GEOTECHNICAL INVESTIGATION FOR REDHOUSE CHELSEA ARTERIAL AND WALKER DRIVE EXTENSION, PORT ELIZABETH, EASTERN CAPE

MAY 2010

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VERIFICATION PAGE

Form 403.1

Rev 10/06

REDHOUSE CHELSEA ARTERIAL AND WALKER DRIVE EXTENSION

GEOTECHNICAL INVESTIGATION REPORT

JOB NO: 400359	**DATE**: May 2010	**REPORT STATUS:** Final Report

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SYNOPSIS:

REDHOUSE CHELSEA ARTERIAL AND WALKER DRIVE EXTENSION GEOTECHNICAL INVESTIGATION, PORT ELIZABETH, EASTERN CAPE

KEY WORDS:

Geotechnical, geology, groundwater

QUALITY VERIFICATION

This report has been prepared under the controls established by a quality management system that meets the requirements of ISO9001: 2000 which has been independently certified by DEKRA Certification under certificate number 90906882

Quality Management ISO 9001:2008
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REDHOUSE CHELSEA ARTERIAL AND WALKER DRIVE EXTENSION

GEOTECHNICAL INVESTIGATION REPORT

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REDHOUSE CHELSEA ARTERIAL AND WALKER DRIVE EXTENSION

GEOTECHNICAL INVESTIGATION REPORT

1. INTRODUCTION

This report presents the results of a geotechnical road prism investigation undertaken for the proposed Redhouse - Chelsea Arterial road and extension to Walker Drive in Port Elizabeth. This road construction project involves the construction of new interchanges on Cape Road and the N2 National Road and the construction of an arterial road linking Cape Road, the N2 and Walker Drive.

The project area is indicated in Figures 1a and 1b in **Appendix A**. The layout of the proposed new roads is indicated in Figure 3, also in **Appendix A**.

The objectives of the investigation are as follows:

- Assess of the subgrade conditions along the routes;
- Assess the quality of the in-situ soils for use as construction materials;
- Identify potential problem areas and provide recommendations for their remediation;
- Assess the need for further investigations.

It must be noted that the recommendations and conclusions made in this report are based upon results and information obtained at specific investigation points. The assessment of the overall geotechnical conditions along the respective routes have been inferred by professional judgement made from the interpolation and extrapolation of the point information gained from trial holes, as well as a visual assessment of surface features. Should significant variations from the inferred conditions become apparent during construction, the advice of a geotechnical professional must be sought.

2. APPOINTMENT

Terratest were appointed to proceed with the investigation by Mr F Duze on behalf of BKS (Pty) Ltd in a letter dated 25 March 2010 (Referenced J00488/letter/050FD). The appointment was based on a quotation submitted on 28th September 2009.

3. DATABASE

The following information was provided by BKS (Pty) Ltd:

- Drawing titled “Geotechnical Trial Hole” (BKS drawing number J00488-01-17-001-D-00 dated 12/04/2010) showing the prescribed trial hole positions.
- The trial hole coordinates in WGS84 projection.

4. FIELDWORK METHODOLOGY

The fieldwork undertaken during the investigation entailed the following activities.

4.1. Trial Hole Excavation

Forty mechanically excavated trial holes were dug at pre-determined positions, as specified by BKS (Pty) Ltd. The trial pit positions were surveyed and marked on site prior to the field investigations. The pits were excavated using a Case 695 Super-R Tractor-Loader-Backhoe (TLB) to depths of between 0.25 and 2.9mm below ground level. The trial holes were profiled according to the method of Jennings, Brink and Williams, 1973 (*Revised Guide to Soil Profiling for Civil Engineering Purposes in Southern Africa*).

The trial holes were loosely backfilled immediately after the profiling and sampling was completed.

The trial pit profile descriptions and photographs are presented in **Appendix B**.

4.2. Sample Recovery & Laboratory Testing

Representative disturbed samples were recovered from selected soil and weathered rock layers for laboratory analysis. The following laboratory testing was undertaken:

- Sixteen (16) Modified AASHTO and California Bearing Ratio tests
- Thirty (30) Road Indicator tests
- Four (4) Foundation Indicator tests

Testing of samples was undertaken by Labco Materials Testing Services (Pty) Ltd, a South African National Accreditation System-accredited (SANAS) laboratory.

The laboratory test results are summarised in Section 7 and are attached in **Appendix C**.

