESTIGATION REPORT FOR STORMWATER MANAGEMENT SYSTEM, PORT OF SALDANHA

INDEX

APPENDICES

A – SITE PLAN
B – INSPECTION PIT LOGS
C – BOREHOLE LOGS (from previous investigations)
D – DCP TEST RESULTS
E – LABORATORY TEST RESULTS
F – PHOTOS

1. INTRODUCTION

1.1 SCOPE OF WORKS

Transnet Ports and Terminals (TPT) in Saldanha is currently proposing the upgrading of the stormwater systems at the Port of Saldanha. The Stormwater Master Plan for the Port of Saldanha covers the areas owned by the National Ports Authority of South Africa and includes the Small Craft Harbour and the Iron Ore Terminal, but excludes the Breakwater. The area covered in this report has been divided into 5 major catchments which are further sub-divided into 21 smaller sub-catchments. The catchments are defined by their physical characteristics including existing drainage system boundaries as well as by the operational activities.

Proposed new development comprise, amongst others, the following infrastructure components at the five areas (Tippler, Stockpile, Multipurpose, Rail Embankment and Causeway Area)

- In all the areas
  - Evaporation ponds
  - Retaining ponds

- At the Ore Jetty and Causeway area, a Water Treatment Plant comprising:
  - Sludge storage tank;
  - Dewatering building;
  - Cloth filter;
  - Clariflocculator;
  - pH correction; and
  - Receiving tank.

- At the Multipurpose and Stockpile area
  - Upgrade to an existing access road;
  - New pavements for the Multipurpose Area.

The objective of the investigation was to investigate founding conditions of 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 materials found; and

- Presenting findings, geotechnical considerations and general recommendations that may have an influence on the design and construction of the proposed structures.

This geotechnical report presents the factual data, analysis of the results of the investigations as well as conclusions and recommendations relevant to development of the site.

1.2 DELIVERABLES IN TERMS OF APPOINTMENT

The scope of work can be briefly summarised as follows:

- Carry out field investigation which includes soil profiling and sampling in accordance with relevant specifications for geotechnical investigations
- Determine the nature, distribution and relevant engineering properties of the soil and rock strata underlying the site in order to advise on the geotechnical characteristics of the materials on site, with specific reference to founding conditions for various structures and suitability of the near surface soils for use in road layer works construction.
- Present all the relevant information in a written geotechnical ground investigation report.

1.3 AVAILABLE INFORMATION

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

- 1:250 000 scale geological map of Cape Town 3318. Geological Survey, printed by the Government Printer, Pretoria, 1990.
- The 1:250 000 scale soil land type map of Cape Town 2628 (Soil and Research Institute, 1985).
- Transnet Capital Projects Port of Saldanha Stormwater Master Plan: Report H340361-0000-10-236-0001.
- Rotary Core borehole information from previous investigations in the area.
- Aerial photographs, sourced from Google Earth®
- Various layout plans were received from

All the abovementioned information has been used in the initial planning and execution phases of this geotechnical investigation.

2. THE SITE

2.1 SITE LOCALITY

Saldanha Bay is located approximately 100 km north of Cape Town on the West Coast. The Port is located on the northern shore of Saldanha Bay. The area along the northern coast of the Bay is characterised by a gently undulating coastal plain with sandy soil and sparse vegetation typical of the West Coast.

Low hills are located to the north and west surrounding the Bay with Malgaskop at 173m above mean sea level located to the west, Karringberg at 175m above mean sea level located to the East and Potsberg on the Langebaan Peninsula at 192.8m above mean sea level located to the south. Granite outcrops frequent this coastal area and surrounding environment, see Figure 1 below:

Figure 1: Aerial view of the Port of Saldanha

The geotechnical study was carried out at the Tippler area, Stockpile Yard, Rail Embankment, Multipurpose Area as well as the Causeway Area as shown in **Drawing 1226551** in Appendix A**.**

In general, the areas under investigation are generally flat and existing infrastructure consist of various buildings, internal access roads. Several subsurface cables, storm water drains, water mains, several subsurface electrical cables and light masts were found to be present. Currently access to the site is by means of the security entrance situated on the western most boundary of the Naval Base.

The extent of the proposed developments in relation to the existing infrastructure is shown on the site plan given in Appendix A of the report.

2.2 CLIMATE

Saldanha normally receives about 249mm of rain per year and because it receives most of its rainfall during winter it has a Mediterranean climate. It receives the lowest rainfall (1mm) in February and the highest (49mm) in June. The average midday temperatures for Saldanha range from 16.4°C in July to 25.1°C in February. The region is the coldest during June when the mercury drops to 8°C on average during the night.

The region where the proposed site is situated is classified as having a climatic N-value (after Weinert, 1980) of less than 5, which indicates a more humid part of the country. Chemical weathering is predominant in this part of the country.

3. NATURE OF THE INVESTIGATION

3.1 DESKTOP STUDY

The investigation for the above site included a desktop study of all the previous works carried out on the site and included a review of the following reports:

i) The Geology of the Cape Town area by J.N. Theron, P.G. Gresse, H.P. Siegfried and J.Rogers (Explanation of Sheet 3318)
ii) Engineering Geology of Southern Africa; Post-Gondwana Deposits, Volume 4, November 1985 by A.B.A. Brink
iii) Port of Saldanha: Construction of New Berthing Facility; Interpretative Geotechnical Report by Transnet Geotechnical Service, dated November 2007
iv) Port of Saldanha: Construction of New Stockpile Areas; Geotechnical Report by Transnet Projects Geotechnical Services, dated January 2007

v) Port of Saldanha: Geotechical Investigation for the Proposed Extensions to the General Cargo Berths; Quay Wall Report by Protekon Design Services, dated July 1995

vi) Port of Saldanha: Paving at the Multi-Purpose Terminal; Geotechnical Report by Transnet Geotechnical Services, dated April 2011

vii) Transnet Saldanha Iron Ore Expansion Tippler 3 Geotechnical Report: Package C002 by AECOM, dated March 2015

Several boreholes, which were found to be relevant to our site have been extracted from the above reports and are included in Appendix C of this report.

3.2 SITE INVESTIGATION

The geotechnical investigations were conducted in February 2017 and comprised test pitting and soil sampling. The site investigation comprised excavation of thirty (30 No) test pits with a tractor loader backhoe (TLB) and conducting fifty seven (57 No) Dynamic Cone Penetrometer Test (DCP).

Test pits were excavated to determine the subsurface soil conditions. The test pits were excavated with a TLB up to a depth that varied between 1,5m and 3,5m or refusal on hard material. Thirty (30 No) test pits were excavated. Coordinates of the test pits were determined using a hand-held GPS on the South African grid with WGS84 coordinate system (Lo 35).

