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GEOTECHNICAL INVESTIGATION
for
Proposed Eskom Sub - Stations at Bophirima & Kalplaats

Mookodi Integration Project, Vryburg

Client: SiVest **Date: October 2012** **Job No:10162/2**

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1 INTRODUCTION

1.1 Preamble

Geopractica was originally appointed by Sivest during April 2010, to carry out a geotechnical scoping study as part of an Environmental Impact Assessment (EIA) on the Mookodi Integration Project at Vryburg, North West Province.

The project involved the linking of the existing Eskom 132 kV distribution power lines from the Vryburg Municipal Sub-Station to the proposed Bophirima Distribution Sub-station (BDss). From BDss, distribution lines will be constructed to a second distribution sub-station at Kalplaats.

On the 15th August 2012, Geopractica was contacted by Mr. S. Taylor of Sivest, requesting Geopractica to carry out detailed geotechnical investigations at the Bophirima and Kalplaats, proposed sub-station sites.

In terms of the requirements of the Department of Environmental Affairs, two alternative sites had to be chosen for each sub-station locality.

On the 23rd August 2012, Geopractica were requested to start with the Geotechnical Investigation and that an Eskom Representative would accompany the investigation team to the proposed sites.

1.2 Database

Geopractica were supplied the following :-

1.2.1 Nine “Google Plans” showing all relevant information related to the proposed route of the electricity supply lines to each of the proposed sub-stations, as well as the location and area of the two alternative sub-station sites at Bophirima.

1.2.2 An “A3 sized” PDF drawing titled “Mookodi Integration Project - Route Overview (EIA Phase)

1.2.3 The centre survey co-ordinates for each of the alternative sites at Bophirima were e-mailed to Geopractica by Mr. Taylor os SiVest.

1.2.4 A survey co-ordinate for the centre of the western most site (Site 1) at Kalplaats, was e-mailed to Geopractica by Hope Masango, a Land Affairs Official employed by Eskom.

Whist visiting the site during the course of the geotechnical investigation, Mr Masango indicated the rough locality of the eastern most site No 2) at Kalplaats. The position of the site was restricted by the required minimum distance between each of the two sites, as well as ensuring that the site did not fall within the road reserve of the gravel road passing between site 1 and site 2.

It was left up to Geopractica to choose the final position of Site 2.

1.3 Objectives

The objectives of the investigation were to:-

- identify the near surface soil profile to a depth of 3.0m using a combination of TLB dug test holes and DPSH probes.

- determine the engineering parameters of the underlying soils

- recommend specific foundations for the proposed structures.

- assess the suitability of the near surface soils for use in general fill and road pavement layers

- comment on any geotechnical problems that may impact upon the construction.

- comment on which of the two alternative sites should be preferred, based on the geotechnical conditions prevalent at each site.

To simplify the reading of the report, it has been divided up into :

Section 2 - 4 : **Bophirima Sites 1 and 2**

Section 5 - 8 : **Kalplaats Sites 1 and 2**

BOPHIRIMA SITES 1 and 2

2 FACTUAL REPORT

2.1 Programme of Work

2.1.1 *Literary Review*

A literary review was conducted in order to obtain data from previous investigations carried out by Geopractica and other consultants in the area. The 1:250 000 geological map, "No 2624 Vryburg " was consulted in order to determine the regional geology in the vicinity of the site. The following Geostrategies Scoping Reports were also consulted:-

Report 10162 dated July 2010 entitled Bophirima to Kalplaats
Report 10162a dated September 2010 Kalplaats to Edward Dam

2.1.2 *Field Work*

On the 5th September 2012, a JCB 3CX TLB excavated five test pits across site 1 and five test pits across site 2. The holes were excavated either to maximum reach of the machine, or refusal.

Each hole was profiled by an engineering geologist according to the Jennings, Brink and Williams system, sampled as necessary and subsequently loosely backfilled.

The survey co-ordinates of the centre point of each site were supplied to us by Sivest.

The positions of the trial holes was set out in the field using a hand held GPS and are shown on the site plan given in Appendix 1, while the detailed trial holes profiles appear in Appendix 3.

2.1.3 *Office and Laboratory Work*

From the soil samples recovered during the fieldwork, the following laboratory tests were conducted:-

CBR tests	4
Foundation Indicators tests	12

The test results are summarized in Tables 4 to 7 below, with the individual test results shown in Appendix 4 of this report.

2.2 Site Description

The two sites investigated were square in shape, each being approximately one hectare in extent.

The sites are situated approximately 7 kms to the south of Vryburg, on undeveloped farm land, covered in dense thorn bush, and indigenous grasses.

Two high voltage Eskom Powerlines are present in relatively close proximity to the site.

The vacant land on which the sites are situated have been fenced off around their perimeter, with access to the site through a farm gate. This sites are accessed parallel to the Eskom Lines, from the point were these lines cross a gravel road to the west.

Both sites slope down to the south east at a gentle gradient of about 2%.

A number of photographs have been taken of the site, and these are presented in Appendix 5.

No underground services were intersected during the geotechnical investigation. The Eskom Powerlines are however lower than normal in this locality, and care will need to be taken should tall construction vehicles and cranes passing under these lines.

2.3 Site Geology

The site is underlain at a shallow depth by basaltic amygdaloidal lava, which belong to the Allanridge Formation, of the Ventersdorp Supergroup.

Typically the upper portion of the bedrock has weathered in situ to form residual soils of variable thickness. Frequent hard rock corestones were present within this horizon.

Recent secondary calcification of certain soil horizons were observed.

These residual soils are in turn overlain by recently transported soils, comprising a thin colluvium of course gravels in a silty sand matrix.

2.4 Hydrology

The mean annual rainfall in this area is between 380mm and 480 mm, most of which occurs as heavy, isolated thunder showers between November and March.

No water seepage was observed in any of the test holes excavated.

The presence of a near surface calcrete horizon suggests that a perched water table may occur during periods of heavy or prolonged rainfall.

Any stormwater runoff is likely to be in the form of sheetwash towards the south on Site 1 and west on Site 2

The depth of the permanent watertable is not known.

2.5 Observations

The geotechnical profile as observed at Site 1 & Site 2 is summarized in Table 1 below.

Table 1 : Summary of soil profile from TP's

Horizon	Ave Thickness (m)	Ave depth to top of horizon (m)	Ave depth to bottom of horizon (m)	Material	Consistency
Bophirima : Site 1					
Topsoil	0.25	0.00	0.25	Silty sand	Loose
Course Colluvium	0.40	0.25	0.55	Silty sand as infilling for abundant coarse gravels and boulders	Medium Dense - Dense
Reworked Residual Basaltic Lava	0.20	0.55	0.75	Fine gravelly clayey silt.	Stiff
Residual Basaltic Lava	1.75	0.75	2.50	Clayey silt with scattered hard corestones	Stiff to very stiff
Bophirima : Site 2					
Topsoil	0.45	0.00	0.45	Silty sand	Loose
Course Colluvium	0.30	0.45	0.75	Silty sand as infilling for abundant coarse gravels and boulders	Medium Dense to Dense
Reworked Residual Basaltic Lava	0.40	0.75	1.10	Fine gravelly clayey silt with abundant calcrete nodules.	Stiff
Residual Basaltic Lava	1.90	1.10	3.00	Clayey silt with scattered hard corestones with abundant calcrete nodules..	Stiff

NOTE * The overall thickness of the residual soil horizons, or the depth to bedrock was not determined.

The Idealised Geological sections contained in Appendix 2, show the various soil horizons present, as well as their thicknesses.

The various soil horisons encountered during the investigation are discussed below:-

2.5.1 *Topsoil*

This horizon comprises a slightly moist, reddish brown, loose, silty sand with scattered gravels and frequent roots.

At Site 1, the topsoil had an average thickness of 250mm, whereas at Site 2, the average thickness increased to 400mm. The topsoil was encountered across both of the sites.

Due the organic content of these soils, it will be necessary to remove them before any construction work begins.

2.5.2 *Coarse Colluvium*

Underlying the topsoil is a silty sand as infilling for abundant sub-rounded coarse gravels and cobbles. The gravels were matrix supported and had a consistency ranging between medium dense to dense.

At Site 1 the average thickness of the colluvium was 400mm, reducing to 300 mm at Site 2. The average depth to the top and bottom of this horizon was 0.25m and 0.4 m respectively.

A laboratory test on these materials indicated that they have a relatively low Plasticity Index of 11%, reducing to 4% for the whole sample. This material classified as a G8 in terms of TRH 14.

Due to the moderate heave potential of the underlying residual soils it is recommended that the coarse colluvium should not be considered suitable as a founding medium for the proposed structures.

2.5.3 *Reworked Residual Basaltic Lava*

These soils are residual in nature derived from the weathering of the original Basaltic Lava bedrock. In this horizon, a reasonable degree of downward mixing has occurred with the upper colluvium, indicated by its gravel content.

2.5.3.1 Site 1

The material was reddish and orange brown in colour, and comprised an intact, gravelly clayey silt with scattered medium to coarse gravels.

At this site, these soils had an average thickness of 0.2m, and were encountered at an average depth of 0.55m below current surface level.

These soils were generally of stiff consistency. No laboratory tests were conducted on these soils, however it is anticipated that they are likely to be of medium plasticity, and “medium” potential activity.

2.5.3.2 Site 2

At Site 2, these soils were similar in colour, consistency, and composition, but contained abundant calcareous nodules. The slight calcification of these soils reduced their overall plasticity and potential activity to "low".