4.3. Dynamic Cone Penetrometer (DCP) Testing

Fifteen Dynamic-Cone-Penetrometer (DCP) tests were conducted alongside selected trial pit positions to depths of 1.0 m below existing surface level or refusal, whichever occurred first.

The DCP apparatus consisted of a 10kg weight falling from a drop height of 450mm onto a shaft with a 25mm diameter cone. The blow counts per 300mm of penetration were recorded.

The DCP test results are attached in **Appendix D**.

5. ROUTE ENVIRONMENT AND PHYSIOGRAPHY

5.1. Topography and Drainage

The topography of the study area is gentle and slope gradients are less than 3 degrees over much of the site. The topography is characteristic of the Southern Coastal Plain that extends from Port Elizabeth in an easterly direction.

A stream flows in a general north-westerly to south-easterly direction across the project area. The proposed arterial road will cross the stream north of the N2. A minor tributary of the stream is located on the southern side of the N2. This drainage feature runs roughly parallel to the highway, approximately 130m to 230m from the existing road reserve boundary.

Ground water seepage was only noted in one trial pit excavated during the investigation (TH 2 at 1.43m below ground level). The investigation was, however, undertaken during a period of unusually low rainfall.

The flat topography over certain areas of the site may result in poor drainage conditions during the wetter winter period or after heavy rains. Given the topography and the presence of relatively impermeable bedrock at shallow depth, shallow ground water conditions may be encountered, particularly during the rainy season. The presence of pedogenic ferricrete or the presence iron oxide colouration was noted in many of the trial holes. Ferricrete is an accumulation of iron and frequently manganese oxides and hydrates. This commonly occurring pedogenic phenomenon is indicative of a shallow, seasonal water table. This process takes place between the limits of a fluctuating water table and results in the formation of mottles and hard concretions, often with black centres, known as nodular ferricrete. With time the concretions may coalesce to form a continuous mass, commonly known as ferricrete.

5.2. Vegetation & Landuse

The vegetation over much of the site consists of low Fynbos scrub. Stands of Black Wattle occur to the south of the N2.

Much of the area is currently vacant and is used for recreation and for subsistence cattle grazing.

5.3. Climate

Weinert (1964), through his work on basic igneous rocks in southern Africa, demonstrated that mechanical disintegration is the predominant mode of rock weathering in areas where his climatic “N-value” is greater than 5, while chemical decomposition predominates where the N-value is less than 5. Weinert's climatic N-value for the Port Elizabeth area is approximately 2.6. This implies that chemical decomposition is the dominant mode of rock weathering.

The N-value is calculated from climatic data as follows:

$$N = 12.E_j / P_a$$

Where: E_j = evaporation during January
P_a = annual precipitation

6. GEOLOGY & ENGINEERING GEOLOGY

The regional geology of the project area is depicted on Figure 2, **Appendix A**.

The project area is underlain by rock units of the Peninsula Formation of the Table Mountain Group, Cape Supergroup. The Peninsula Formation is the main unit of the Cape Supergroup and consists of quartzitic sandstones with minor shale and conglomerate. The rock units display a variety of sedimentary structures such as bedding and cross bedding and have undergone large-scale folding and faulting. The rock units therefore exhibit varying degrees of fracturing and may be brecciated to a fine powder in fault zones.

Colluvial, fluvial, pedogenic and residual soils were found to overly the bedrock. Fill material was noted at some test positions.

An overview of the various geological materials encountered in the trial holes are provided hereunder.

6.1. Fill Material

Fill material, generally consisting of re-worked sandy soils and containing building rubble, was observed at shallow depths (0.22m to 0.94m) in four trial holes.

6.2. Colluvial Soils

Colluvial soils have been transported and deposited by non-fluvial processes. It is likely that some of the soils described as colluvial may have an aeolian (windblown) origin.

The colluvial soils were encountered in 14 trial holes to a maximum depth of 0.51m below ground level. The soils were fairly consistent in composition and typically consisted of silty fine sand with occasional cobbles and boulders. The soil consistency was generally described as "loose" to "very loose", although "medium dense" colluvial soils also were recorded.

6.3. Fluvial Soils

Fluvial soils have been transported and deposited by rivers or streams.
Fluvial soils were observed in 23 trial holes to depths of between 0.17m and 1.39m. The soils were fairly similar in composition to the colluvial soils discussed above, but typically contained rounded pebbles and cobbles that are indicative of their fluvial origin. The soil consistency was typically described as “very loose” to “loose”.