A two-person team carried out the test pitting in order to comply with accepted safety requirements as reflected in the South African Code of Practice (SAICE: 2007). The test pits were profiled by an engineering geologist according to the method proposed in SANS 633 (2009). The excavations were loosely backfilled after completion of soil profiling and sampling.

The test pits were set out and profiled by a team of engineering geologists/ geotechnical engineers in accordance with South African standards (Standards South Africa. South African. National Standard. Profiling, Percussion Borehole and Core Logging in Southern Africa SANS 633:2012.). The Test pit logs are included in Appendix B.

Representative samples were recovered and sent to the SANAS-accredited Engineering Materials Laboratory for testing. Testing included the determination of the Foundation

Indicators (comprising grading analyses – both sieve and hydrometer analyses – and Atterberg Limits), chemical tests (pH and Conductivity) as well as the determination of the compaction characteristics i.e. maximum dry densities (MDD) and optimum moisture contents (OMC) as well as determination of the California Bearing Ratio (CBR).

Hand held Dynamic cone penetration (DCP) tests were carried out at each test pit position. The tests were done to determine the in situ strength of the near surface soils and were done to either refusal or a maximum depth of 3,0m. Two DCP tests were conducted at each test pit, one test on the surface and another one at the base of the pit. The penetration (in mm) to advance the cone for every five blows was recorded. The DCP results are included in Appendix D.

The positions of the test pits are listed below in Table 1 with the detailed test pit logs attached in Appendix B.

Table 1: Test Pit Summary

Test Pit No	Coordinates (UTM WGS84, ZONE 35J)		Depth (m)	Remarks
	Easting	Northing		
TIPPLER AREA				
TTP-01	17°59'57.7"	32°59'56.3"	2,20	Refusal on bedrock
TTP-02	17°59'56.7"	32°59'57.0"	2,00	Refusal on bedrock
TTP-03	17°59'56.6"	32°59'58.1"	1,80	End of hole
TTP-04	17°59'55.9"	32°59'59.0"	1,62	End of hole
STOCKPILE YARD				
STP-01	18°00'07.6"	33°00'07.5"	1,10	End of hole
STP-02	18°00'11.9"	33°00'09.4"	1,30	End of hole
STP-03	18°00'09.5"	33°00'17.3"	1,20	Refusal on Hardpan Calcrete
STP-04	18°00'08.3"	33°00'21.4"	0,90	Refusal on Hardpan Calcrete
STP-05	18°00'03.0"	33°00'29.2"	1,30	End of hole
STP-06	17°59'57.4"	33°00'27.9"	1,40	End of hole
STP-07	17°59'48.0"	33°00'27.7"	1,10	Refusal on boulders

Test Pit No	Coordinates (UTM WGS84, ZONE 35J)		Depth (m)	Remarks
	Easting	Northing		
MULTIPURPOSE AREA				
MTP-01	17°59'25.0"	33°00'48.2"	1,40	End of hole
MTP-02	17°59'28.0"	33°00'46.5"	1,00	End of hole
MTP-03	17°59'32.2"	33°00'46.1"	1,20	End of hole
MTP-04	17°59'32.2"	33°00'51.6"	1,20	End of hole
MTP-05	17°59'28.8"	33°00'59.5"	0,60	Refusal on boulders
MTP-06	17°59'26.1"	33°01'05.5"	1,10	End of hole
MTP-07	17°59'23.3"	33°01'11.9"	0,90	Refusal on boulders
RAIL EMBANKMENT				
RTP-01	17°59'42.4"	33°00'27.1"	1,20	End of hole
RTP-02			1,20	End of hole
RTP-03	17°59'39.8"	33°00'32.8"	1,30	End of hole
RTP-04	17°59'40.0"	33°00'34.6"	0,80	Refusal on boulders
RTP-05	17°59'36.7"	33°00'39.6"	1,20	End of hole
RTP-06	17°59'35.7"	33°00'44.0"	1,30	End of hole
CAUSEWAY AREA				
CTP-01	17°59'20.2"	33°01'19.0"	1,30	End of hole
CTP-02	17°59'16.3"	33°01'27.2"	1,20	End of hole
CTP-03	17°59'13.8"	33°01'28.5"	0,60	Refusal on boulders
CTP-04	17°59'13.5"	33°01'29.2"	1,00	Refusal on boulders
CTP-05	17°59'10.5"	33°01'33.0"	1,40	Refusal on boulders
CTP-06	17°59'10.1"	33°01'33.7"	1,50	End of hole

4. SITE GEOLOGY

According to the 1:250 000 scale geological map, Sheet 3318 Cape Town, The Port of Saldanha is typically underlain at depth by granite of the Cape Granite Suite, that have weathered to varying degrees. Residual soil derived from the in situ weathering of the bedrock is evident in most areas above the bedrock. Younger, typically unconsolidated, transported sediments overlie the bedrock strata; these are typically calcrete and limestone and calcified parabolic sand of Quaternary in age and consist of the Langebaan Formation which was deposited as a coastal spit (aeolianites and channel fill deposits) along the shoreline, the Harbour Beds that occur inland of the coastal sand dunes, the recent surface beach deposits along the shoreline.

5. GEOTECHNICAL EVALUATION OF SITE CONDITIONS

The evaluation given below is based on existing ground levels at the time of the fieldwork. It is recognised that final surface levels may be different to the present levels, but these details were not known at the time of the investigation.

The ground profiles encountered at each of the respective structures are described separately below.

5.1 TIPPLER AREA

The summarised findings of the respective test pits are presented in Table 2 below. Four (4No) test pits were excavated at tippler area site. Information from three (3No) boreholes (BH7, BH8 and BHE3) from previous investigations was also used in evaluating the profile of this area.

The geological profile at the tippler area position may be summarised as follows:

- Topsoil;
- Fill;
- Marine Deposits; and
- Pedogenic Layer.

Topsoil

The topsoil is classified as dry, reddish brown, intact gravelly sand. It has an average thickness of 0,1m. The topsoil was profiled as generally having a loose to medium dense consistency in the test pit.

Fill

The fill is classified as dry, dark grey, cream brown and reddish brown, sandy grave gravelly sand to sandy gravel, domestic waste (plastics and bottles) and bricks. It has an average thickness of 0,80 m. The fill was profiled as generally having loose to medium dense consistency in the test pits.

Quaternary Deposits

The marine deposits were encountered in all the test pits at the site. They comprise a cream brown, off white to grey fine sand to gravelly sand with occasional shell fragments. This layer has loose to medium dense consistency. The marine deposits reach a depth up to 10,3m in BH7.