At this site, these soils had an average thickness of 0.45m, and were encountered at an average depth of 0.75m below current surface level.

Due to the limited thickness and inconsistent development of this horizon, it is not considered as a suitable founding medium for any of the structures.

2.5.4 *Residual Basaltic Lava*

These soils were derived from weathering of the original Basaltic Lava bedrock.

2.5.4.1 Site 1

These materials were greenish grey, mottled orange brown in colour, and comprised a highly micro-shattered and slickensided clayey silt.

At this site, these soils had an average thickness of 1.75m. The actual thickness of this horizon will be greater than this, as the maximum excavation depth of the TLB was in the vicinity of 3m. These materials were encountered at an average depth of 0.75m below current surface level.

These soils were generally of stiff consistency.

Laboratory tests on these soils, indicate that they are moderately plastic, with a plasticity index of 22%, and a potential activity of "medium". These results were confirmed by the high degree of micro-shattering and slickensided observed in the trail pits.

Due the medium potential activity of these soils, and the possibility that they could soften up if the become saturated, they are not considered as a suitable medium for any of the structures.

2.5.4.2 Site 2

At Site 2, these soils were similar in colour, but generally dense in consistency. Furthermore, they contained abundant calcareous nodules, and did not exhibit any significant micro-shattering or slickensided. The slight calcification of these soils has reduced their overall plasticity (7 to 12 %) and led to a potential activity of "low".

At this site, these soils had an average thickness of 1.9 m, and were encountered at an average depth of 1.10 m below current surface level.

These soils, due to the degree of calcification, have significantly reduced their plasticity and can be considered as a suitable founding medium for lightly loaded structures. An applied bearing pressure of 175 kPa can be assumed.

2.6 Laboratory Test Results

A summary of the laboratory test results are given in tables 2 and 3 below.

Table 2 : Summary of Foundation Indicators						
TP No.	**Depth(m)**	**Material**	**PI**	**PI(ws)**	**GM**	**Activity**
Bophirima : Site 1						
Bop 1 TP 1	0.20 - 0.70	Silty Sand - Hillwash	12	5	1.89	Low
Bop 1 TP 2	0.85 - 1.30	Shattered Clayey Silt - Residual Lava	22	20	0.40	Medium
Bop 1 TP 3	1.20 - 1.70	Shattered Clayey Silt - Residual Lava	17	9	1.36	Low
Bop 1 TP 3	1.30 - 2.00	Shattered Clayey Silt - Residual Lava	22	14	1.00	Medium
Bop 1 TP 5	1.20 - 1.60	Slightly clayey silt with calcrete nodules - Residual Lava	14	13	0.24	Medium
Bophirima : Site 2						
Bop 2 TP 1	0.90 - 2.00	Silty sand with calcrete nodules - Reworked Residual Lava	15	10	1.07	Low - Medium
Bop 2 TP 1	0.80 - 1.30	Silty sand with calcrete nodules - Reworked Residual Lava	6	5	0.69	Low
Bop 2 TP 2	1.00 - 2.00	Slightly clayey silt with calcrete nodules - Residual Lava	9	6	1.03	Low
Bop 2 TP 3	1.10 - 1.90	Slightly clayey silt with calcrete nodules - Residual Lava	12	8	0.92	Low
Bop 2 TP 4	1.30 - 2.30	Slightly clayey silt with calcrete nodules - Residual Lava	7	6	0.55	Low
Bop 2 TP 5	0.40 - 0.70	Silty sand with gravels and cobbles - Coarse Colluvium	11	4	2.03	Low
Bop 2 TP 5	1.30 - 2.00	Slightly clayey silt with calcrete nodules - Residual Lava	9	7	0.67	Low

Table 3 : Summary of Earthworks Testing, Indicator and CBR tests								
TP No.	**Depth (m)**	**Material**	**PI**	**PI(ws)**	**LS**	**GM**	**CBR @ 93%**	**TRH14 Class**
Bophirima : Site 1								
Bop 1 TP 1	0.20 - 0.70	Silty Sand - Hillwash	12	5	6	1.89	22	G7
Bop 1 TP 3	1.20 - 1.70	Shattered Clayey Silt - Residual Lava	17	9	9	1.36	1	G10
Bophirima : Site 2								
Bop 2 TP 1	0.90 - 2.00	Silty sand with calcrete nodules - Reworked Residual Lava	15	10	7	1.07	11	G8
Bop 2 TP 5	0.40 - 0.70	Silty sand with gravels and cobbles - Coarse Colluvium	11	4	5	2.03	13	G7/G8

2.7 Geotechnical Mapping of Site.

In order to map the geotechnical characteristics of the underlying soil, the residential site class designations method as proposed by Watermeyer and Tromp (1992) and the Joint Structural Division of the SAICE as prescribed by the NHBRC has been applied to this site. Table 4 given below, indicates the various geotechnical characteristics and the criteria used to evaluate the soils.

TABLE 4 : Geotechnical Characteristics

TYPICAL FOUNDING MATERIAL	CHARACTER OF FOUNDING MATERIAL	EXPECTED RANGE OF TOTAL SOIL MOVEMENTS (mm)	ASSUMED DIFFERENTIAL MOVEMENT (% OF TOTAL)	SITE CLASS
Rock (excluding mud rocks which may exhibit swelling to some depth)	STABLE	NEGLIGIBLE	-	R
Fine grained soils with moderate to very high plasticity (clays, silty clays, clayey silts and sandy clays)	EXPANSIVE SOILS	<7,5 7,5-15 15-30 >30	50% 50% 50% 50%	H H1 H2 H3
Silty sands, sands, sandy and gravelly soils	COMPRESSIBLE AND POTENTIALLY COLLAPSIBLE SOILS	<5,0 5,0-10 >10	75% 75% 75%	C C1 C2
Fine grained soils (clayey silts and clayey sands of low plasticity), sands, sandy and gravelly soils	COMPRESSIBLE SOIL	<10 10-20 >20	50% 50% 50%	S S1 S2
Contaminated soils; Controlled fill; Dolomitic areas; Landslip; Land fill; Marshy areas; Mine waste fill; Mining subsidence; Reclaimed areas; Very soft silt/silty clays; Uncontrolled fill	VARIABLE	VARIABLE		P

Based on these parameters, and the soil profile encountered, this site has been classified as:

Class H2/S1

3 INTERPRETIVE REPORT

3.1 Design Parameters

Design parameters for the major soil horizons are given in Table 5 overleaf.

Table 5: Design Parameters		
Material Type	**Allowable Bearing Capacity (kPa)**	**TRH 14 Classification**
Bophirima Site 1		
Hillwash	n/a	G7 Tested
Coarse Colluvium	n/a	G7 / G8 Tested
Reworked Residual Basaltic Lava	n/a	G8 / G9 Assumed
Residual Basaltic Lava	n/a	G10
Bophirima Site 2		
Coarse Colluvium	n/a	G7 / G8 Tested
Reworked Residual Basaltic Lava	n/a	G8 Tested
Residual Basaltic Lava	175 (where stiff or better)	G8 Assumed

Note: n/d Not determined
n/a Not applicable

3.2 Design Solutions

Potential founding solutions for the proposed development are presented below:-

3.2.1 *Structures on Site 1*

The following foundation solutions were considered for the proposed structures depending upon the required founding loading :-

Engineered Soil Mattress
Reinforced Concrete Raft

3.2.1.1 Engineered Soil Mattress

Over an area 1m larger than the foot print of the structure, excavate to the very stiff / very soft rock basaltic lava at depths of between 2.0 to likely 3.5m deep, and then follow the procedure given below:-

- During excavation, all excavated clayey silt, residual basaltic lava should be carted to spoil.

- Once the bulk excavation is completed, the base of the excavation should be ripped and re-compacted to 90% Mod AASHTO.

- As the residual lava subsoils, are considered unsuitable as a backfill material, they will need to be carted to spoil and suitable quality fill of at least G7 quality or better, will need to be imported, for use in the engineered soil mattress.

- The imported G7 quality fill material should be placed in 150mm layers and compacted to 93% Mod AASHTO.

- The two layers which will be immediately below any proposed new foundations are to be compacted to 95% Mod AASHTO, while the balance of the fill required to reach terrace level should be compacted to 93% Mod AASHTO.

- Foundations should then be excavated back into the fill, to the minimum depth compatible with the provision of services into the building (approximately 600mm). An allowable bearing capacity of 150kPa can be assumed for foundations, not exceeding 0.8m wide , placed in the engineered fill.

- Normally loaded floor slabs should be placed on the imported compacted backfill.

3.2.1.2 *Reinforced Concrete Raft*

The reinforced concrete raft should be designed by a competent structural engineer, and all structures will be placed on the raft.

3.2.2 Structures on Site 2

At this site, the slight calcification of the Residual Basaltic Lava has reduced their plasticity, and they were classified as being of low potential activity.

It is recommended therefore that all structures be placed on lightly reinforced conventional strip footing, or on reinforced concrete pad footings.

These footings can be placed at an approximate depth of 1m below current ground level, where an allowable bearing capacity of 175kPa can be assumed.

Before the concrete is placed, the exposed soil surface should be scarified to a depth of 150mm, and then compacted to a density of 93% Mod AASHTO at OMC.

3.2.3 *Internal Roads and Terraces (both sites).*

CBR tests carried out on the Residual Basaltic Lava indicated that they are of poor quality, classifying as being of G8 - G10 according to TRH 14.

These soils are therefore not considered be suitable for use as a general fill for earthwork terraces or in road construction.