6.4. Pedogenic Soils

Pedogenic ferricrete, in the form of nodular to honeycomb ferricrete, was encountered in 8 trial holes. Signs of ferruginisation were also observed in other trial holes. The ferricrete layer was typically observed between the transported soils and the underlying bedrock.

Based on the trial holes, the ferricrete appears to be best developed in two areas of the site. Ferricrete was encountered in TH12, TH13, TH15 and TH18 in the southern section and in TH28, TH32, TH33 and TH34 in the northern section.

The TLB was unable to excavate through the ferricrete at TH15, TH18, TH32 and TH33.

6.5. Residual Soils

Residual soils, formed from the complete in-situ weathering of the underlying sandstone bedrock, were encountered in 11 trial holes. The soils were more variable in composition that the overlying soils and ranged from silty fine sand to sandy gravel. The soils were typically described as “loose” to “medium dense” in consistency.

6.6. Peninsula Formation

Weathered sandstone rock of the Peninsula Formation was observed in 36 of the 40 trial holes at an average depth of 0.66m, ranging from 0.07m to 2.88m below ground level. Refusal of the TLB occurred at shallow depth on hard ferricrete in the four trial holes where sandstone was not encountered.

The sandstone showed varying degrees of weathering, typically ranging from “completely weathered” to “highly weathered” near the rock head surface to “medium weathered” at refusal depth of the TLB.

7. ROAD PRISM INVESTIGATION

The material properties as defined by the profiling and laboratory testing are summarised in Table 7.1 overleaf.

The results of the DCP tests are summarised in Table 7.2.

TABLE 7.1: SUMMARY OF ROAD PRISIM TRIAL HOLES

TH	DEPTH (m)	DESCRIPTION	GM	PI	LS	MMD/ OMC	CBR AT			SWELL	CLASS	REMARKS
							95%	93%	90%			
TH 1	0-0.51	Silty fine sand (Colluvium)	0.94	NP	0.0							
	0.51-0.86	Weathered sandstone										Near refusal at 0.86m
TH2	0-0.32	Silty fine sand (Colluvium)										
	0.32-0.75	Sandy gravel (Fluvial)	2.51	SP	0.5	2204/5.8	87	80	70	0.0	G4	
	0.75-1.63	Weathered sandstone										Near refusal at 1.63m Ground water seepage below 1.43m
TH3	0-0.94	Fill (fine sand, pebbles, bricks)										
	0.94-2.12	Weathered sandstone (recovered as sandy gravel)	2.21	SP	1.0							Near refusal at 2.12m
TH4	0-0.26	Silty fine sand (Colluvium)										
	0.26-0.85	Weathered sandstone										Near refusal at 0.85m
TH5	0-0.56	Sand (Fluvial)	0.84	NP	0.0							
	0.56-0.87	Sandy gravel (Residual)										
	0.87-1.15	Weathered sandstone										Near refusal at 1.15m
TH6	0-0.62	Silty fine sand (Fluvial)										
	0.62-1.13	Weathered sandstone (ferruginous)	2.15	17	8.5							
	1.13-1.37	Weathered sandstone										Near refusal at 1.37m
TH7	0-1.08	Sand (Fluvial)	0.79	NP	0.0	1924/10.2	14	9	5	0.0	G9	
	1.08-2.54	Sand (Residual)										
	2.54-2.88	Weathered sandstone										Near refusal at 2.88m
TH8	0-0.07	Gravely sand (Colluvium)										
	0.07-0.25	Weathered sandstone										Near refusal at 2.88m
TH9	0-0.17	Silty sand (Fluvial)										
	0.17-0.63	Weathered sandstone										Near refusal at 0.63m