Pedogenic layer

The pedogenic layer (calcrete) generally underlies the marine deposits and was encountered in TTP-01on this site. This layer comprises a horizon of of-white to cream, very soft rock, hardpan calcrete. From the boreholes the calcrete reaches depths of 7,5m (BH8) and a maximum depth of 15,0m (BHE3).

Table 2: **Test pit summary –Tippler area**

Test Pit No	Topsoil (m)	Fill (m)	Residual Layer(m)	Bedrock(m)
TTP-01	0-0,10	0,10-0,60	0,60-2,10	2,10-2,20
TTP-02	0-0,10	0,10-1,00	1,00-2,00-	
TTP-03	0-0,08	0,08-1,20	1,201,80-	
TTP-04	0-0,06	0,06-0,71	0,71-1,62	

5.2 STOCKPILE YARD

The summarised findings of the respective test pits are presented in Table 3 below. Seven (7 No) test pits were excavated at stockpile yard area site. Information from seven (7No) boreholes (BHTP3, BHTP4, BHTP6, BH9, BH10, BH21 and IOS9) from previous investigations was also used in evaluating the profile of this area.

The geological profile at the stockpile yard position may be summarised as follows:

- Topsoil;
- Fill;
- Marine Deposits; and
- Pedogenic Layer.

Topsoil

The topsoil is classified as dry, reddish brown, intact, silty fine sand to gravelly sand. It has an average thickness of 0,1m. The topsoil was profiled as generally having a loose to medium dense consistency in the test pits.

Fill

The fill at this site is classified as dry, dark grey, cream brown and reddish brown, sandy gravel to gravelly sand and boulders, domestic waste (plastics and bottles) and bricks. The fill was profiled as generally having a dense consistency in the test pits. Some of this fill is subbase layer. The gravels in the fill are iron rich. The fill extends to depth of 5,60 m in borehole IOS9.

Quaternary Deposits

The marine deposits comprise a cream brown, light olive green, light yellowish brown fine sand with occasional shell fragments. This layer has loose to very loose consistency. The marine deposits extends to depth of 19,95m (BH21).

Pedogenic layer

The pedogenic layer (calcrete) generally underlies the marine deposits and was encountered in STP-03 and STP-04 on this site. This layer comprises a horizon of of-white to cream, very soft rock, hardpan calcrete.

Table 3: Test pit summary –Stockpile Yard Area

Test Pit No	Topsoil (m)	Fill (m)	Marine Deposits(m)	Hardpan Calcrete(m)
STP-01		0-0,77	0,77-1,10	
STP-02		0-1,30		
STP-03		0-1,10		1,10-1,20-
STP-04		0-0,80		0,80-0,90
STP-05	0-0,12	0,12-0,57	0,57-1,30	
STP-06	0-0,12	0,12-1,40		
STP-07	0-0,20	0,20-1,10		

5.3 MULTIPURPOSE AREA

The summarised findings of the respective test pits are presented in Table 4 below. Seven (7 No) test pits were excavated at multipurpose site. Information from four (4

No) boreholes (BH1, BH2, BH3 and BH4) from previous investigations was also used in evaluating the profile of this area.

The geological profile at the multipurpose position may be summarised as follows:

- Topsoil;
- Fill; and
- Marine Deposits.

Topsoil

The topsoil is classified as dry, reddish brown, intact, silty fine sand to gravelly sand. It has an average thickness of 0,13m. The topsoil was profiled as generally having a loose to medium dense consistency in the test pits.

Fill

The fill at this site is classified as dry to slightly moist, light creamish white to light olive green, grey and orange brown sandy gravel to gravelly sand, cobbles, boulders, domestic waste (plastics and bottles). The fill was profiled as generally having a medium dense consistency in the test pits.

Quaternary Deposits

The marine deposits comprise a cream white, light olive green, light yellowish brown gravelly sand with occasional shell fragments. This layer has loose to medium dense consistency. This material extends to depths up to18,70 m on the boreholes.

Table 4: Test pit summary –Multipurpose Area

Test Pit No	Topsoil (m)	Fill (m)	Marine Deposits(m)
MTP-01	0-0,30		0,30-1,40
MTP-02	0-0,05	0.05-1,00	
MTP-03	0-0,04	0.05-0,82	0,82-1,20
MTP-04	0-0,05	0.05-1,20	
MTP-05	0-0,10	0,10-0,60	0,57-1,30
MTP-06	0-0,06	0,06-1,10	
MTP-07	0-0,29	0,29-0,90	

5.4 RAIL EMBANKMENT AREA

The summarised findings of the respective test pits are presented in Table 5 below. Six (6 No) test pits were excavated at this site.

The geological profile at the rail embankment position may be summarised as follows:

- Topsoil; and
- Fill.

<u>Topsoil</u>

The topsoil is classified as dry, reddish brown, intact, silty fine sand to gravelly sand. The topsoil was profiled as generally having a loose to medium dense consistency in the test pits.

<u>Fill</u>

The fill at this site is classified as dry, light creamish white, green brown to mottled cream, yellowish brown, sandy gravel to gravelly sand, cobbles and boulders. The fill was profiled as generally having a medium dense consistency in the test pits.

Table 5: Test pit summary –Railway embankment Area

Test Pit No	Topsoil (m)	Fill (m)
RTP-01	0-0,03	0.03-1,20
RTP-02	0-0,04	0.04-1,20
RTP-03	0-0,02	0.02-1,30
RTP-04	0-0,10	0.10-0,80
RTP-05	0-0,20	0,20-1,20
RTP-06	0-0,11	0,11-1,30

5.5 CAUSEWAY AREA

The summarised findings of the respective test pits are presented in Table 6 below. The site includes the following proposed structures:

- Sludge storage tank;
- Dewatering building;
- Cloth filter;
- Clariflocculator;
- pH correction; and

- Receiving tank

Six (6 No) test pits and eight were excavated at this site. Information from four (4 No) onshore boreholes from previous investigations was also used in evaluating the profile of this area.

The geological profile at the causeway area position may be summarised as follows:

- Topsoil;
- Fill; and
- Concrete Paving.

Topsoil

The topsoil is classified as dry, reddish brown, intact, silty fine sand to gravelly sand. The topsoil was profiled as generally having a loose to very loose consistency in the test pits.

Fill

The fill at this site is highly variable and is classified as dry, light creamish white, yellowish brown, sandy gravel to gravelly sand, cobbles and boulders. The fill was profiled as generally having a medium dense consistency in the test pits. A subbase layer was intersected at CTP-06. This fill extends to depth of about 10,5m in the boreholes (AUR.B01A and AUR.B01B).

Paving

This layer was encountered in test pit CTP-06 at the site and comprised of dry, black, dense, asphalt paving with fine gravel.

