GEOTECHNICAL INVESTIGATION FOR THE PROPOSED PHOTOVOLTAIC PLANT NEAR LIBANON GOLD MINE – INTERPRETIVE REPORT

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Geotechnical Investigation for the proposed Photovoltaic Plant near Westonaria – Interpretive Report

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Executive summary

Aurecon was appointed by Sibanye Gold to conduct a feasibility level geotechnical investigation of the proposed Photovoltaic Plant. Typical infrastructure across the site would include photovoltaic panels, substations, and transformers, light buildings (including gate houses, administration buildings, etc) and gravel roads. This investigation was purely for shallow geotechnical foundation recommendation (based only on test pits), the risk of developing on dolomite land is being addressed separately by the client.

The total area of the site is approximately 500 ha and is located near Libanon Gold Mine approximately 8 km east of Carletonville on the R501 road. The topography of the site is relatively flat. The site is used for maize farming. Part of the assessment is to determine whether any geotechnical risks are associated with the sites which could influence the proposed development.

The current investigation commenced with a desk study, which included an appraisal of all previous work undertaken as well as an overview of available maps and databases available. The investigation comprised test pitting, DCP (dynamic cone penetration) testing, resistivity surveys and DPSH (Dynamic Probe Super Heavy). The test pitting exercise was conducted between 7 and 15 December 2015 and comprised the excavation of one hundred and thirteen (113 No) tests pits on a grid of approximately 250m x 250m at the solar farm site using a light tractor-loader-backhoe (TLB).

Based on the test pitting and mapping exercise, the ground profile typically comprises near surface transported sand (typically soft to firm), ferricrete which occurs either as nodular ferricrete, hardpan ferricrete, lenses, or boulders and gravel in a sand matrix. This is underlain by dolomite and chert boulders.

Geological observations indicated a relatively uniform geological profile, which can be summarised as follows:

- Topsoil, comprising sandy silt with roots;
- A sandy silt layer with a red colour and occasionally gravel
- A pedogenic horizon comprising of firm sandy silt with ferricrete nodules, occasionally hardpan ferricrete was intercepted;
- Dolomite residuum comprising, dense to very soft rock sandy gravel with sporadic rounded cobble sized core stones of chert and sandstone in places.

Groundwater or seepage was not encountered within the depth limits afforded by the TLB. However because of the observed ferruginisation, a perched water table within the gravelly sandy residual dolomite horizon cannot be totally ruled out.

A resistivity survey for electrical grounding of equipment was carried out in different types of soft materials at the site. In addition, the results were correlated to the electrical conductivity of the material in order to assess the potential corrosiveness. The results indicated the in situ soils at the site are relatively uniform, with no hard layers.

Selected horizons at the site were identified for laboratory testing, which included foundation indicator, California Bearing Ratio and Mod AASHTO (compaction tests) and chemical tests. The results indicate that the transported material and Karoo clay at the site is classified as G10 material, thus suitable for use as a subgrade material and in the construction of engineered fills of low stiffness. The pedogenic material encountered on site is classified as G7 according to the TRH 14 (CSIR: 1987) guidelines and considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills. The dolomite residuum is classified as G7 and G8. It is considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills (G7) also considered suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills (G8).

The Department of Mineral Resources requirement is that an in-depth, detailed geotechnical risk assessment be carried if the proposed structures are within a horizontal distance of 100 metres from underground workings. The gold mining within the rock of the Witwatersrand Group occurs at depths of more than 800m. This is far beyond the 100 metres stated in the legislation, and these undermined areas therefore do not need to be considered in a detailed geotechnical risk assessment.

Photovoltaic panels can be founded by means of piled foundations. Two alternatives, driven and predrilled grouted piles were investigated. Due to the variable founding conditions on the site, the following is recommended for founding of the PV panels:

- Pre-drilling holes with a percussion drill to a depth of 3 m. The diameter of the holes must be slightly larger than the diameter of the steel pile (i.e approximately 175 mm holes for 150 mm diameter piles)
- Filling the holes with a cement grout.
- Installing the open ended steel tube piles or precast concrete piles to a depth of 3 m. The tube piles can be manufactured to allow easy connection of the PV panels and connecting cabling.