TH	DEPTH (m)	DESCRIPTION	GM	PI	LS	MMD/ OMC	CBR AT			SWELL	CLASS	REMARKS
							95%	93%	90%			
TH10	0-0.06	Silty sand (Colluvial)										
	0.06-0.31	Gravely sand (Residual)										
	0.31-0.85	Weathered sandstone										Near refusal at 0.85m
TH11	0 -0.11	Silty sand (Colluvium)										
	0.11-0.67	Sandy silt (Residual / Pedogenic)	0.94	3	2.5							
	0.67-1.32	Gravely silty sand (Residual)	1.21	NP	0.0							
	1.32-1.89	Weathered sandstone										Near refusal at 1.89m
TH12	0-0.55	Silty sand, pebbles (Fluvial)										
	0.55-1.15	Ferricrete (recovered as sandy gravel)	2.48	NP	0.0	2333/6.8	22	21	20	0.0	G7	
	1.15-2.95	Weathered sandstone (Recovered as sandy silt)	0.26	7	4.5							No refusal
TH13	0-0.57	Silty gravely sand (Fluvial)	1.7	NP	0.0							
	0.57-1.12	Ferricrete										
	1.12-2.08	Weathered sandstone										Near refusal at 2.08m
TH14	0-0.29	Gravely sand (Colluvium)										
	0.29-0.66	Sandy gravel (Residual sandstone)	2.54	4	2.5							
	0.66-1.62	Weathered sandstone (Recovered as silty sand)	0.75	3	1.5	2046/10.5	47	44	39	0.0	G7	Near refusal at 1.62m
TH15	0-0.81	Sand (Fluvial)	0.89	NP	0.0	1930/10.8	27	25	21	0.0	G7	
	0.81-1.12	Ferricrete										Near refusal at 1.12m
TH16	0-0.45	Silty sand (Fluvial)										
	0.45-0.69	Weathered sandstone										Near refusal at 0.69m
TH17	0-0.13	Silty sand (Colluvial)										
	0.13-0.74	Weathered sandstone										Near refusal at 0.74m

TH	DEPTH (m)	DESCRIPTION	GM	PI	LS	MMD/ OMC	CBR AT			SWELL	CLASS	REMARKS
							95%	93%	90%			
TH18	0-0.37	Sand (Fluvial)	0.88	NP	0.0							
	0.37-0.86	Ferricrete										Near refusal at 0.86m
TH19	0-0.34	Silty sand, pebbles (Fluvial)										
	0.34-0.59	Weathered sandstone	2.06	25	11.5	2032/7.3	1	1	1	0.6	N/A	Poorer that G10 quality
	0.59-1.46	Weathered sandstone										Near refusal at 1.46m
TH20	0-0.12	Silty sand (Residual)										
	0.12-0.49	Weathered sandstone										Near refusal at 0.49m
TH21	0-0.48	Silty sand (Fluvial)										
	0.48-0.64	Gravely silty sand (Residual sandstone)	0.90	3.0	1.5							
	0.64-0.94	Weathered sandstone										Near refusal at 0.94m
TH22	0-0.86	Silty sand (Fluvial)	0.73	NP	0.0	1876/14.5	4	3	3	0.0	G10	
	0.86-2.67	Weathered sandstone (Recovered as fine sandy silt)	0.44	20	10.5							Near refusal at 2.67m
TH23	0-0.42	Silty sand (Fluvial)	0.57	NP	0.0							
	0.42-1.22	Weathered sandstone										Near refusal at 1.22m
TH24	0-0.30	Silty sand, pebbles (Fluvial)										
	0.30-0.54	Gravely sand (Residual sandstone)										
	0.54-1.65	Weathered sandstone										Near refusal at 1.65m
TH25	0-0.38	Silty sand (Fluvial)	0.65	NP	0.0							
	0.38-0.77	Weathered sandstone										Near refusal at 0.77m
TH26	0-0.08	Gravely sand (Colluvial)										
	0.08-0.56	Sandy gravel (Residual sandstone)	2.39	11	6.5							
	0.56-1.47	Weathered sandstone										Near refusal at 1.47m

TH	DEPTH (m)	DESCRIPTION	GM	PI	LS	MMD/ OMC	CBR AT			SWELL	CLASS	REMARKS
							95%	93%	90%			
TH27	0-0.41	Silty sand (Colluvial)										
	0.41-0.52	Sandy pebbles, cobbles (Fluvial)										
	0.52-1.45	Weathered sandstone										Near refusal at 1.45m
TH28	0-0.28	Fill (sand, pebbles, bricks)										
	0.28-1.37	Ferricrete (sandy gravel)	2.79	9	5.5							
	1.37-1.87	Weathered sandstone										Near refusal at 1.87m
TH29	0-0.36	Silty sand (Fluvial)										
	0.36-0.91	Silty sand (Fluvial)	0.77	NP	0.0	1776/12.5	15	13	12	0.0	G9	
	0.91-2.03	Weathered sandstone										Near refusal at 2.03m
TH30	1.39	Silty sand (Fluvial)										
	1.39-2.07	Weathered sandstone (recovered as sandy gravel)	2.24	8	4.5	2291/6.0	14	11	9	0.2	G9	Near refusal at 2.07m
TH31												
TH32	0-0.38	Fill (Silty sand, pebbles, bricks)										
	0.38-0.88	Ferricrete (recovered as sandy gravel)	2.41	4	2.0	2312/8.8	76	52	30	0.0	G5	Near refusal at 0.88m
TH33	0-0.33	Silty sand (Fluvial)										
	0.33-0.82	Ferricrete										
TH34	0-0.09	Silty sand (Colluvial)										
	0.09-0.95	Ferricrete (Recovered as slightly sandy gravel)	2.77	2	1.0	2181/4.6	42	33	23	0.0	G6	
	0.95-1.45	Weathered sandstone (recovered as gravely sandy silt)	1.09	12	7.0							Near refusal at 1.45m