A relatively thin layer of average depth of 0.6m of course colluvium comprising a silty sand as infilling for coarse gravels and pebbles, could however in our opinion be used as a general fill. A CBR test carried out n these soils classified them as being of G7/G8 quality. If these materials are however well mixed in with good grade imported materials, the overall classification of these soils should improve to a G7 or better.

The coarse colluvium does however contain a certain percentage of cobbles (60 - 200mm in size) as well as small boulders (> 200mm in size). Before this material is used as a fill, it will therefore need to be passed through a grid of suitable hole size, to remove the oversize material present.

All required materials of a higher quality classification than G7, will need to be imported.

Table 6 below, summarises the material classes as recommended in TRH 14.

Table 6 : Pavement Material Classes

Pavement Layer	TRH 14 Material Class (natural gravels)
Base	G1, G2, G3, G4
Subbase	G5, G6
Selected Layers	G6, G7
Subgrade	G8, G9, G10

3.2.4 *Excavation Classification*

The site excavation characterisation when measured in terms of SABS 1200 D: Earthworks are anticipated to be within the "soft" category to depths of about 3m. The consistency of the materials below this depth were not determined in this investigation.

It is recommended that the sides of the excavations should be sloped to 1:1.5, to ensure sufficient slope stability to prevent collapse of the sidewalls. During periods of heavy rainfall however, the sides of the excavations should be regularly examined, to ensure the safety of construction personnel and equipment.

3.2.5 *Stormwater Management*

No groundwater seepage was observed in any of the trial pits excavated at the site. Should strong periods of rainfall occur during the construction phase, diversion channels could be required to prevent flooding of the excavations.

It is recommended that appropriate damp proofing and ground water control precautions are implemented beneath all the structures and paved areas, as well as on any exposed excavated surfaces on the terraces.

3.2.6 *General*

Prior to construction it is recommended that all trial pits be identified and be properly back filled with suitable material compacted to 93% Mod AASHTO density at +2% to - 1% of optimum moisture content.

3.2.7 *Construction Problems*

It is recommended that all excavations be suitably battered back or temporarily supported, to ensure stable and safe conditions for any personnel entering such excavations.

Precautions should be in place to deal with water seepage into the excavations, should a periods of heavy rainfall occur during the construction phase.

It is possible that large corestones may be intercepted at depths of less 3m below current ground level, blasting or pneumatic equipment maybe required in order to remove these corestones.

3.3 Additional Investigations

No additional investigations are considered necessary.

3.4 Site Selection

In our opinion the most suitable site, from a geotechnical point of view is Site 2. Our site selection criteria has been based upon a conventional foundation system, which should prove to be the most cost effective.

3.5 Environmental Impact (Geotechnical Only) Assessment

Table 7: Environmental Rating		
Nature		
In order to facilitate the construction of the substation, the site will need to "cleared and grubbed" which entails the removal of all the vegetation within the proposed "footprint" of the development (including the access road) in addition approximately 200mm of topsoil will need to be removed, which can be stockpiled for later use in landscaping. **Removal of Vegetation and Topsoil**		
Geographical Extent		
1	Site	The impact will only impact the site
Probability		
4	Definite	The impact will certainly occur (>75% chance of occurrence)
Reversibility		
2	Partially reversible	The impact may be partially reversible depending upon the degree of mitigation measures expected from the contractor
Irreplaceable Loss of resource		
2	Marginal loss of resource	The impact will result in a marginal loss of resource
Duration		
1	Short term	The impact and its effects will with mitigation be negated within 2 years
Cumulative Effect		
2	Low cumulative Impact	The impact will result in insignificant cumulative effects
Intensity/Magnitude		
1	Low	The impact is likely to be barely perceivable to the operation of the substation.
Significance		
1 + 4 + 2 + 2 + 1 + 2 x 1 = 12		
Points	**Impact Significance Rating**	**Description**
6 to 28	Negative Low Impact	The anticipated impact will have a negligible negative effect and requires little or no mitigation

Table 8:- Impact Assessment	
Environmental Parameter	**Vegetation and topsoil**
Nature	**Removal** of vegetation and topsoil from the “footprint” of the substation site including the access road
Extent	**Site.** Will only effect the proposed development
Probability	**100%**. Must take place to facilitate construction
Reversibility	**Partially reversible**. Where necessary important trees, shrubs and other plants could be carefully removed and replanted around the development upon completion of the project. In addition the stockpiled topsoil can be used fro landscaping purposes after construction.
Irreplaceable loss of resource	**Marginal loss of resource**. Some vegetation and topsoil will be lost during construction. However, construction monitoring may reduce this to a minimum
Duration	**Short Term**. The earthworks phase of construction is likely to be <3months. Whilst the rehabilitation around the development should be complete within 2 years of completion of the construction
Cumulative Effect	**Low Cumulative Effect.** With the exception of the new substation plant, the surrounding area should remain unaffected as a result of the construction.
Intensity/Magnitude	**Low.** The impact will have a negligible effect on the component
Significance Rating	**Negative Low Impact.**
Mitigation Measures	The following mitigation measures can be applied:- - Removal of vegetation to be monitored and those species worth saving can be carefully removed and re-plated at a later stage. - The topsoil should be stockpiles and reused for landscaping upon completion of the project. - Earthworks to be undertaken during winter to avoid erosion by rainwater - Terrace slopes to be kept as low as possible to avoid erosion during heavy or prolonged rainfall - The clients representative to monitor the contractor during construction in order to limit addition un-necessary impacts.

4 CONSTRUCTION MONITORING

4.1 Excavation Inspection

It is recommended that all foundation excavations be inspected by a competent person prior to placing any concrete.

4.2 Control Testing

Regular checks on the quality and compaction of the backfill should be made.

KALPLAATS SITES 1 and 2

5 FACTUAL REPORT

5.1 Programme of Work

5.1.1 *Literary Review*

A literary review was conducted in order to obtain data from previous investigations carried out by Geopractica and other consultants in the area. The 1:250 000 geological map, "No 2624 Vryburg " was consulted in order to determine the regional geology in the vicinity of the site.

The following Geostrategies Scoping Reports were also consulted:-

Report 10162 dated July 2010 entitled Bophirima to Kalplaats
Report 10162a dated September 2010 Kalplaats to Edward Dam

5.1.2 *Field Work*

On the 6th September 2012, a JCB 3CX TLB excavated six test pits across Site 1 and five test pits across Site 2. The holes were excavated either to maximum reach of the machine, or refusal.

Each hole was profiled by an engineering geologist according to the Jennings, Brink and Williams system, sampled as necessary and subsequently loosely backfilled.

The Geopractica field staff were accompanied on site by Hope Masango, Eskoms's Land Affairs officer, who supplied the centre co-ordinate of Site 1.

Mr. Masango indicated the approximate extent of the site on the Google earth image. The four corners of the site were set out in the field using a hand held GPS by the Geopractica field geologist. It was agreed on site that this location was acceptable

The positions of the trial holes are shown on the site plan given in Appendix 6, while the detailed trial holes profiles appear in Appendix 8.

5.1.3 *Office and Laboratory Work*

From the soil samples recovered during the fieldwork, the following laboratory tests were conducted:-

CBR tests	5
Foundation Indicators	14
Collapse and double oedometer tests	6

The test results are summarized in Tables 7 to 10 below, with the individual test results shown in Appendix 9 of this report.

5.2 Site Description

The two sites investigated were square in shape, each being approximately one hectare in extent.

The sites are situated approximately 80 kms to the north of Vryburg and are located on the farm Morester. The area comprises undeveloped farm land, covered in indigenous grasses.

The two sites are separated by a gravel road which passes between them. Both sites are fenced off with access through a steel farm gate.

Site 1 is situated approximately 750m to the north west of the main farm buildings, whilst Site 2 is situated approximately 450m to the west of these structures.

A number of photographs have been taken of the site, and these are presented in Appendix 10.

No underground services were intersected during the geotechnical investigation.

5.3 Site Geology

From the available literature as well as the observations during the site investigation, it is evident that the site is underlain by granitic rocks of the Gordonia Formation, Kalahari group.

In the general area the bedrock has weathered to a residual soil, and has been covered in recent times by Aeolian Sand deposits.

5.4 Hydrology

The mean annual rainfall in this area is between 380mm and 480 mm, most of which occurs as heavy, isolated thunder showers between November and March.

No water seepage was observed in any of the boreholes excavated.

Site 1 slopes gently down to the south at an approximate gradient of 2%, whilst Site 2 also slopes down at a gentle gradient of 2%, but to the west.

In periods of very heavy or prolonged rainfall, stormwater in the form of sheetwash is likely to flow from the sites in a southerly and westerly direction respectively.

The depth of the permanent watertable is not known.

5.5 Observations

The geological profile as observed at Site 1 & Site 2 is summarized in Table 7 below.