Two substation sites are proposed and both have been evaluated according to their foundation conditions.

For the south eastern site it is recommended that transformers and inverters be founded on soil rafts constructed of imported G7 material. In order to support ancillary components it is recommended that they be founded on a 2.0 m thick engineered soil raft. The transformers shall be founded on a concrete plinth at the surface of the soil raft. The thickness of the imported fill below the proposed transformer foundations shall not be less than 2.0m.

The light buildings should be founded on soil rafts constructed of imported G7 material. It is recommended that they be founded on a 1.5 m thick engineered soil raft. The thickness of the imported fill below the proposed ancillary foundations shall not be less than 1.5 times the proposed foundation width of the structure (e.g.: engineered fill below a 0.6 m wide strip footing will be required to extend to a depth of not less than 0.9 m below the proposed founding level of the strip footing). Consequently, for a structure founded at a depth of 0.5m into the 1.5m thick engineered fill, the foundations shall not exceed a width of 0.6 m.

For a 1.5 m thick engineered fill the foundations will be placed at a depth of not more than 0.5m below the final platform level. The width of such foundations shall not exceed 0.6 m. Should wider foundations be required (assuming greater loads) the thickness of the engineered fill shall be appropriately increased. Foundation loads exerted by the foundations supported on the G7 engineered fill shall not exceed 100 kPa.

Excavations shall be battered at angles no steeper than 45° (from the horizontal). Alternatively excavations shall be shored in the instance where vertical excavations are required.

For the south western site it is recommended that transformers and circuit breakers as described above be founded on soil rafts constructed of imported G7 material. In order to support ancillary components it is recommended that they be founded on an engineered soil raft extending to 1 m depth where stiff sandy silt occurs below 1,0m and to 1,5m depth where stiff sandy silt is encountered below 1,5 m depth.

The light buildings should be founded on soil rafts constructed of imported G7 material. It is recommended that they be founded on an engineered soil raft extending to 1 m depth where stiff sandy silt occurs below 1,0m and to 1,5m depth where stiff sandy silt is encountered below 1,5 m depth. It is recommended that the strip foundations will be placed at a depth of not more than 0.5m below the final platform level. Foundation loads exerted by the foundations supported on the G7 engineered fill shall not exceed 100 kPa.

The following drainage measures shall be implemented in conjunction with a formal drainage design.

- No accumulation of surface water is permitted and the entire development shall be properly drained.
- All trenches and excavation works shall be properly backfilled and compacted in order prevent infiltration of surface run-off into these excavations. Backfilling should be done at optimum moisture content, in layers not exceeding 150 mm to at least 93 % of modified AASHTO density.
- All structures should be provided with a paved apron of 1.0 m wide around the building to prevent water ingress next to foundations and to avoid erosion from surface runoff from roofs. The ground surface shall be suitably sloped to prevent ponding of water in the vicinity of the structures.

All exposed cut slopes and fill batters shall be vegetated to prevent long-term erosion. Stormwater shall be collected in drainage channels and conveyed down slopes in a controlled manner.

1. Introduction

Aurecon was appointed by Sibanye Gold to conduct a feasibility level geotechnical investigation of the proposed Photovoltaic Plant. Typical infrastructure across the site would include photovoltaic panels, substations, transformers, light buildings (including gate houses, administration buildings, etc) and gravel roads.The site is located on Sibanye Gold Mine property at Farm Uitval 280 IQ, approximately 8 km east of Carletonville on the R501 road.

The objective of this investigation is to determine the shallow foundation conditions expected at the solar farm location. This report addresses issues pertaining to founding conditions of materials within the upper 4m of the geological profile (based only on test pits excavated on the site). The risk of developing on land underlain by dolomite is being addressed separately by the client.