TH	DEPTH (m)	DESCRIPTION	GM	PI	LS	MMD/ OMC	CBR AT			SWELL	CLASS	REMARKS
							95%	93%	90%			
TH35	0-0.22	Fill (Sand, pebbles, bricks)										
	0.22-1.06	Sandy gravel (Residual)	2.48	9	5.5	1986/6.0	17	16	15	0.1	G7	
	1.06-3.05	Weathered sandstone										No refusal
TH36	0-0.21	Silty sand. Pebbles (Fluvial)										
	0.21-1.65	Weathered sandstone (Recovered as sandy gravel)	2.33	4	3.0	2021/7.5	42	38	34	0.0	G6	Near refusal at 1.65m
TH37	0-0.42	Silty sand (Fluvial)										
	0.42-0.81	Silty sand (Fluvial)	0.95	NP	0.0	2056/10.6	23	17	12	0.0	G7	
	0.81-1.13	Weathered sandstone										Near refusal at 1.13m
TH38	0-0.12	Silty sand (Colluvial)										
	0.12-0.71	Weathered sandstone										Near refusal at 0.71m
TH39	0-0.07	Silty sand (Colluvium)										
	0.07-0.34	Sandy gravel (Residual sandstone)										
	0.34-0.51	Weathered sandstone (recovered as silty gravel)	1.59	20	10.0							
	0.51-1.39	Weathered sandstone (recovered as silty sandy gravel)	1.89	3	1.5	2091/8.6	28	28	27	0.1	G6	Near refusal at 1.39m
TH40	0-0.50	Silty sand (Fluvial)										
	0.50-0.83	Silty sandy gravel (Residual sandstone)	1.84	15	8.0	2048/11.1	2	2	2	0.8	N/A	Poorer than G10 quality
	0.83-1.17	Weathered sandstone										Near refusal at 1.17m

Where:
GM is the grading modulus
LL is the liquid limit
PI is the plasticity index
NP is non-plastic
SP is slightly plastic
LS is the % linear shrinkage
MDD is the modified AASHTO maximum dry density (kg/m^3)
OMC is the % optimum moisture content
CBR is the California bearing ratio at % of MDD
Class is the material class according to TRH14 material specification

TABLE 7.2: SUMMARY OF DCP TEST RESULTS

DCP NUMBER	TH NUMBER	DEPTH (mm)	COMPUTED IN-SITU CBR	REMARKS
1	1	0-300	110	Refusal at 440mm
2	2	0-300	3	
		300-600	78	
		600-900	54	No refusal
3	3	0-300	6	
		300-600	7	
		600-900	80	Refusal at 930mm
4	4	0-300	63	Refusal at 480mm
5	5	0-300	5	
		300-600	5	Refusal at 680mm
6	6	0-300	5	Refusal at 570mm
7	7	0-300	4	
		300-600	15	
		600-900	56	No refusal
8	8	0-300	-	Refusal at 210mm
9	34	0-300	12	Refusal at 380mm
10	10	0-300	43	
		300-600	36	
		600-900	105	No refusal
11		0-300	16	
		300-600	27	
		600-900	41	No refusal
12	16	0-300	21	
		300-600	70	Refusal at 780mm
13	13	0-300	4	Refusal at 510mm
14	24	0-300	9	Refusal at 340mm
15	25	0-300	43	Refusal at 550mm

Notes:
1) DCP test results affected by cobbles, gravel, boulders within the soil profile
2) Computed in-situ CBR results may over-estimate actual ground conditions

7.1. Subgrade Conditions and Materials Assessment

The investigation indicates a fairly consistent subsurface profile along the major portion of the routes.