Table 6: Test pit summary –Causeway Area

Test Pit No	Topsoil (m)	Fill (m)	Paving
CTP-01	0-0,10	0.10-1,30	
CTP-02	0-0,30	0.30-1,20	
CTP-03		0-0,60	
CTP-04		0-1,00	
CTP-05		0-1,40	
CTP-06		0,02-1,50	0-0,02

6. LABORATORY TESTING

6.1 LABORATORY TEST RESULTS

Representative samples were collected for laboratory testing at selected test pit positions and submitted for foundation indicator tests. The test results are attached in Appendix E and summarised in Table 7 below.

Table 7: Laboratory results summary

TP No. (sample depth below NGL [m])	Soil Origin	GM	PI	MDD (kg/m3)	OMC (%)	SWELL (%)	CBR Values at % MOD AASHTO				COLTO / TRH 14	pH	Conductivity (mS/m)
							93%	95%	98%	100%			
TIPPLER POSITION													
TTP1 (0.60-1.30)	Marine Deposits	1.33	NP	1685	8.6	0	11	13	15	16	G8	8.59	
TTP2 (0.20-0.25)	Fill	1.53	NP	1923	8.6	0	22	25	30	34	G6	8.63	0.151
TTP3 (1.20-1.80)	Marine Deposits	1.65	NP	1688	8.8	0	10	13	19	25	G8	8.56	

TP No. (sample depth below NGL [m])	Soil Origin	GM	PI	MDD (kg/m3)	OMC (%)	SWELL (%)	CBR Values at % MOD AASHTO				COLTO / TRH 14	pH	Conductivity (mS/m)
							93%	95%	98%	100%			
MULTIPURPOSE AREA													
MTP1 (0-0.30)	Topsoil	1.84	NP									8.22	0.022
MTP1 (0.30-1.40)	Marine Deposits	1.40	NP	1729	8.5	0	14	15	17	19	G8	9.75	0.087
MTP2 (0.20-0.44)	Fill	1.22	6.9	2010	6.9	0	3	4	5	5	G10	8.51	
MTP2 (0.44-1.00)	Fill	1.76	NP	1887	8.6	0	24	30	44	54	G6	8.63	
MTP3 (0.17-0.34)	Fill	2.07	NP	2090	6.4	0	40	52	80	100	G4	8.20	0.356
MTP3 (0.34-0.62)	Fill	1.24	NP	1931	7.7	0	2	3	4	4	G10	8.56	1.88
MTP4 (0.23-1.20)	Fill	1.40	NP	1923	8.6	0	23	29	42	53	G6	8.93	0.187
MTP6 (0.16-1.10)	Fill	1.35	NP	1975	7.3	0	40	50	69	82	G5	8.36	1.695
CAUSEWAY AREA													
CTP1 (0.10-1.00)	Fill	1.38	NP	2039	7.4	0	38	42	50	58	G6	9.73	0.144

TP No. (sample depth below NGL [m])	Soil Origin	GM	PI	MDD (kg/m3)	OMC (%)	SWELL (%)	CBR Values at % MOD AASHTO				COLTO / TRH 14	pH	Conductivity (mS/m)
							93%	95%	98%	100%			
CTP2 (0.90-1.20)	Fill	1.29	NP	1966	6.8	0	22	27	39	49	G6	8.55	0.937
CTP4 (0.63-1.00)	Fill	1.48	NP	2090	7.1	0	20	27	44	60	G6		
CTP5 (0.34-0.64)	Fill	1.33	NP	1945	8.6	0	22	27	39	50	G6		
CTP6 (0.34-0.64)	Fill	2.23	NP	2039	7.4	0	39	45	58	69	G5		
STOCKPILE YARD AREA													
STP1 (0.90-1.10)	Marine Deposits	1.07	NP	1719	9.2	0	14	15	18	20	G8	9.86	0.0113
STP2 (0.10-0.50)	Fill	1.71	NP	2175	7.1	0	14	15	18	20	G8	8.21	
STP2 (0.79-1.00)	Fill	1.85	NP									8.67	0.0123
STP2 (1.00-1.30)	Fill	1.00	NP									9.99	0.0026
STP3 (0-0.15)	Fill	2.05	5.8									8.23	0.0125
STP3 (0.15-0.37)	Fill	1.33	NP									9.38	0.0813

TP No. (sample depth below NGL [m])	Soil Origin	GM	PI	MDD (kg/m3)	OMC (%)	SWELL (%)	CBR Values at % MOD AASHTO				COLTO / TRH 14	pH	Conductivity (mS/m)
							93%	95%	98%	100%			
STP3 (0.37-1.20)	Fill	1.68	NP									10.62	0.0124
STP4 (0-0.40)	Fill	1.54	5.8	2606	6.1	0	36	45	60	75	G10	8.52	
STP4 (0.40-0.80)	Fill	1.75	NP									9.43	0.258
STP5 (0.57-1.30)	Fill	1.22	NP	1638	9.3	0	14	15	18	19	G8	9.74	0.083
STP6 (0.18-1.40)	Fill	1.55	NP	2040	7.6	0	25	27	30	33	G6	8.44	
STP7 (0.20-0.46)	Fill	1.43	NP									8.36	0.051
STP7 (0.46-0.80)	Fill	1.47	NP	1786	8.8	0	20	26	40	50	G6	8.44	0.011
RAIL EMBANKMENT AREA													
RTP1 (0.13-0.30)	Fill	1.58	NP	2175	7.1	0	22	45	70	100	G5	8.42	0.091
RTP2 (0.28-0.59)	Fill	1.32	NP	1935	7.2	0	19	25	36	50	G6	10.40	0.473
RT.2 (0.59-1.20)	Fill	1.30	NP	1933	7.3	0	19	22	26	30	G7	8.56	

TP No. (sample depth below NGL [m])	Soil Origin	GM	PI	MDD (kg/m3)	OMC (%)	SWELL (%)	CBR Values at % MOD AASHTO				COLTO / TRH 14	pH	Conductivity (mS/m)
							93%	95%	98%	100%			
RTP3 (0.70-1.30)	Fill	1.45	NP	1939	8.6	0	24	27	33	37	G10	9.88	0.113
RTP5 (0.20-0.49)	Fill	1.40	NP	1946	7.2	0	18	25	41	60	G6	9.66	0.446
RTP6 (0.11-0.88)	Fill	1.28	NP	1948	8.6	0	28	32	41	50	G6	8.52	

GM – Grading Modulus **GM- Liquid Limit** **PI – Plasticity Index** **UC – Unified Classification**

MDD. – Maximum Dry Density **OMC – Optimum Moisture Content**

6.2 SUMMARY OF LABORATORY TEST RESULTS

6.2.1 TIPPLER POSITION

The **marine deposits** and **fill** on this site generally consists of silty sand. The layer has a very high (1,33 to 1,65) grading modulus. The fine fractions of this material also exhibit no liquid limit and linear shrinkage, indicating that the material has no plasticity characteristics. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **marine deposits** have a low (1685kg/m^3 to 1688kg/m^3) maximum dry density and moderate optimum moisture content values. The CBR swell values are very low and the tests yielded relatively low to moderate CBR values at densities typically specified in the field (93% to 95%). The material is classified as G8 material according to the TRH 14 guidelines (CSIR: 1987). The material is considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills (**G8**).