The field work was carried out on 7 to 15 December 2015 by a registered engineering geologist who was assisted by a geologist.

The scope of geotechnical investigations is summarised as follows:

- Provide an overview of the geology of the site;
- Present a description of soil and bedrock profiles, including geological maps the soil and rock profiles;
- Identify problem soil areas;
- Determination of surface and ground water regime;
- Assess the engineering properties of the soil and rock ;
- Identify geotechnical considerations that may influence the proposed development; and
- Provide geotechnical-related recommendations for design.

2. Available information

The following published information was used for this investigation:

- The 1:250,000 scale geological map of the West Rand (sheet 2626 West Rand); Council for Geoscience,
- Seismic hazard maps from Kijko *et al*. (2003) - Probabilistic Peak Ground Acceleration and Spectral Seismic Hazard Maps for South Africa. Report number 2003-0053, Council for Geoscience, Pretoria.
- Borehole logs from Sibanye Gold drilled on or near the property, and
- Database from Sibanye Gold on the sinkholes, depressions and cracks in the area.

3. Site location and description

The site is located on Sibanye Gold Mine property at Farm Uitval 280 IQ, approximately 8 km east of Carletonville on the R501 road. (see Figure 2). The total area of the site is approximately 500 ha. The site can be classified as a greenfield site and the present land use for portions of the site is for farming purposes; large portions of the sites are used for cultivation (maize) as seen in below. As can be seen from Figure 1, the entire site is relatively flat.

Figure 1: View of the site with crop cultivation

Figure 2: Plan showing the location of the site

4. Climate

Westonaria normally receives about 559mm of rain per year, with most rainfall occurring during summer. It receives the lowest rainfall (0mm) in June and the highest (104mm) in December. The average midday temperatures for Westonaria range from 16.6°C in June to 26.7°C in January. The region is the coldest during July when the mercury drops to 0.1°C on average during the night.

The Weinert N-value (after Weinert, 1980) in the region is below 5; which generally indicates a humid, warm climate and moist soils in general. These conditions are consistent with chemical decomposition of underlying rock being the dominant mode of weathering, and generally thick transported soil sequences. This correlated well with the site, with soils found to be deeper than 3 m in most cases.

5. Site geology

A review of the 1:250 000 Geological Series Map (2626 West-Rand) indicates that the site is mainly underlain by the Malmani Subgroup (Vmd) of the Chuniespoort Group. The geology of the Malmani Subgroup is described as dolomite, chert, and remnants of chert breccia of the Rooihoogte Formation. There are also small outliers where the geology is that of the Volksrust and Vryheid formations of the Ecca Group (Pe), which is described as shale, sandstone and coal. This small pocket is expected to be underlain at depth by the dolomite and chert etc. of the Malmani Subgroup as can be seen in Figure 3.