Table 7 : Summary of soil profile from TP's

Horizon	Ave Thickness (m)	Ave depth to top of horizon (m)	Ave depth to bottom of horizon (m)	Material	Consistency
Kalplaats : Site 1					
Topsoil	0.35	0.00	0.35	Silty sand	Loose
Aeolian Sands	1.40	0.35	1.75	Silty sand	Medium Dense to Dense
Pebble Marker	0.40	1.70	1.95	Silty sand as infilling for abundant sub-rounded coarse gravels	Medium Dense to Dense

Table 7 : Summary of soil profile from TP's					
Horizon	**Ave Thickness (m)**	**Ave depth to top of horizon (m)**	**Ave depth to bottom of horizon (m)**	**Material**	**Consistency**
Reworked Residual Granite	0.35	1.90	2.25	Fine gravelly sand	Medium Dense to Dense
Residual Granite	0.75	2.00	2.80	Fine gravelly sand	Generally dense to very dense
Very dense to very soft rock Granite *	0.65	1.95	2.60	Highly weathered, highly fractured, medium to coarse gravels	Very dense to very soft rock
Kalplaats : Site 2					
Topsoil	0.25	0.00	0.25	Silty sand	Loose
Aeolian Sands	1.75	0.25	2.00	Silty sand	Medium Dense to Dense
Reworked Residual Granite	0.35	2.00	2.35	Silty sand as infilling for abundant sub-rounded coarse gravels	Medium Dense to Dense
Residual Granite	0.75	2.50	3.30	Fine gravelly sand	Medium Dense to Dense
Very dense to very soft rock Granite *	1.20	1.95	3.15	Fine gravelly sand	Generally dense to very dense

NOTE * The overall thickness of the residual soil horizons, or the depth to bedrock was not determined. Intermediate soils classify as very dense to very soft rock.

The various soil horisons encountered during the investigation are discussed below:-

A general assessment of the soil horizons has been summarized below.

5.5.1 Topsoil

A topsoil horizon comprising a reddish brown silty sand with scattered roots covers both sites. The average thickness of this horizon was 0.35m at Site 1, decreasing to 0.25m at Site 2.

Due to the organic content of this horizon, it will need to be removed before any construction commences.

5.5.2 *Aeolian Sands*

Beneath the topsoil is an orange to reddish brown, silty sand, covering both sites. These sands are considered to be Aeolian in nature (ie Wind Transported.)

5.5.2.1 Site 1

At this site these sands have an average thickness of 1.4m and extend to an average depth of 1.75m. They are generally dense in consistency.

Laboratory tests on these soils indicate that they are non plastic in nature, and therefore of very low potential activity. They are reasonably coarse grained with an average grading modulus of 0.78.

Collapse potential tests undertaken on these soils are very variable, which in our opinion is due to a variation of the insitu moisture content.
The lowest collapse settlement was 0.9 % , which according to research by Jennings and Knight indicates that these soils can be regarded as representing “No Trouble”. A further 2 tests elsewhere on the site however gave collapse potentials of 3.3 % and 6.7%, respectively. The former value classifies as “Moderate Trouble”, with latter value indicating “Trouble”. A double oedometer tests gave a collapse value of approximately 5%, which also indicates “Trouble” soils.

Due to the variable and significant collapse potential of these soils in various parts of the site, they are not considered as a suitable founding medium for any of the structures in its present state.

5.5.2.2 Site 2

At this site these sands have an average thickness of 1.75m, and extend to an average depth of 2.0m. They are generally medium dense to dense in consistency.

Laboratory tests on these soils indicate that they are non plastic in nature, and therefore of very low potential activity. They are also relatively coarse with and average grading modulus of 0.74.

Collapse potential tests undertaken on these soils are also highly variable, which in our opinion is due to a variability within the insitu moisture content.

The lowest collapse settlement was 1.6 % , which according to research by Jennings and Knight indicates that these soils can be regarded as representing “Moderate Trouble”. A Double Oedometer Test however gave a collapse settlement of 8.2%, which classifies “Trouble”.

Due to the variable and significant collapse potential of these soils in various parts of the site, they are not considered as a suitable founding medium for any of the structures, in its present state.

5.5.3 *Pebble Marker*

The pebble marker represents the contact between the transported and residual soils. They comprised an orange to reddish brown silty sand as infilling for abundant, sub-rounded, coarse gravels.

5.5.4 *Reworked Residual Granite Horizon*

This horizon represents the upper residual granite soils which have been reworked by the downward movement of the overlying Aeolian sands.

These soils comprised an orange brown mottled grey and occasionally speckled or stained black, occasionally slightly ferruginised gravelly sand with minor ferricrete nodules in places. Their consistency has been described as dense but medium dense around TP1, and contained shallow weathering runnels filled with loose to medium dense silty sand.

These soils were intersected at an average depth of 2.0m below current ground level at both sites 1 and 2.

5.5.5 *Residual Granite Horizon*

These soils resulted from the complete weathering of the original granite bedrock. They comprised greyish to orange brown, fine gravelly sand.

5.5.5.1 Site 1

Residual granite or very dense to very soft rock granite was present as the lowermost horizon across the site.

This horizon was intercepted at an average depth of 2.0m below current ground level and extended depths in excess of 3.0m.

These soils were generally dense in consistency, increasing to very dense with depth.

Laboratory tests on these residual soils indicated them to be non-plastic in nature.

These materials are considered to be suitable as a founding medium for the structures, with a bearing pressure of 175 kPa being applicable, where the consistencies of dense or better are observed.

5.5.5.2 Site 2

Residual granite or very dense to very soft rock granite was present as the lowermost horizon across the site.

Here the residual granite horizon had an average thickness of 0.75 m. The formation exceeds this thickness however, as the TLB had a maximum reach of about 3,0 m.

These soils were intersected at an average depth of approximately 2,4m below current ground level.

In contrast to Site 1, these soils were frequently of loose consistency. Dense material was only observed in Kal 2, TP 2.

Laboratory tests on these residual soils indicated them to be non-plastic in nature.

These materials are not considered to be suitable as a founding medium for the structures, due to their generally very low consistency in the loose range.

5.5.6 *Bedrock Granite*

Very soft rock granite bedrock was encountered in a few trial holes, at refusal depth of the TLB.

5.5.6.1 Site 1

Very soft rock granite was only observed in Kal 1 TP 2 and 3. They had an average horizon thickness of 0.65m, and continued down to TLB refusal depth at approximately 2.5 m.

These materials were intersected at an average depth of approximately 2,0m.

These materials are considered to be suitable as a founding medium for the structures, with a bearing pressure of 250 kPa being applicable, where the consistency is very dense or better.

5.5.6.2 Site 2

Very soft rock granite was only observed in Kal 1 TP1 and TP4. They had an average horizon thickness of 1.2 m, and continued down to TLB refusal depth at approximately 3.0 m.

These materials were intersected at an average depth of approximately 2,0m.

These materials are considered to be suitable as a founding medium for the structures, with a bearing pressure of 250 kPa being applicable, where the consistency is very dense or better.

5.6 Laboratory Test Results

A summary of the laboratory test results are given in tables 8 to 11 overleaf.

Table 8 : Summary of Foundation Indicators

TP No.	Depth(m)	Material	PI	PI(ws)	GM	Activity
Kalplaats : Site 1						
Kal 1 TP 1	0.30 - 1.50	Silty Sand - Aeolian Sands	np	np	0.78	Very low
Kal 1 TP 1	1.00 - 1.20	Silty Sand - Aeolian Sands	np	np	0.78	Very low
Kal 1 TP 3	1.30 - 1.50	Silty Sand - Aeolian Sands	np	np	0.77	Very low
Kal 1 TP 3	2.60 - 2.90	Silty Sand - Aeolian Sands	np	np	0.83	Very low
Kal 1 TP 4	0.9 - 1.1	Silty Sand - Aeolian Sands	np	np	0.77	Very low
Kal 1 TP 5	0.20 - 1.50	Silty Sand - Aeolian Sands	np	np	0.74	Very low
Kal 1 TP 5	0.70 - 0.90	Silty Sand - Aeolian Sands	np	np	0.69	Very low
Kalplaats : Site 2						
Kal 2 TP 1	0.3 - 1.5	Silty Sand - Aeolian Sands	np	np	0.80	Very low
Kal 2 TP 2	1.00 - 1.20	Silty Sand - Aeolian Sands	np	np	0.69	Very low
Kal 2 TP 3	2.25 - 2.40	Silty Sand - Reworked Residual Granite	np	np	0.74	Very low

Table 8 : Summary of Foundation Indicators						
TP No.	**Depth(m)**	**Material**	**PI**	**PI(ws)**	**GM**	**Activity**
Kal 2 TP 3	2.60 - 2.90	Silty Sand - Residual Granite	np	np	0.80	Very low
Kal 2 TP 4	1.1 - 1.3	Silty Sand - Aeolian Sands	np	np	0.75	Very low
Kal 2 TP 4	0.2 - 1.50	Silty Sand - Aeolian Sands	np	np	0.77	Very low
Kal 2 TP 5	2.1 - 2.3	Silty Sand - Aeolian Sands	np	np	0.72	Very low
Kal 2 TP 5	2.6 - 2.8	Silty Sand - Reworked Residual Granite	np	np	0.60	Very low
Kal 2 TP 5	3.1 - 3.2	Silty Sand - Residual Granite	np	np	0.72	Very low

Table 9 : Summary of Earthworks Testing. Indicator and CBR tests								
TP No.	**Depth (m)**	**Material**	**PI**	**PI(ws)**	**LS**	**GM**	**CBR @ 93%**	**TRH14 Class**
Kalplaats : Site 1								
Kal 1 TP 1	0.30 - 1.50	Silty Sand - Aeolian Sands	np	np	0	0.78	19	G7
Kal 1 TP 5	0.20 - 1.50	Silty Sand - Aeolian Sands	np	np	0	0.74	25	G7
Kalplaats : Site 2								
Kal 2 TP 1	0.30 - 1.50	Silty Sand - Aeolian Sands	np	np	0	0.79	31	G7
Kal 2 TP 4	0.20 - 1.50	Silty Sand - Aeolian Sands	np	np	0	0.77	19	G7