The **fill** has a moderate (1923 kg/m^3) maximum dry density and moderate (8,6%) optimum moisture content value. The CBR swell values are very low the tests yielded high CBR values and considered to be suitable for the construction of an engineered fill of subbase layer material and in moderate stiffness of engineered fills (**G6**).

Based on Evans guideline (1977) a soil pH less than 6 indicates serious corrosion potential. The **marine deposits** and **fill** have high pH and low corrosive values hence the materials are considered to be non-corrosive**.**

6.2.2 MULTIPURPOSE PURPOSE AREA

The **marine deposits** and **fill** on this site generally consists of silty sand. The layer has a very high (1,4 to 2,07) grading modulus. The fine fractions of this material also exhibit no liquid limit and linear shrinkage, indicating that the material has no plasticity characteristics. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **marine deposits** have a low (1729kg/m^3) maximum dry density and moderate optimum moisture (8,5%) content values. The CBR swell values are very low and the tests yielded relatively low to moderate CBR values at densities typically specified in the field (93% to 95%). The material is classified as G8 material according to the TRH

14 guidelines (CSIR: 1987). The material is considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills (**G8**).

The **fill** has a moderate (1887 kg/m^3) to high (2090 kg/m^3) maximum dry density and low (6,4%) to moderate (8,6%) optimum moisture content value. The CBR swell values are very low the tests yielded high CBR values at densities typically specified in the field (93% to 95%). The material is classified as G4, G5, G6, G8 and G10 according to the TRH 14 guidelines (CSIR: 1987). Where it was encountered as (**G4**) the material is considered to be suitable for the construction of an engineered fill of base layer material and in high stiffness of engineered fills. The material is considered to be suitable for the construction of an engineered fill of subbase layer material and in moderate stiffness of engineered fills (**G5 & G6**). Where encountered as (**G8**) this material is considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills. Furthermore where the layer is encountered as (**G10**) material it is also suitable for use in construction of subgrade layer material and in low stiffness engineered fills.

Based on Evans guideline (1977) a soil pH less than 6 indicates serious corrosion potential. The **marine deposits** and **fill** have high pH and corrosive values hence the materials are considered to be non-corrosive**.**

6.2.3 CAUSEWAY AREA

The **fill** on this site generally consists of silty sand. The layer has a very high (1,29 to 2,23) grading modulus. The fine fractions of this material also exhibit no liquid limit and linear shrinkage, indicating that the material has no plasticity characteristics. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **fill** has a moderate (1945kg/m^3) to high (2090kg/m^3) maximum dry density and low (6,8%) to moderate (8,6%) optimum moisture content values. The CBR swell values are very low the tests yielded high CBR values at densities typically specified in the field (93% to 95%). The material is classified as G6 according to the TRH 14 guidelines (CSIR: 1987). The material is considered to be suitable for the construction of an engineered fill of subbase layer material and in moderate stiffness of engineered fills (**G5 & G6**).

The **marine deposits** and **fill** have high pH and low corrosive values hence the materials are considered to be non-corrosive**.**

6.2.4 STOCKPILE YARD AREA

The **marine deposits** and **fill** on this site generally consists of silty sand. The layer has a high to very high (1,07 to 2,05) grading modulus. The fine fractions of this material also exhibit no liquid limit and linear shrinkage, indicating that the material has no plasticity characteristics. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **marine deposits** have a low (1719kg/m^3) maximum dry density and moderate optimum moisture (9,2%) content values. The CBR swell values are very low and the tests yielded relatively low to moderate CBR values at densities typically specified in the field (93% to 95%). The material is classified as G8 material according to the TRH 14 guidelines (CSIR: 1987). The material is considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills (**G8**).

The **fill** has a very low (1638 kg/m^3) to very high (2606 kg/m^3) maximum dry density and low (6,1%) to moderate (9,3%) optimum moisture content value. The CBR swell values are very low, the tests yielded moderate to high CBR values at densities typically specified in the field (93% to 95%). The material is classified as G6 and G8 according to the TRH 14 guidelines (CSIR: 1987). The material is considered to be suitable for the construction of an engineered fill of subbase layer material and in moderate stiffness of engineered fills (**G6**). Where encountered as (**G8**) this material is considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills.

Based on Evans guideline (1977) a soil pH less than 6 indicates serious corrosion potential. The **marine deposits** and **fill** have high pH and corrosive values hence the materials are considered to be non-corrosive**.**

6.2.5 RAILWAY EMBANKMENT AREA

The **fill** on this site generally consists of silty sand. The layer has a very high (1,28 to 1,58) grading modulus. The fine fractions of this material also exhibit no liquid limit and linear shrinkage, indicating that the material has no plasticity characteristics. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **fill** has a moderate (1933 kg/m^3) to very high (2175 kg/m^3) maximum dry density and low (7,1%) to moderate (8,6%) optimum moisture content value. The CBR swell values are very low the tests yielded low to high CBR values at densities typically specified in the field (93% to 95%). The material is classified as G5, G6, G7 and G10 according to the TRH 14 guidelines (CSIR: 1987). The material is considered to be suitable for the construction of an engineered fill of subbase layer material and in moderate stiffness of engineered fills (**G5 & G6**). Where it was encountered as (**G7**) the material is also considered suitable for the construction of an engineered fill of selected subgrade layer material and in moderate stiffness of engineered fills. Furthermore where the layer is encountered as (**G10**) material it is also suitable for use in construction of subgrade layer material and in low stiffness engineered fills.

Based on Evans guideline (1977) a soil pH less than 6 indicates serious corrosion potential. The **marine deposits** and **fill** have high pH and corrosive values hence the materials are considered to be non-corrosive**.**

6.3 DYNAMIC CONE PENETROMETER TEST RESULTS

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 a distance of 575 mm and the results are expressed as the penetration rate (PR) in mm per blow.

Table 8 below shows the summarised results of from the DCP tests. Further information and graphs relating to the DCP results are attached in Appendix D.