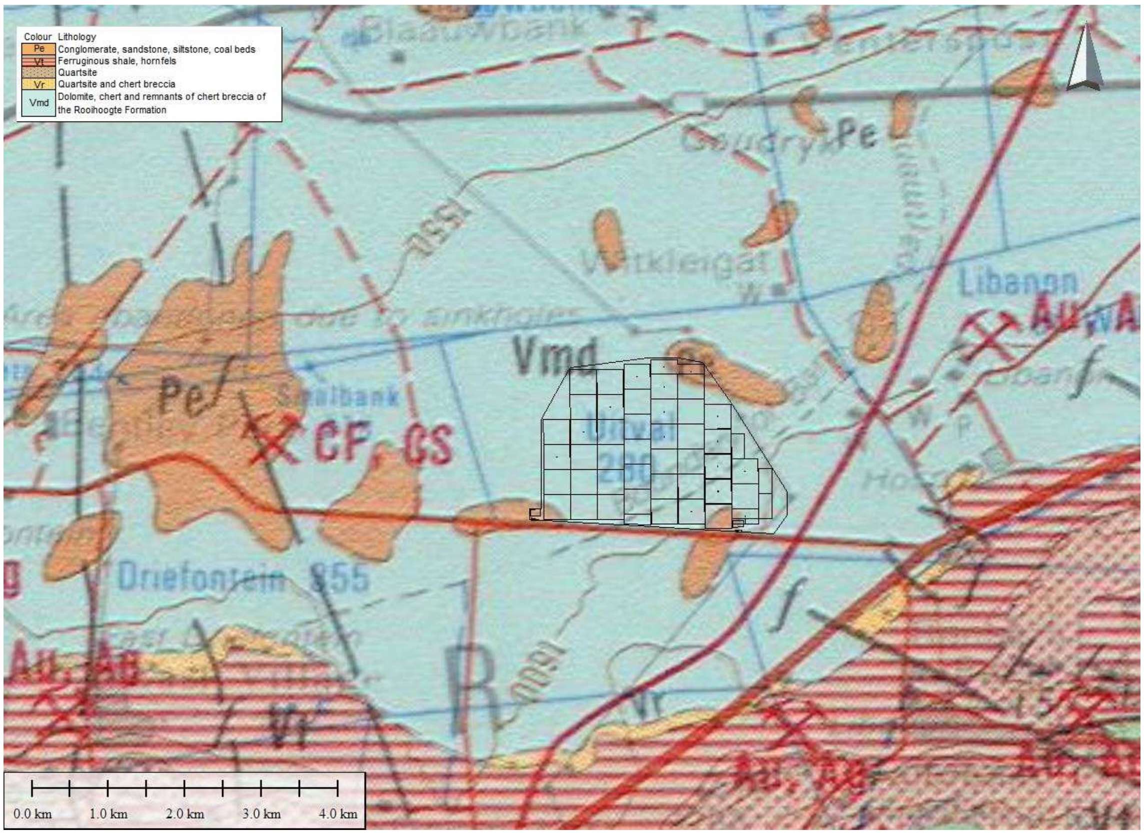

Figure 3: Geology map with the site indicated (1:250,000 scale geological map, sheet 2626 West Rand)

5.1 Conglomerated Siltstone and Sandstone of the Ecca Group

Small outliers of the Ecca Group are present at the site, indicated on the geology map as the brown areas marked (Pe). The remnants of the Ecca Group overlie the dolomite in places and comprise conglomerated siltstone and sandstone deposits that were preserved in depressions in the dolomites, thereby escaping erosion. Many of these depressions are related to palaeo- sinkholes or dolines in the dolomites. At the site the extent of these outliers is not known, but sandstone boulders were found next to certain mealie fields in the southern portion of the site indicating that Ecca Group rocks occurs in the immediate vicinity. A typical profile of the Karoo clays is presented in Figure 4.

Figure 4: Typical profile of the Karoo clays at test pit PV5

5.2 Dolomite and chert of the Malmani Subgroup

With the exception of the small Ecca Group outliers the entire site is underlain by southerly dipping rocks of the Transvaal Supergroup. It is underlain by rocks of the Malmani Subgroup which comprises a thick succession of dolomitic sediments with interbedded chert layers and occasional limestones. At surface the majority of the site is underlain by red brown soils with scattered pieces of chert (Figure 5). Chert cobbles and boulders have been removed from the maize fields and mounds of these rocks are scattered across the area.

From previous borehole information of the area, it is evident that dolomite bedrock occurs from 70 m to 90 m below the surface. It is overlain by chert, clay and wad and some shale.

Figure 5: Typical profile of the dolomite at test pit PV42

6. Seismicity

Seismic hazard is defined in terms of peak ground acceleration on the published seismic hazard map of South Africa (Kijko, *et al.* 2003). Based on the seismic hazard map, the area has a potential of 0.2g peak ground acceleration (

Figure 6), indicating high expected seismic hazard levels. A 10% probability exists that this value will be exceeded in a 50 year period. Most of these earthquakes will be mining induced.