Table 10 : Oedometer Collapse Tests						
TP No	**Depth (m)**	**Material**	**Dry Density (kg/m^3)**	**Moisture Content %**	**Initial voids ratio**	**Collapse Potential %**
Kalplaats : Site 1						
Kal 1 TP 3	1.30 - 1.50	Silty Sand - Aeolian Sands	1380	12.2	0.86	0.9
Kal 1 TP 4	0.90 - 1.10	Silty Sand - Aeolian Sands	1389	5.4	0.90	3.3
Kal 1 TP 5	0.70 - 0.90	Silty Sand - Aeolian Sands	1302	5.5	1.02	6.7
Kalplaats : Site 2						
Kal 2 TP 3	2.25 - 2.40	Silty Sand - Reworked Residual Granite	1433	5.2	0.85	4.2
Kal 2 TP 4	1.10 - 1.30	Silty Sand - Aeolian Sands	1312	6.9	1.01	1.6

Table 11 : Double Oedometer Tests						
TP No	Depth (m)	Material	Dry Density (kg/m^3)	Moisture Content %	Initial voids ratio	Collapse Potential %
Kalplaats : Site 1						
Kal 1 TP 1 (NMC)	1.00 - 1.20	Silty Sand - Aeolian Sands	1501	16.98	0.76	5.97
Kal 1 TP 1 (Soaked)	1.00 - 1.20	Silty Sand - Aeolian Sands	1505	18.18	0.76	.
Kalplaats : Site 2						
Kal 2 TP 1 (NMC)	1.00- 1.20	Silty Sand - Aeolian Sands	1501	16.98	0.77	8.20
Kal 2 TP 1 (Soaked)	1.00- 1.20	Silty Sand - Aeolian Sands	1492	16.87	0.78	.

5.7 Geotechnical Mapping of Site.

In order to map the geotechnical characteristics of the underlying soil, the residential site class designations method as proposed by Watermeyer and Tromp (1992) and the Joint Structural Division of the SAICE as prescribed by the NHBRC has been applied to this site. Table 12 given below, indicates the various geotechnical characteristics and the criteria used to evaluate the soils.

TABLE 12 : Geotechnical Characteristics				
TYPICAL FOUNDING MATERIAL	CHARACTER OF FOUNDING MATERIAL	EXPECTED RANGE OF TOTAL SOIL MOVEMENTS (mm)	ASSUMED DIFFERENTIAL MOVEMENT (% OF TOTAL)	SITE CLASS
Rock (excluding mud rocks which may exhibit swelling to some depth)	STABLE	NEGLIGIBLE	-	R
Fine grained soils with moderate to very high plasticity (clays, silty clays, clayey silts and sandy clays)	EXPANSIVE SOILS	<7,5 7,5-15 15-30 >30	50% 50% 50% 50%	H H1 H2 H3
Silty sands, sands, sandy and gravelly soils	COMPRESSIBLE AND POTENTIALLY COLLAPSIBLE SOILS	<5,0 5,0-10 >10	75% 75% 75%	C C1 C2
Fine grained soils (clayey silts and clayey sands of low plasticity), sands, sandy and gravelly soils	COMPRESSIBLE SOIL	<10 10-20 >20	50% 50% 50%	S S1 S2
Contaminated soils; Controlled fill; Dolomitic areas; Landslip; Land fill; Marshy areas; Mine waste fill; Mining subsidence; Reclaimed areas; Very soft silt/silty clays; Uncontrolled fill	VARIABLE	VARIABLE		P

Based on these parameters, and the soil profile encountered, this site has been classified as:

Class C1

6 INTERPRETIVE REPORT

6.1 Design Parameters

Design parameters for the major soil horizons are given in Table 13:-

Table 13: Design Parameters		
Material Type	**Allowable Bearing Capacity (kPa)**	**TRH 14 Classification**
Kalplaats Site 1 & 2		
Aeolian Sands	n/a	G7 Tested
Pebble Marker	n/a	G7 Assumed
Reworked Residual Granite	n/a	n/d
Residual Granite	175 kPa (dense or better)	G7 Assumed
Very dense to very soft rock Granite	250 kPa (very dense or better)	G7 Assumed

Note: n/d Not determined n/a Not applicable

6.2 Design Solutions

Potential founding solutions for proposed development are presented below:-

6.2.1 *Structures*

The following foundation solutions were considered for the proposed structures depending upon the required founding pressures :-

Engineered soil mattress
Deep conventional foundations
Reinforced Concrete Raft
Impact Rolling

Considering the prevailing soil conditions, and the fact that we have been informed that all the structures will be lightly loaded, it is our opinion that either an engineered soil mattress or impact rolling should be considered the most practical founding solutions.

6.2.1.1 Engineered Soil Mattress

Over an area 1m larger than the foot print of the structure, excavate approximately 2.0m deep (Site 1 & 2), and then follow the procedure given below:-

- During excavation, all excavated orange brown, Aeolian silty sand should be stockpiled.

- Once the bulk excavation is completed, it is recommended that it be inspected by a qualified person to ensure that the base of the excavation has terminated in competent material.

- Once approved the base of the excavation should be ripped and re-compacted to 90% Mod AASHTO.

- The stockpiled Aeolian silty sands, which have been classified as being of G7 quality, should then be brought back and placed in 150mm layers and compacted to 93% Mod AASHTO.

- The two layers which will be immediately below any proposed new foundations are to be compacted to 95% Mod AASHTO, while the balance of the fill required to reach terrace level should be compacted to 93% Mod AASHTO.

- Foundations should then be excavated back into the fill, to the minimum depth compatible with the provision of services into the building (approximately 600mm). An allowable bearing capacity 150kPa can be assumed for foundations, not exceeding 2.0m, placed in the engineered fill.

- Normally loaded floor slabs should be placed on the imported compacted backfill.

6.2.1.2 Impact Rolling

Due to the extent of the site and the nature of the transported soils across the site it is recommend that impact rolling be considered.

For this solution to be effective we require significant soil improvement to a depth of at least 2.0m below current ground level. It is recommended that strict control be placed on the contractor during this operation and that Dynamic Probing and plate load testing be undertaken during this operation. An allowable bearing capacity of 150kPa must be attained from this soil improvement method, prior to construction.

6.2.2 *Internal Roads and Terraces.*

CBR tests carried out on the Aeolian Sands covering the site classify it as being of G7 quality according to TRH 14.

These soils are considered to be suitable for use a s a general fill for earthworks terraces as well as use in road construction.

All required materials of a higher quality classification will need to be imported.

Table 14 below, summarises the material classes as recommended in TRH 14.

Table 14 : Pavement Material Classes

Pavement Layer	TRH 14 Material Class (natural gravels)
Base	G1, G2, G3, G4
Subbase	G5, G6
Selected Layers	G6, G7
Subgrade	G8, G9, G10

6.2.3 *Excavation Classification*

The site excavation characterisation when measured in terms of SABS 1200 D: Earthworks are anticipated to be within the "soft" category to depths of between 2.6m and 3.0m. The consistency of the materials below this depth were not determined in this investigation.

It is recommended that the sides of the excavations should be sloped to 1:1.5, to ensure sufficient slope stability to prevent collapse of the sidewalls. During periods of heavy rainfall however, the sides of the excavations should be regularly examined, to ensure the safety of construction personnel.

6.2.4 *Stormwater Management*

No groundwater seepage was observed in any of the trial pits excavated at the site. Should strong periods of rainfall occur during the construction phase, diversion channels could be required to prevent flooding of the excavations.

It is recommended that appropriate damp proofing and ground water control precautions are implemented beneath all the structures and paved areas, as well as on any exposed excavated surfaces on the terraces.

6.2.5 *General*

Prior to construction it is recommended that all trial pits be identified and be properly back filled with suitable material compacted to 93% Mod AASHTO density at +2% to - 1% of optimum moisture content.

6.2.6 *Construction Problems*

It is recommended that all excavations be suitably battered back or temporarily supported, to ensure stable and safe conditions for any personnel entering such excavations.

Precautions should be in place to deal with water seepage into the excavations, should a periods of heavy rainfall occur during the construction phase.

It is possible that large corestones may be intercepted at depths of less 3m below current ground level, blasting or pneumatic equipment maybe required in order to remove these corestones.

6.3 Additional Investigations

No additional investigations are considered necessary.

6.4 Site Selection

In our opinion the most suitable site, from a geotechnical point of view is Site 1. Whilst both sites will require soil improvement techniques, Site 1 has exposed more consistent materials with a greater consistency below the depth of soil improvement.