The results of the DCP field tests have been used to evaluate in situ strength or CBR values of the near surface subsoils. Cognisance of the moisture content, potential gravels and/or boulders in the subsoil profile should be taken into account when using these DCP-derived CBR values. The results of the DCP tests are given in Appendix D and should be referred to for specific details. The tables below summarises all the DCP test results in terms of the average in-situ CBR values with depth for tests conducted from surface and at reduced levels. The tables present the average CBR obtained from the in-situ DCP test values at specific depth intervals with different colours denoting the different layers as specified in the COLTO Specifications. Below all the tables, a legend specifying the various colours according to the CBR values obtained is presented. The summary of the tables does not include the laboratory results which means it does not take certain parameters such as the plasticity index into account.

Table 8 Summary of average in situ CBR values (%) along the Tippler Area

Depth (mm)	TTP1A	TTP2A	TTP3A	TTP4A	TTP1B	TTP2B	TTP3B	TTP4B
0-200	95	87	100	87	15	84	7	87
200-400		100		100	11		18	100
400-600					92		34	
600-800							24	
800-1000							18	
1000-1200							19	
1200-1400							48	

Table 9 Summary of average in situ CBR values (%) along the Causeway Area

Depth (mm)	CTP1A	CTP2.1A	CTP2.2A	CTP3A	CTP4A	CTP5.1A	CTP5.2A	CTP1B	CTP2B
0-200	32	100	58	92	92	79	100	0	86
200-400	6	75						4	26
400-600	72	49						10	9
600-800	72	47						12	5
800-1000	53	100						13	100
1000-1200	62	100						80	100
1200-1400		100							
1400-1600		100							

Table 10 Summary of average in situ CBR values (%) along the Multipurpose Area

Depth (mm)	MTP1A	MTP2A	MTP3A	MTP4A	MTP5.1A	MTP1B	MTP2B	MTP4B
0-200	100	100	100	89	16	0	94	0
200-400	97	100		95	95	7		88
400-600	100			63		14		100
600-800	100			55		25		
800-1000	100			24		75		
1000-1200	100			18		100		
1200-1400	100			12		100		
1400-1600	100			14		100		

Table 11 Summary of average in situ CBR values (%) along the Causeway Area

Depth (mm)	RTP1A	RTP2A	RTP3A	RTP4A	RTP5A	RTP6A	RTP6B	RTP1B	RTP2B	RTP3B	RTP5B	RTP6B
0-200	47	100	60	29	89	94	82	14	0	16	0	0
200-400	100		100	98				61	23	16	12	6
400-600				88				71	82	16	8	20
600-800								82	99	67	10	62
800-1000								35	64	100	9	92
1000-1200								37	70		8	88
1200-1400								59			71	46
1400-1600								29				74

Table 12 Summary of average in situ CBR values (%) along the Stockpile Yard Area

Depth (mm)	STP1A	STP2A	STP4A	STP5A	STP6A	STP7A	STP1B	STP2B	STP5B	STP6B
0-200	99	93	100	94	100	34	10	0	0	0
200-400	100					100	30	7	6	5
400-600							46	26	14	27
600-800							57	57	19	7
800-1000							84	67	21	6
1000-1200							76	69	23	5
1200-1400							70	68	30	70
1400-1600							58	63	25	

7. GEOTECHNICAL EVALUATION OF SITE CONDITIONS

7.1 EXCAVATION CLASSIFICATION

Excavation procedures likely to be encountered on the site have been evaluated in terms of the SANS 1200D - Earthworks classification system. In terms of this classification system, *soft and boulder excavation* conditions are expected to occur to beyond a depth of 1,00m generally in the sandy/gravel/boulder fill horizons. Depending on the depth of excavations, allowances for possibly working beneath the water table (if encountered) should be made and overbreak of excavation sides may occur and should be taken into account.

7.2 SOIL PERMEABILITY (LINING FOR PONDS)

Representative samples were collected for laboratory testing at selected test pit positions and submitted for permeability tests in order to measure the ease in which water can flow through a soil volume of the different soil types tested. The test results are attached in Appendix E and summarised in Table 13 below.

Table 13: Permeability results for the various sites

Area	Soil Type	Permeability (cm/s)
Tippler Area	Fine sand to sandy gravel	$1{,}45 \times 10^{-3}$ to $4{,}45 \times 10^{-4}$
Causeway Area	Fine sand	$1{,}50 \times 10^{-4}$ to $2{,}89 \times 10^{-4}$
Stockyard Pile Area	Fine sand to gravelly sand	$1{,}63 \times 10^{-3}$ to $1{,}34 \times 10^{-6}$

In a classification of soils according to their coefficients of permeability, Terzaghi and Peck (1967), the abovementioned subsoil horizons are general of low to moderate permeability (k between $1{,}45 \times 10^{-3}$ and $1{,}34 \times 10^{-6}$m/sec).

Leakage from a pond depends on the permeability of the material that lines the pond. The permeability of soil present at the pond site (in-situ soil) is important in determining the type of pond liner that is required, particularly as it might be able to be used to line the pond.

Due to the low and moderate permeabilities of the in-situ soils on site, it is highly advisable that the following solutions be used:

- Concrete linings be used on site, or
- Clayey materials be imported onto the site,
- Re-use of existing material is only advised if the materials are stabilised with cement and well compacted to decrease the soils permeability and void ratio (in the case of gravelly fills).

7.3 MATERIAL SUITABILITY

The variable ***fill*** material that covers the sites typically classifies as G4 trough to G10 quality materials in terms of the COLTO and TRH14 classification. The in-situ moisture content of the soils is typically dry of optimum moisture content and only locally close to/or wet of the optimum moisture content. Therefore, depending on the type of material used, wetting and/or drying may be required as part of the in-situ pavement foundation preparation and re-use of the soils in layerworks. The laboratory results indicate typical optimum moisture content values from 5,6% to 9,2%.

The ***marine deposits*** are classified as G8 material according to the TRH 14 guidelines and are considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills.