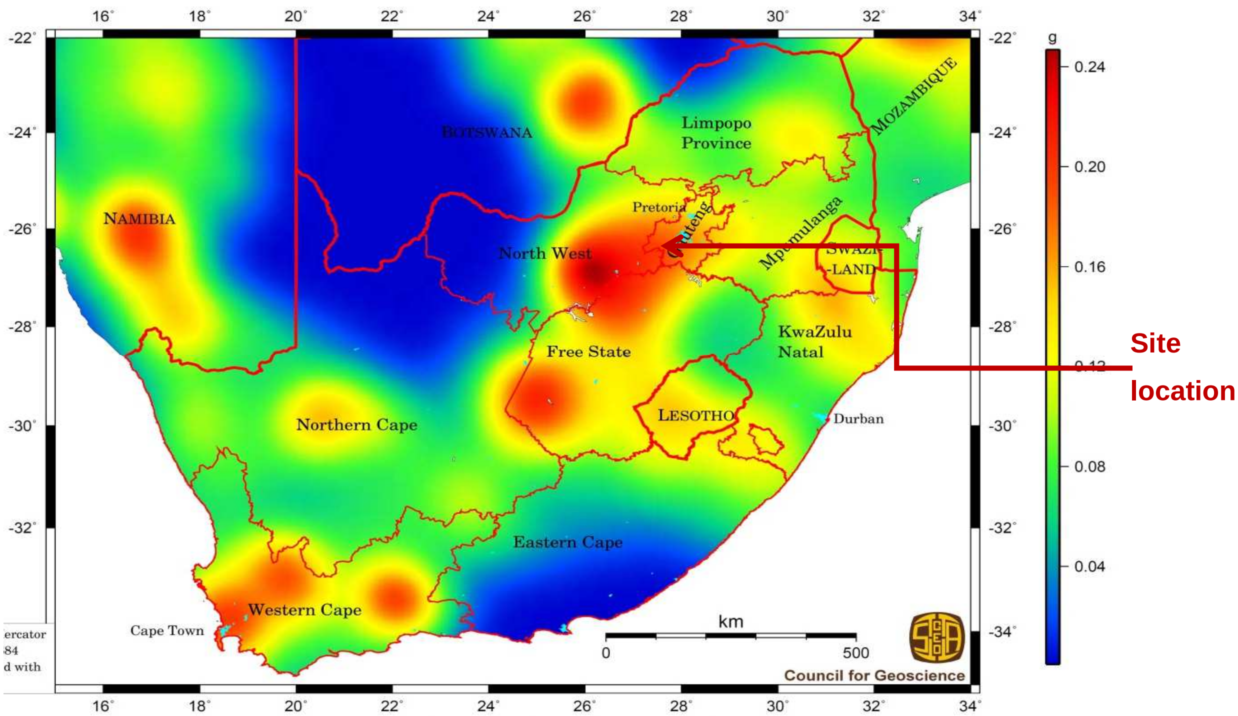

Figure 6: Peak ground acceleration (g) with 10% probability for being exceeded in a 50 year old period (Kijko et al 2003).

7. Site Investigation Methodology

7.1 Desk top study

The desktop study comprised evaluation of geological data obtained from previous investigations, the execution of a gap analysis and tailoring of the field investigation accordingly.

7.2 Test pitting

A total of one hundred and thirteen (113 No) test pits were excavated to refusal or the maximum reach of a tractor loader backhoe (CAT428 E TLB). The investigation was conducted between 7 and 15 December 2015.

The positions of the test pits were set out using a hand-held GPS. All test positions are indicated on a site layout plan (Drawing 112286-G1-001; Appendix F). Coordinates were recorded in WGS 84 format, using the South African grid system (LO27). All test pits were profiled according to South African standards. After completion of the profiling and sampling, all test pits were backfilled, but not compacted.

Table 1 provides a summary of the information obtained from the test pits. Detailed soil profile descriptions are attached in Appendix B.