6.5 Environmental Impact (Geotechnical Only) Assessment

Table 15:- Environmental Rating		
Nature		
In order to facilitate the construction of the substation, the site will need to "cleared and grubbed" which entails the removal of all the vegetation within the proposed "footprint" of the development (including the access road) in addition approximately 200mm of topsoil will need to be removed, which can be stockpiled for later use in landscaping. Additional subsoil will need to be removed below the foundations to a depth of 2.0m. This subsoil has no commercial or engineering value and will need to be carted to spoil **Removal of Vegetation, Topsoil and Subsoil**		
Geographical Extent		
2	Local	The removal of the vegetation and topsoil will only effect the site, whilst the carting of the unusable subsoil will have an effect on a local dump site
Probability		
4	Definite	The impact will certainly occur (>75% chance of occurrence)
Reversibility		
2	Partially reversible	The impact may be partially reversible depending upon the degree of mitigation measures expected from the contractor
Irreplaceable Loss of resource		
2	Marginal loss of resource	The impact will result in a marginal loss of resource
Duration		
1	Short term	The impact and its effects will with mitigation be negated within 2 years
Cumulative Effect		
2	Low cumulative Impact	The impact will result in insignificant cumulative effects
Intensity/Magnitude		
2	Medium	The impact is likely to be barely perceivable to the operation of the substation.
Significance		
2 + 4 + 2 + 2 + 1 + 2 x 2 = 26		
Points	**Impact Significance Rating**	**Description**
6 to 28	Negative Low Impact	The anticipated impact will have a negligible negative effect and requires little or no mitigation

Table 16:- Impact Assessment	
Environmental Parameter	**Vegetation, topsoil and sub soil**
Nature	**Removal** of vegetation and topsoil from the "footprint" of the substation site including the access road. **Dumping. I**n addition the subsoil will need to be removed and carted to a local dump site. Suitable fill will need to be imported from a commercial source within the area
Extent	**Local.** The removal of vegetation and topsoil will only effect the site, whilst the dumping of the sub soil and importation of the engineered fill will effect the local area

Table 16:- Impact Assessment	
Probability	**100%**. Must take place to facilitate construction
Reversibility	**Partially reversible**. Where necessary important trees, shrubs and other plants could be carefully removed and replanted around the development upon completion of the project. In addition the stockpiled topsoil can be used fro landscaping purposes after construction. The sub soil if diligently used in a controlled dump site will benefit the local area.
Irreplaceable loss of resource	**Marginal loss of resource**. Some vegetation and topsoil will be lost during construction. The subsoil will be removed and replaced with imported material. However, construction monitoring may reduce this to a minimum
Duration	**Short Term**. The earthworks phase of construction is likely to be <3months. Whilst the rehabilitation around the development should be complete within 2 years of completion of the construction
Cumulative Effect	**Low Cumulative Effect.** With the exception of the new substation plant, the surrounding area should remain unaffected as a result of the construction.
Intensity/Magnitude	**Low.** The impact will have a negligible effect on the component
Significance Rating	**Negative Low Impact.**
Mitigation Measures	The following mitigation measures can be applied:- - Removal of vegetation to be monitored and those species worth saving can be carefully removed and re-plated at a later stage. - The topsoil should be stockpiles and reused for landscaping upon completion of the project. - The contractor must source a dump site in the local area, which will accept the subsoil, without incurring an additional impact on the dump site. - Similarly the contractor must source engineering fill from a commercial source which does not impact upon the environment - Earthworks to be undertaken during winter to avoid erosion by rainwater - Terrace slopes to be kept as low as possible to avoid erosion during heavy or prolonged rainfall - The clients representative to monitor the contractor during construction in order to limit addition un-necessary impacts.

7 CONSTRUCTION MONITORING

7.1 Excavation Inspection

It is recommended that all foundation excavations be inspected by a competent person prior to placing any concrete.

7.2 Control Testing

Regular checks on the quality and compaction of the backfill should be made.

8 COMPONENT ALTERNATIVE ASSESSMENT

Table 17 summarises the selection process based upon geotechnical constraints only for the powerline routes, with alternative and the substation sites.

Table 17:- Project Alternative Assessment			
Alternative	**Preference**	**Specialist Geotechnical Concerns**	**Fatal Flaws**
Bophirima to Kalplaats corridor (Alt 1)	no preference	Removal of poor quality soil and replacement with engineered fill due to poor foundation conditions. Blasting of rock at surface to install foundations	No
Bophirima to Kalplaats corridor (Alt 2)			
Kalplaats to Edwards Dam (Alt 1)	no preference	Removal of poor quality soil and replacement with engineered fill due to poor foundation conditions.	No
Kalplaats to Edwards Dam (Alt 2)			
Bophirima Substation (Alt 1)	Not Preferred	More costly foundation solution	No
Bophirima Substation (Alt 2)	Preferred	Removal of vegetation and topsoil, which can however be re-used on the project	
Kalplaats Substation (Alt 1)	Preferred	Removal of vegetation, topsoil and subsoil together with the importation of engineered fill from a commercial source	No
Kalplaats Substation (Alt 2)	Not Preferred	More costly foundation solution	

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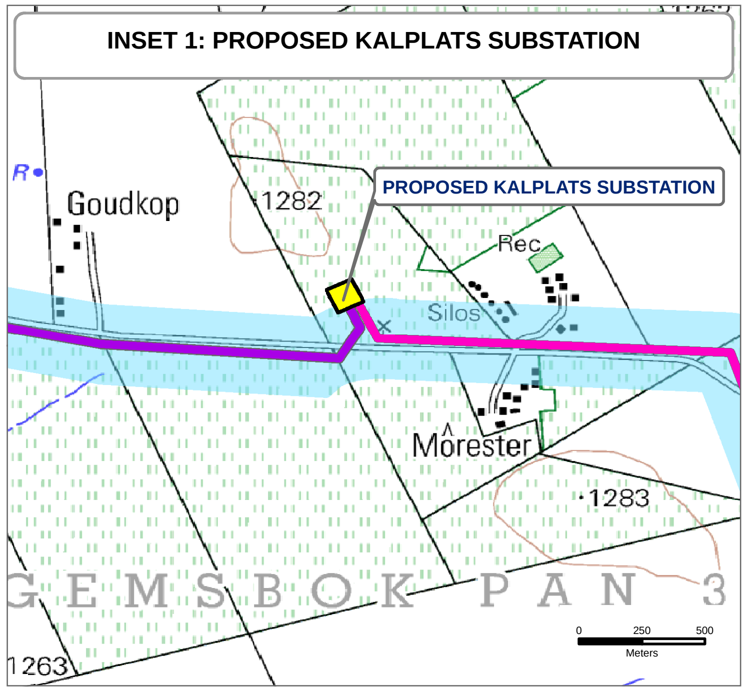
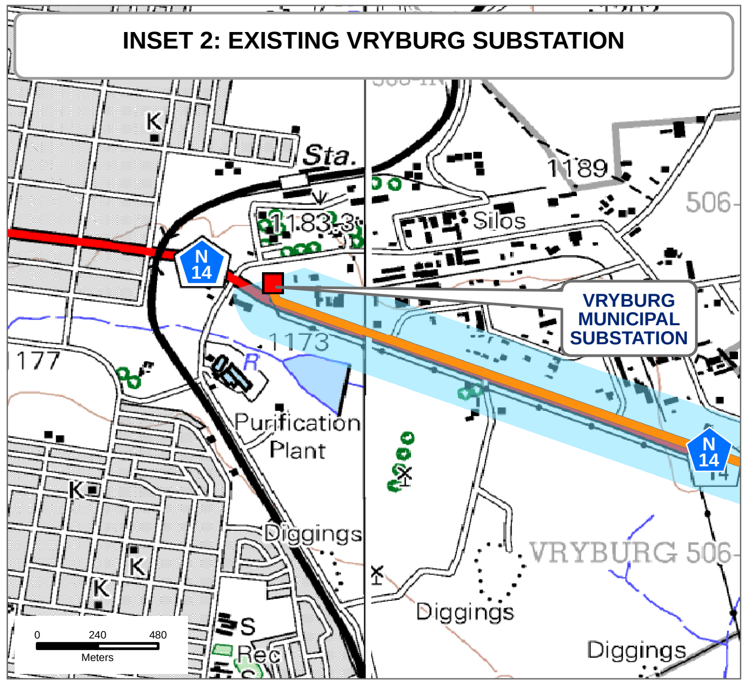
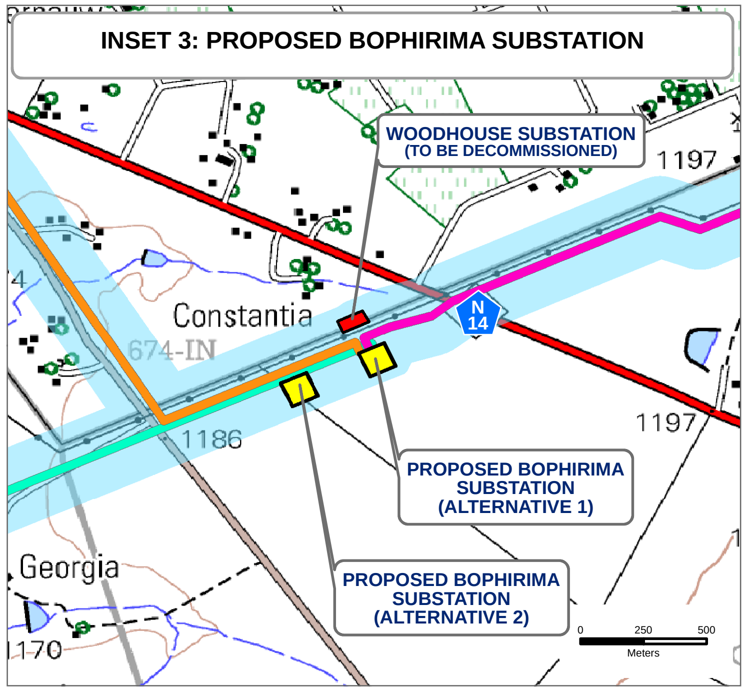
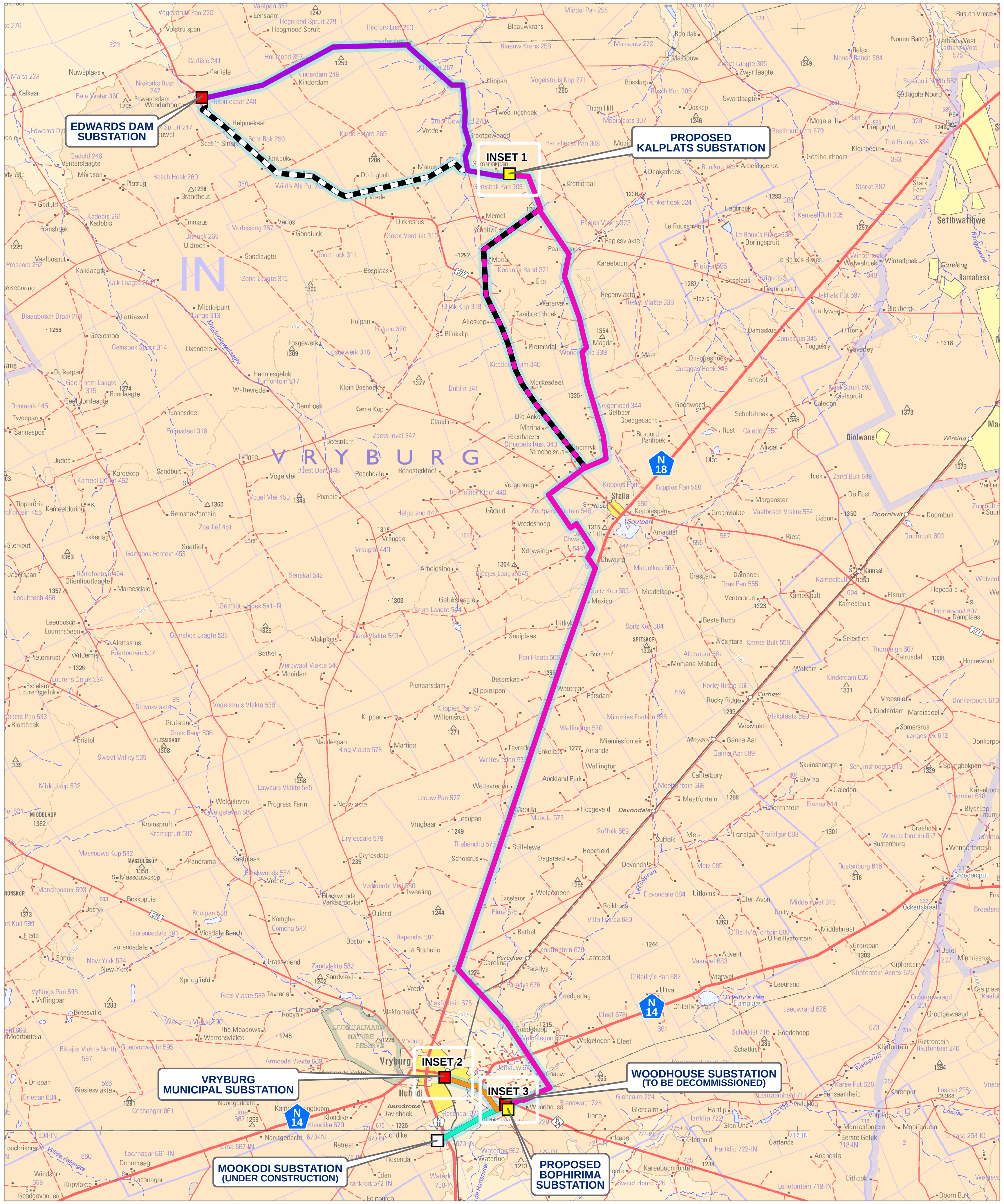
APPENDIX 1
SITE PLAN