7.4 GENERAL FOUNDING CONDITIONS

An evaluation of the founding conditions has been carried out based on assumptions as the foundation information was unknown at the time of writing this report. Various settlement calculations have been done and depending on the type of structure (lightly to heavy load), the depth of the foundation level and foundation type, the results can be summarized as follows:

- In the vicinity of the ***Sludge storage tank***, using a bearing pressure of approximately 100kPa with a circular footings size of 15,2m diameter, and a founding depth of 5,50m below NGL, total settlements in the order of 21mm are to be expected.
- Using a footing size of 1,5m by 1,5m founded at 1,5m from NGL, and using a bearing pressure of 100kPa for the ***Dewatering Building***, total settlement was calculated to be in the order of 4mm

- For the ***Cloth filter*** using a footing size of 1,5m by 1,5m founded at 2,0m from NGL, and using a bearing pressure of 100kPa for the Dewatering Building, total settlement was calculated to be in the order of 4,0mm
- In the vicinity of the ***Clariflocculator***, using a bearing pressure of approximately 100kPa with a circular footings size of 8,0m diameter, and a founding depth of 5,0m below NGL, total settlements in the order of 16,0 mm are to be expected.
- Using a footing size of 1,5m by 1,5m founded at 1,5m from NGL, and using a bearing pressure of 100kPa for the ***pH Correction & Rapid Mixing structure***, total settlement was calculated to be in the order of 4mm
- At the proposed ***Receiving tank***, using a bearing pressure of approximately 100kPa with a circular footings size of 16,5m diameter, and a founding depth of 0,50m below NGL, total settlements in the order of 24mm are to be expected.
- Using a footing size of 1,5m by 1,5m founded at 1,5m from NGL, and using a bearing pressure of 100kPa for the ***Chemical dosing storage***, total settlement was calculated to be in the order of 4mm

GENERAL FOUNDING RECOMMENDATIONS FOR LIGHT STRUCTURES

Although allowable bearing pressures of between 100kPa and >250kPa is achievable in the fill for shallow foundations, these soils in its in-situ state are considered unsuitable as a founding medium for the proposed structures due to the variable consistency of the soils. Due to heavy anticipated loading of some of the structures, settlement problems are anticipated if founding within these soils.

The choice of the most suitable founding solution is dictated by various factors. The main geotechnical factors to be considered are:-

- The variable subsoil conditions in terms of bearing and settlement in both the fill and the marine deposits.
- The structural loads, required tolerances and sensitivity of the structure to settlement (at the time of preparation of this report, the particular load cases and required tolerances of the structure were not available).

In order to reduce settlement and improve the allowable bearing capacity it is recommended that the sludge drying facility and the tanker bay facility should be founded on an engineered soil raft using conventional footings (strip or pad footings).

This will involve the removal of the fill/marine soils below the footings to a depth and width of at least 1,5 times the foundation width. The in-situ soil at the base of the excavation should then be compacted to at least 95% Mod AASHTO. The removed material should be placed in compacted layers not exceeding 150mm to 95% Mod AASHTO density at the optimum moisture content up to the desired founding levels. A bearing capacity of at least 100kPa is achievable within the soil raft.

For the heavier loaded and settlement sensitive structures a piled foundation solution is advisable. Precast piles (designed to carry the load in friction) driven into the very dense sandy Harbour Beds are likely to be the most suitable pile type.

The above values should only be used for conceptual design and must be revised when the actual foundation configuration and loads are known. As part of the design of the foundations, calculations for bearing capacity failure must also be conducted. The settlement of a foundation is almost always the governing criteria and it is therefore proposed that the bearing capacity evaluation be conducted for the actual foundation configurations loads.

7.5 FOUNDATION OF TANKS

The structures can be founded at a depth of 0,5m below the ground level. The preparation of the foundation involves the following:

- Strip and spoil the upper 150mm of topsoil in an area 2m wider than the footprint of the water tank. Roots and organic material in the footprint of the structures must also be removed.
- Remove the in-situ material in and 1 m wider than the footprint of the water tank to a depth of 0,5m. Excavations must be battered at 60° to ensure stability of the excavation slopes.
- Compact the base of the excavation to 93% Mod AASHTO density to a minimum depth of 150mm.

Excavations for the individual footings can then proceed, ensuring that all loose material is removed in the base of the excavations.

It is furthermore recommended that the remainder of the excavation be backfilled to 200mm above natural ground level with compacted material (minimum of 93% Mod AASHTO density). An impervious barrier (e.g. concrete slab) must be created at this level to prevent saturation of the soil profile from spillage or leaking pipes. The barrier must be sloped to drain away from the structure.

7.6 ROCK MATTRESS

Depending on the settlement sensitivity of the structures founded on shallow footings, a rock mattress compacted by a 5-10t vibratory compactor will eliminate differential settlement. The rock fill is to be compacted in 300-500mm thick layers.

7.7 WATER INGRESS INTO SOIL BELOW STRUCTURES

Water could enter the soil from surface runoff during the rainfall and due to leaking pipes and overtopping of the ponds. This can reduce bearing capacity by 40% and cause differential settlement. In view of this an impervious barrier (concrete slab below the water ponds) should be installed.

7.8 DRAINAGE MEASURES

Drainage of the site is required to avoid ponding of water close to the structure. All water bearing services should be of a high quality and flexible to accommodate some movement without damage.Water must be kept away from the foundations. The following drainage precaution must be adhered to:

- No accumulation of surface water is permitted and the entire development must be properly drained;
- A 1m apron must be constructed around each building to keep surface runoff away from foundations;
- Waterborne sewerage reticulation must be installed. All water services should be sleeved;

All trenches and excavation works must be properly backfilled and compacted in order to prevent them from functioning as French drains. Backfilling should be done at optimum moisture content, in 150mm thick layers to at least 90% of modified AASHTO density.

8. PAVEMENT DESIGN AT THE MULTIPURPOSE TERMINAL AREA

8.1 EXCAVATION CLASSIFICATION

Excavation procedures likely to be encountered on site have been evaluated in terms of the SANS 1200D - Earthworks classification. In terms of this classification system, *soft excavation* conditions are expected to occur to variable depths of 0.15m and 1.55m. Below this, *boulder and/or hard rock excavation* should be allowed for in the stabilised structural road layers and/or hardpan calcrete. Blasting may be required in the harder layers.

Depending on the depth of excavations, allowances for possibly working beneath the water table should be made and over break of excavation sides may occur and should be taken into account.

8.2 EVALUATION FOR EARTHWORKS

The results of the fieldwork and laboratory testing indicate that the near surface soil conditions on the site are variable in terms of consistency and composition. To a large extent, most of the variability will be removed by the preparatory earthworks for the multi-purpose terminal paving i.e. assuming the level of the paving will be similar to the existing ground levels.

The soil conditions of test pit MTP1 are considered to be the controlling conditions for the paving design. An in situ CBR of 10% has been taken to represent the strength of the in-situ subgrade. The nature of the subgrade soils are such that after excavation of the overlying soils, in-situ compaction will be required. In this regard, the in-situ surface should be ripped to a depth of 150mm and the moisture content modified to within 2% of the optimum moisture content. Thereafter it should be compacted to a minimum density of 93% Mod AASHTO density.

The excavated soils are expected to comprise the surface layer of sandy gravel (crusher run), the 150mm thick layer of stabilized sand (in the vicinity of MTP1), and the underlying silty sand and gravelly (mainly calcrete) sand. None of the soils meet the requirements for the subbase layer required below the paving and would therefore be spoiled.