Table 1: Summary of test pits

Test Pit No	Coordinates (WGS84, LO27)		Excavation Depth (m)	Remarks
	X	Y		
PV 01	58050.76	-2917385	3.00	No Refusal
PV 02	58092.44	-2917406	3.00	No Refusal
PV 03	58057.87	-2917337	3.00	No Refusal
PV 04	58129.55	-2917335	3.00	No Refusal
PV 05	55664.51	-2917227	3.00	No Refusal
PV 06	55703.69	-2917250	3.00	No Refusal
PV 07	55672.02	-2917140	3.00	No Refusal
PV 08	55729.43	-2917179	3.00	No Refusal
PV 09	55826.01	-2917238	3.00	No Refusal

Test Pit No	Coordinates (WGS84, LO27)		Excavation Depth (m)	Remarks
	X	Y		
PV 10	56127.68	-2917247	3.00	No Refusal
PV 12	55817.16	-2916928	3.00	No Refusal
PV 13	56127.69	-2916926	3.00	No Refusal
PV 14	55968.04	-2916762	3.00	No Refusal
PV 15	55814.89	2916601	3.00	No Refusal
PV 16	56127.37	-2916601	3.00	No Refusal
PV 17	55968.53	-2916437	3.00	No Refusal
PV 18	55812.67	-2916276	3.00	No Refusal
PV 19	56127.85	-2916276	3.00	No Refusal
PV 20	55968.45	-2916113	3.00	No Refusal
PV 21	55815.02	-2915953	3.00	No Refusal
PV 21 1	56127.71	-2915950	2.05	Refusal at 2.05 m on chert boulders
PV 22	56129.36	-2915627	3.00	No Refusal
PV 23	56127.19	-2915301	3.00	No Refusal
PV 24	56283.45	-2915463	3.00	No Refusal
PV 25	56283.19	-2915789	3.00	No Refusal
PV 26	26283.23	-2916113	3.00	No Refusal
PV 27	56283.52	-2916438	3.00	No Refusal
PV 28	56283.61	-2916763	3.00	No Refusal
PV 29	56283.78	-2917087	3.00	No Refusal
PV 30	56441.29	-2917247	3.00	No Refusal
PV 31	56440.29	-2916924	3.00	No Refusal
PV 32	56443.28	-2916603	3.00	No Refusal
PV 33	56442.35	-2916276	3.00	No Refusal
PV 34	56442.59	2915952	2.00	Refusal at 2.00 m on hardpan ferricrete
PV 35	56442.28	-2915627	3.00	No Refusal
PV 36	56441.66	-2915306	3.00	No Refusal
PV 37	56598.25	-2915464	3.00	No Refusal

Test Pit No	Coordinates (WGS84, LO27)		Excavation Depth (m)	Remarks
	X	Y		
PV 38	56598.53	-2915788	3.00	No Refusal
PV 39	56597.38	-2916113	3.00	No Refusal
PV 40	56598.41	-2916437	3.00	No Refusal
PV 41	56599.23	-2916763	3.00	No Refusal
PV 42	56597.75	-2917086	1.70	Refusal at 1.70 m on chert boulders
PV 43	56753.59	-2917246	2.70	Refusal at 2.70 m on Karoo Clays
PV 44	56753.21	-2916921	3.00	No Refusal
PV 45	56747.71	-2916592	1.45	Refusal at 1.45 m on hardpan ferricrete
PV 46	56752.81	-2916271	3.00	No Refusal
PV 47	56751.63	-2915949	3.00	No Refusal
PV 48	56749348	-2915622	3.00	No Refusal
PV 49	56745.19	-2915310	3.00	No Refusal
PV 50	56913.82	-2915400	3.00	No Refusal
PV 51	56914.12	-2915723	3.00	No Refusal
PV 52	56913.85	-2916049	3.00	No Refusal
PV 53	56913.35	-2916373	3.00	No Refusal
PV 54	56912.27	-2916698	3.00	No Refusal
PV 55	56913.61	-2917021	2.60	Refusal at 2.60 m on chert boulders
PV 56	56908.15	-2917313	2.60	Refusal at 2.60 m on chert boulders
PV 57	57065.41	-2917181	2.50	Refusal at 2.50 m on Karoo clays
PV 58	57072.08	-2916862	3.00	No Refusal
PV 59	57070.55	-2916539	3.00	No Refusal
PV 60	57065366	-2916212	3.00	No Refusal
PV 61	57068.99	-2915887	3.00	No Refusal
PV 62	57064.02	-2915557	2.35	Refusal at 2.35 m on hardpan ferricrete
PV 63	57069.5	-2915239	1.50	Refusal at 1.50 m on hardpan ferricrete
PV 64	57229.14	-2915269	1.10	Refusal at 1.10 m on hardpan ferricrete
PV65	57228.35	-2915528	1.50	Refusal at 1.50 m on hardpan ferricrete