MOOKODI INTEGRATION PROJECT

ROUTE OVERVIEW (EIA PHASE)

Legend

- Existing Substations
- Proposed Substations
- Substation Under Construction

Proposed Routes

- Bophirima Substation to Kalplats Substation (Alternative 1)
- Bophirima Substation to Kalplats Substation (Alternative 2)
- Bophirima Substation to Existing Vryburg Substation
- Mookodi Transmission Substation to Bophirima Substation
- Kalplats Substation to Existing Edwards Dam Substation (Alternative 1)
- Kalplats Substation to Existing Edwards Dam Substation (Alternative 3)
- 300m Wide Assessment Corridor

N

0 5 10
Kilometers

SiVEST
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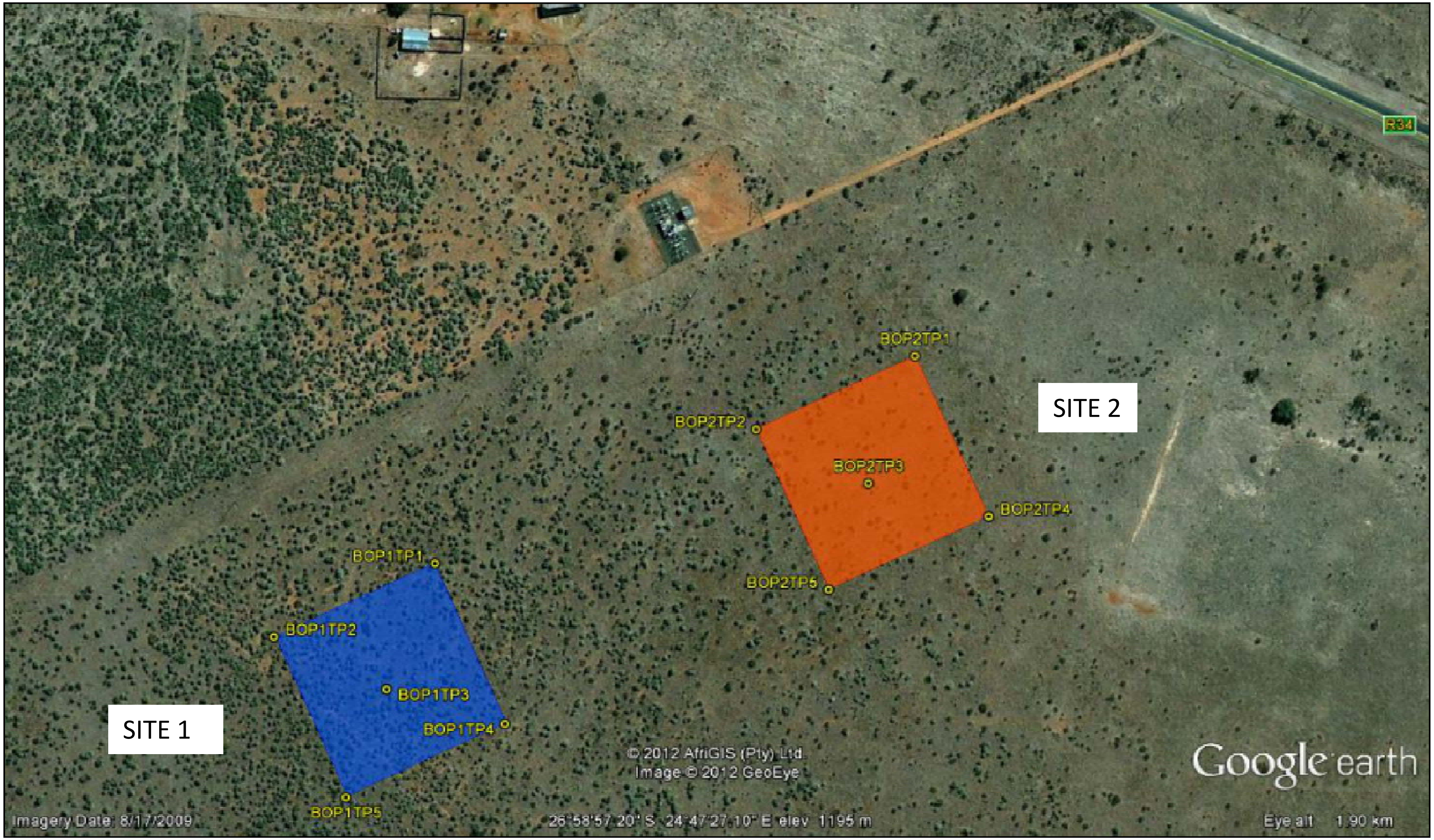
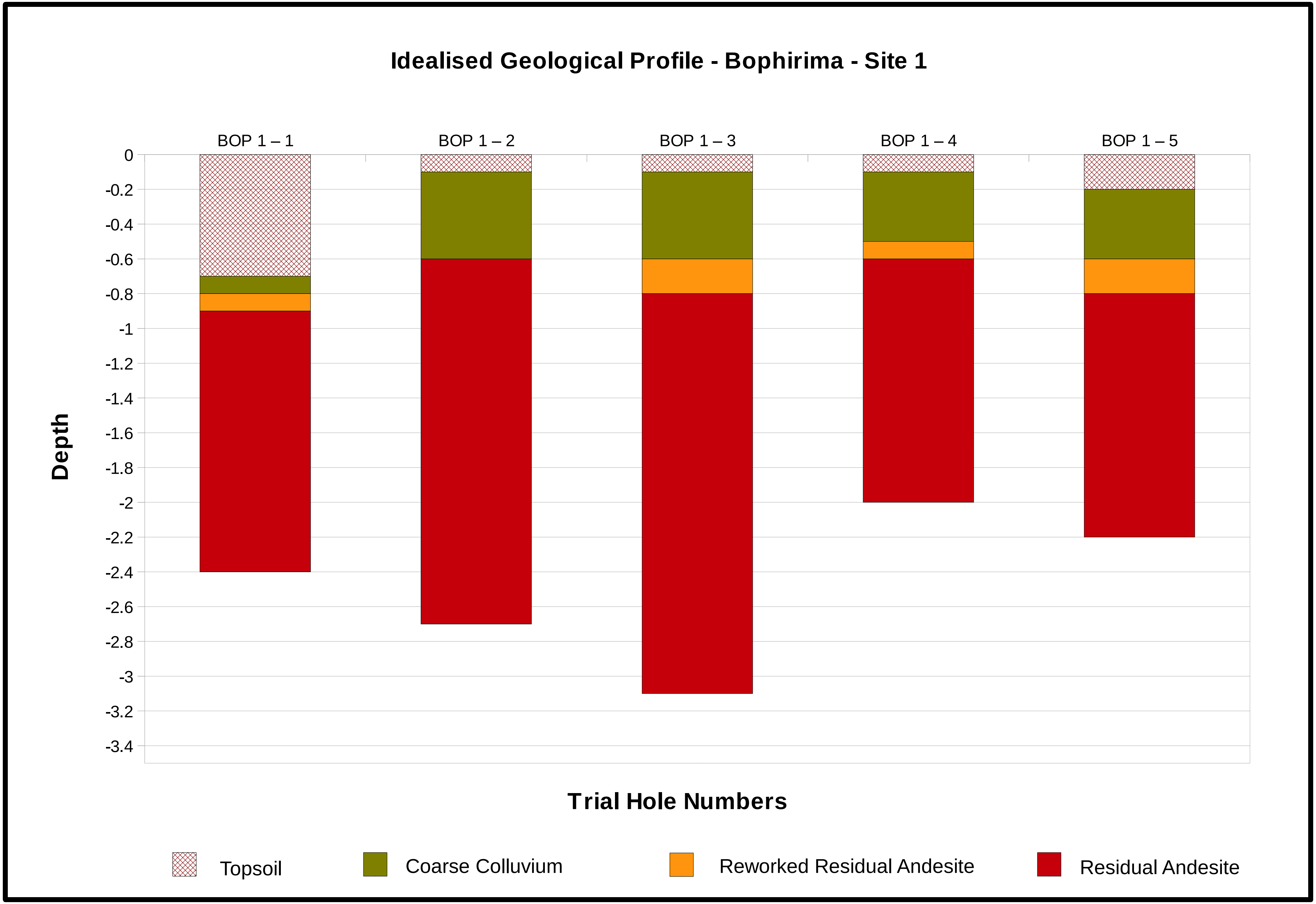
Project No	Map Ref No	Prepared By	Date
10466	10466/EIA_01	KLS	17/08/2012