The sandy colluvial and fluvial soils were found to classify as G7 to G10 quality material (according to TRH14 material specification). One fluvial gravel sample classified as G4 quality.

The pedogenic ferricrete was found to classify as G5 to G7 quality.

The sandstone rock was noted to be variable in both degree of weathering and composition and this variability is reflected in the material classifications. The weathered sandstone was found to range from G6 to poorer than G10 quality.

Many of the sandy colluvial and fluvial soils were described as having a “very loose” to “loose” consistency. This observation was correlates with many of the DCP tests, which recorded low blow counts at shallow depth.

7.2. Problematic Areas

The investigation indicates that the subgrade conditions are generally adequate to good. However the following potentially problematic geotechnical conditions were encountered during the investigation.

7.2.1. Loose Subgrade Materials

“Very loose” to “loose” sandy soils were found to overly the more dense residual and pedogenic soils and weathered rock at many test positions. The thickness of the loose to very loose soil horizons are, however, limited.

These very loose to loose soils are considered a poor subgrade material and have a low strength in their in-situ state. The laboratory test results indicate that the material strength may be improved with compaction. Heavy compaction, possibly utilizing a vibratory roller, is recommended for the subgrade preparation.

7.2.2. Potentially Expansive Clays

The material recovered from TP22 (0.86-1.83m) was found to be potentially expansive. Possible heave and shrinkage movements may be expected from this material under fluctuating soil moisture conditions.

Previous investigations undertaken by Terratest in this area also indicated the presence of some potentially expansive subgrade material in the area south of the N2 highway.

Where this potentially problematic clayey material occurs at subgrade level below the base of the pavement structure, the material is to be removed to a depth of 300mm below the base of the pavement structure and replaced with granular material of G9 or better quality. Sections requiring this specialised subgrade treatment must be delineated on site during construction.

7.2.3. Shallow Ground Water Conditions

As discussed in Section 5.1, seasonal shallow groundwater conditions are expected in some areas of the site. Groundwater seepage is also expected to be encountered in lower-lying areas adjacent to streams and drainage lines.

The design of drainage measures for the roads must take into account the possible shallow groundwater conditions.

7.3. Excavation Conditions

The vertical alignment of the roads was not provided at the time of the investigation. However, it is understood that deep excavations for cuttings are not required.

Intermediate (to possibly hard excavation in some areas), classified according to COLTO classes of mechanical excavation, may be expected should excavations into the sandstone rock be required.

8. FURTHER INVESTIGATIONS

Detailed geotechnical investigations for the proposed bridge structure were not undertaken during this investigation, owing to the close proximity of hard rock to surface.

It is normally recommended that rotary core drilling be undertaken in order to prove the founding conditions for bridge abutments and piers. This may be considered, to verify the nature of the hard rock below bridge foundation positions and confirm the lack of voids or unseen structures.

9. CONCLUSIONS

This report presents the results of a geotechnical road prism investigation undertaken for the proposed Redhouse - Chelsea Arterial road and extension to Walker Drive in Port Elizabeth.

The investigation indicates that the route is underlain by quartzitic sandstones of the Peninsula Formation of the Table Mountain Group, Cape Supergroup. The bedrock is overlain by colluvial, fluvial and residual soils which were found to be predominantly sandy in composition. Pedogenic ferricrete was encountered in some areas.

The subgrade materials along the major proportion of the route were classified in the range of G7 to G10 (according to TRH14 material specification). Classifications for the various types of materials are provided.

Very loose to loose soils were encountered at shallow depth in many of the trial pits. These soils are considered a poor subgrade material and will have a low strength in their in-situ state. Heavy compaction, possibly utilizing a vibratory roller, is recommended for the subgrade preparation.

Potentially expansive soils were encountered in some areas south of the N2 highway. Where this potentially problematic clayey material occurs at subgrade level undercutting and replacement of this material is recommended.

The design of drainage measures for the roads must take into account the possible shallow groundwater conditions.

Intermediate (to possibly hard excavation in some areas), classified according to COLTO classes of mechanical excavation, may be expected should excavations into the sandstone rock be required.

10. REFERENCES

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APPENDIX A:

DRAWINGS

APPENDIX B:

TRIAL HOLE PROFILES & PHOTOGRAPHS

APPENDIX C:

LABORATORY TEST RESULTS

APPENDIX D:

DCP TEST RESULTS