8.3 DESIGN PREMISE

The information that was provided on which the paving design has been based is as follows:

- *None of the vehicles that will be using the paved area will be carrying any*
- *load, ie unloaded static and dynamic loading conditions will apply.*
- *Forklift, the 42 tonner is the heaviest with a gross weight of 58 tons.*
- *Front end loader 26,4 tons.*
- *Haulers 24,4 tons.*

In light of the limited information provided by the Owner, several assumptions were made. These are described below and should be agreed upon by the Owner before commencing with the construction.

- *20 year design life, 20 of each vehicle movement per day over any given area of the pavement*
- *Dynamic loading factors to account for braking and cornering*
- *Existing buried services on the site will either be relocated prior to construction or alternatively, are of sufficient depth not to be damaged by compaction of the subgrade at an approximate depth of 0,5m below the levelof the final paving surface. Compaction of any relocated service trench backfills within the area of the paving is to be the same as the adjacent soil mass after the in situ compaction of the subgrade to prevent uneven support of the paving and paving layers*
- *The final level of the paving will be similar to the existing ground levels*
- *The wheel loads on the forklift (unloaded) are the critical wheel loads and have been used in the analysis*
- *A static load of 140kN occurs on each side of the rear axle which, when the dynamic factors are applied, gives a critical design load of 238kN*
- *A tyre pressure of 1MPa*
- *Paving will comprise unreinforced, cast in situ C32/40 concrete, cast in panels with provision made for load transfer and surface water drainage.*
- *No mention was made of loads due to stacking and/or storage and therefore only vehicle loads have been taken into account*

8.4 PAVING DESIGN

Using the principle of Material Equivalence Factors, various pavement options can be determined. These are described as follows:

8.4.1 OPTION 1

200mm thick cast in situ 30MPa concrete on
150mm G2 (graded crushed stone) on
Compacted in situ subgrade

8.4.2 OPTION 2

80mm thick, 200 by 100mm in plain, interlocking concrete blocks on
30mm bedding sand on
200mm C1 (cement stabilized G2 graded crushed stone) on
150mm G2 (graded crushed stone) on
Compacted in situ subgrade

8.4.3 OPTION 3

50mm asphalt (modified bitumen, Salviacim or equivalent) on
150mm G2 (graded crushed stone) on
300mm (2 layers 150mm thick) C1 on
Compacted in situ subgrade

In all cases, the in situ preparation of the subgrade beneath the pavement layers is to comprise rip to 150mm, modify the moisture content to within 2% of the optimum moisture content, and compact to a minimum density of 95% Mod AASHTO effort. The G2 graded crushed stone layer should be compacted to a minimum of 88% of bulk relative density, the C1 base to 98% Mod AASHTO density and the C1 subbase to 96% Mod AASHTO density.

8.5 PAVING DESIGN AT THE STOCKPILE AREA

The condition of the existing road in the Stockpile area generally indicates that the Surfacing material has become thin. The subsoils in this area are mainly indicated in test pits STP3 and STP4.

STP3 can be summarised as follows:

20mm Surfacing
130mm Base / Subbase (Gravel)

180mm Selected Fill (Silty Sand)

730mm Selected Fill / Subgrade (Sandy Gravel)

In-situ Hardpan Calcrete

STP4 can be summarised as follows:

20mm Surfacing

80mm Base / Subbase (Gravel)

300mm Subbase / Selected Fill (Sand Gravel)

400mm Selected Fill / Subgrade (Gravel and small Boulders)

In-situ Hardpan Calcrete

As seen from the two test pits the road design has slight variations and the upgrade would have to take this factor into account.

The **TRH4** and **South African mechanistic paving design procedure** were followed for the asphalt pavement design of the road. The typical pavement design for "Pavement Class" ES1, with failure defined at a 20mm rut would be as follows:

40mm Asphalt Surfacing

150mm G3 Base Course

150mm G5 Subbase

150mm G7 Selected Layer

150mm G9 Selected / Subgrade Layer

300mm Pioneer Layer (*Optional dependant on the in situ conditions)

The existing road would therefore have to be upgraded to include, at a minimum, a thicker surfacing (40mm) and base (150mm) layer.

9. CONCLUSION

In general, this geotechnical ground investigation report covers all geotechnical aspects with regard to the proposed infrastructure developments at the Saldahna storm water developments. The report covers in detail the nature, distribution and relevant engineering properties of the soil and rock strata underlying the site with specific reference to founding conditions for proposed structures and pavement layers for roads.

It should be noted that the information presented in the report and summarised in the tables have been taken from the proposed geotechnical fieldwork and results of the laboratory tests. A certain amount of interpretation was necessary in the generalisation of the results during the evaluation of the various structures sites, along the boundary wall and access roads.

During construction, site conditions should be constantly monitored to ensure that the actual conditions are not at variance with the generalisations made in the report. Should variations be found, these areas would be treated on an ad hoc basis at the time.

N.MAGIGABA
ENGINEERING GEOLOGIST

10. REFERENCES

1. Guidelines for Soil and Rock Logging in South Africa, 2nd impression 2001, eds A.B.A Brink and R.M.H Bruin, Geoterminology Workshop organised by AEG, SAICE and SAIEG, 1990.

2. Geological Survey (Council for Geoscience), 1986. 1:250 000 Geological Map Sheet #2628 East Rand. Pretoria.

3. Jennings, J E B, Brink, A B A and Williams, A A B, (1973). Revised Guide to Soil Profiling for Civil Engineering Purposes in Southern Africa. The Civil Engineer in S A, p 3-12. January 1973.

4. Kijko, A, Graham, G, Bejaichund, M, Roblin, DL and Brandt, MBC. Probabilistic Seismic Hazard Maps for South Africa. Council for Geoscience, 2003.

5. Partridge TC, Wood CK and Brink ABA "Priorities for urban expansion within the PWV metropolitan region: The primacy of geotechnical constraints". South African Geographical Journal, Vol. 75, pp 9 – 13, 1993.

6. Soil and Irrigation Research Institute, 1985. 1:250 000 Land Type Map Sheet # 2628 East Rand. Pretoria.

7. South African Institution of Civil Engineering. Geotechnical Division. 2007. The Safety of Persons Working in Small Diameter Shafts and Test Pits for Geotechnical Engineering Purposes –Code of Practice, First Edition.2007.

8. Weinert, H.H. 1980. The Natural Road Construction Materials of Southern Africa. Academica. Pretoria.

10. Cernica, J.N., (1995), Geotechnical Engineering: Foundation Design. John Wiley & Sons, Inc. New York, 1995.

APPENDICES

APPENDIX A

SITE PLANS

APPENDIX B

INSPECTION PIT LOGS

APPENDIX C

BOREHOLE LOGS

(from previous investigations)

APPENDIX D

DCP RESULTS

APPENDIX E

LABORATORY TEST RESULTS

APPENDIX F

PHOTOS