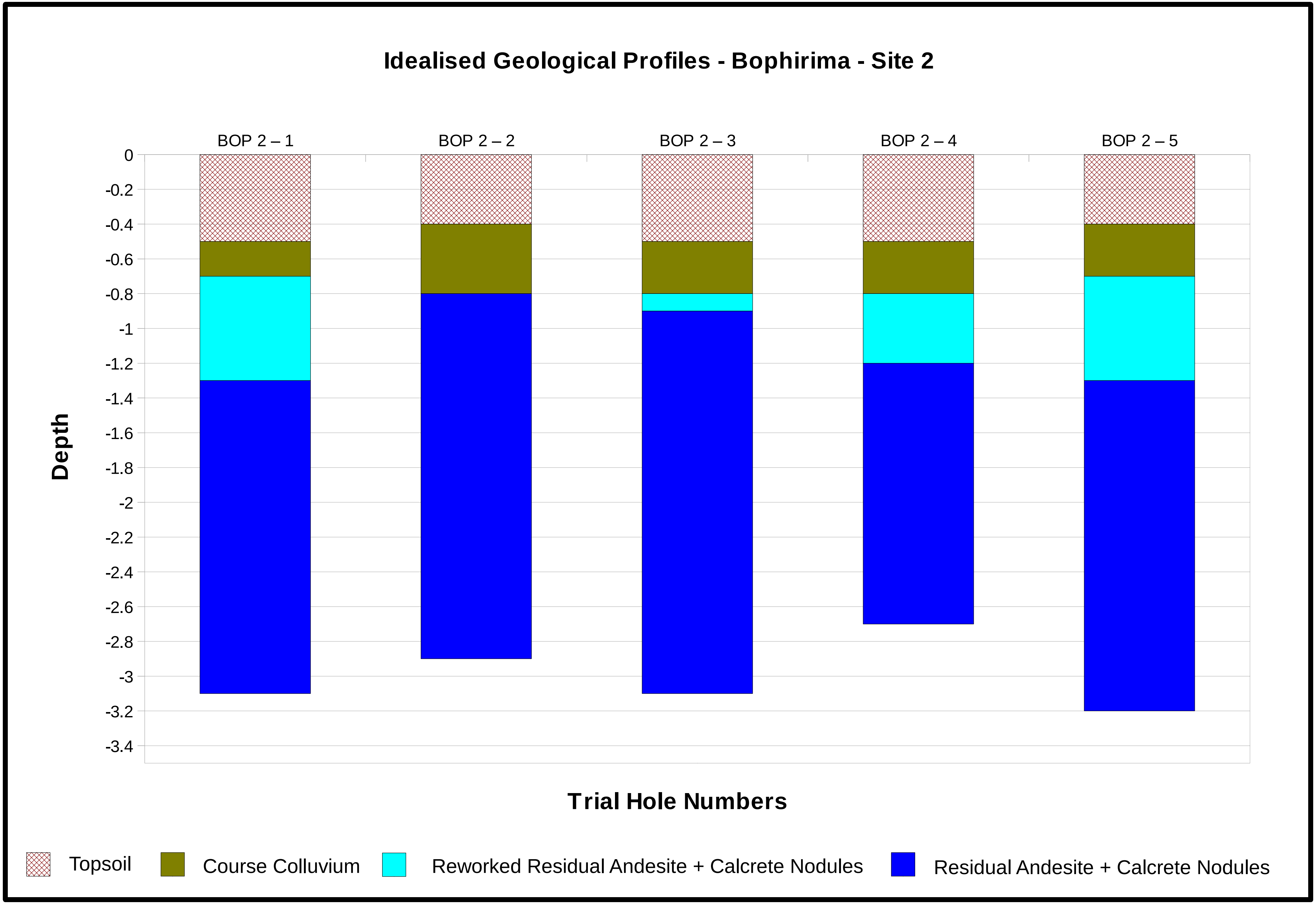


Site Plan – Bophirima Site, Vryburg

APPENDIX 2

IDEALISED GEOLOGICAL PROFILES


Idealised Geological Profile - Bophirima - Site 1
BOP 1 – 1
BOP 1 – 2
BOP 1 – 3
BOP 1 – 4
BOP 1 – 5
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
-1.8
-2
-2.2
-2.4
-2.6
-2.8
-3
-3.2
-3.4
Depth
Trial Hole Numbers
Topsoil
Coarse Colluvium
Reworked Residual Andesite
Residual Andesite



Idealised Geological Profiles - Bophirima - Site 2
BOP 2 – 1
BOP 2 – 2
BOP 2 – 3
BOP 2 – 4
BOP 2 – 5
0
-0.2
-0.4
-0.6
-0.8
-1
-1.2
-1.4
-1.6
-1.8
-2
-2.2
-2.4
-2.6
-2.8
-3
-3.2
-3.4
Depth
Trial Hole Numbers
Topsoil
Course Colluvium
Reworked Residual Andesite + Calcrete Nodules
Residual Andesite + Calcrete Nodules

APPENDIX 3

TRIAL PIT PROFILES

GEOPRACTICA
Consulting Engineers
Tel:011 674 1325 - email: all@geopractica .co.za

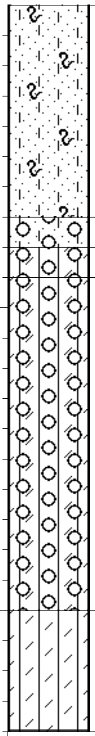
Sivest
Bophirima
Vryburg

HOLE No: BOP1-1
Sheet 1 of 1

JOB NUMBER: 10162/2

Scale
1:25

CBR/FI



0.00 – 0.70	Dry, reddish brown, intact, loose, silty sand with scattered fine to medium roots: Topsoil.
0.70 – 0.80	Dry to slightly moist, silty sand as infilling for abundant sub-rounded coarse gravels and cobbles, overall consistency of dense: Course Colluvium.
0.80 – 0.90	Slightly moist, reddish brown and orange brown, speckled dark red to black, stiff, intact, gravelly clayey silt with minor coarse gravels: Highly Reworked Residual Basaltic Lava.
0.90 – 2.00	Moist, greenish grey, mottled orange brown and speckled dark red, stiff, slickensided and highly micro shattered, clayey silt with scattered coarse gravels: Residual Basaltic Lava.
2.00 – 2.40	Moist, greenish grey, very stiff, intact, clayey silt with scattered hard andesitic corestones approximately 40mm: Residual Basaltic Lava.

NOTES

1) Refusal of TLB @ 2.4m on hard andesite corestone

2) No water seepage observed

3) Disturbed sample CBR/FI 0.2--0.7m

CONTRACTOR : Die Timmerman
MACHINE : JCB
DRILLED BY : David
PROFILED BY : M.C.

TYPE SET BY : Rev 1
SETUP FILE : STANDARD.SET

INCLINATION : Vertical
DIAM :
DATE : 05/09/2012
DATE :

DATE : 08/10/12 09:09
TEXT : ..\2012\10162-~2.TXT

ELEVATION :
X-COORD : 2985796
Y-COORD : 0020923

HOLE No: BOP1-1