Test Pit No	Coordinates (WGS84, LO27)		Excavation Depth (m)	Remarks
	X	Y		
PV 66	57229.02	-2915855	3.00	No Refusal
PV 67	57225.93	-2916178	1.50	Refusal at 1.50 m on chert boulders
PV 68	57228.36	-2916502	3.00	No Refusal
PV 69	57227.76	-2916827	3.00	No Refusal
PV 70	57229.45	-2917152	3.00	No Refusal
PV 71	57383.16	-2917312	1.70	Refusal at 1.70 m on hardpan ferricrete
PV 72	57387.78	-2916992	3.00	No Refusal
PV 73	57385.65	-2916667	3.00	No Refusal
PV 74	57388.31	-2916341	1.50	Refusal at 1.50 m on chert boulders
PV 75	57387.33	-2916017	3.00	No Refusal
PV 76	57391.74	-2915693	2.80	Refusal at 2.80 m on hardpan ferricrete
PV 77	57389.29	-2915370	2.20	Refusal at 2.20 m on ferricrete nodules
PV 79	55672.02	-2917140	2.40	Refusal at 2.40 m on chert boulders
PV 80	57542.49	-2915529	2.80	Refusal at 2.80 m on chert boulders
PV 81	57543.66	-2915854	1.10	Refusal at 1.10 m on Karoo clays
PV 82	57544.13	-2916177	2.00	Refusal at 2.00 m on chert boulders
PV 83	57543.2	-2916404	3.00	No Refusal
PV 84	57544.24	-2916831	3.00	No Refusal
PV 85	57545.21	-2917153	3.00	No Refusal
PV 86	57708.14	-2917374	2.40	Refusal at 2.40 m on hardpan ferricrete
PV 87	57703.23	-2917056	3.00	No Refusal
PV 88	57702.88	-2916730	3.00	No Refusal
PV 89	57703.98	-2916407	1.90	Refusal at 1.90 m on chert boulders
PV 90	57703.92	-2916080	3.00	No Refusal
PV 91	57701.58	-2915757	1.40	Refusal at 1.40 m on chert boulders
PV 92	57706.92	-2915373	3.00	No Refusal
PV 93	57858.41	-2915920	2.50	Refusal at 2.50 m on chert boulders
PV 94	57857.71	-2916243	2.50	Refusal at 2.50 m on hardpan ferricrete

Test Pit No	Coordinates (WGS84, LO27)		Excavation Depth (m)	Remarks
	X	Y		
PV 95	57857.33	-2916568	2.40	Refusal at 2.40 m on chert boulders
PV 96	57859.03	-2916892	3.00	No Refusal
PV 97	57857.57	-2917217	3.00	No Refusal
PV 98	58007.82	-2917369	3.00	No Refusal
PV 99	58012.11	-2917054	3.00	No Refusal
PV 100	58016.58	-2916732	2.40	Refusal at 2.40 m on chert boulders
PV 101	58016.22	-2916405	1.60	Refusal at 1.60 m on chert boulders
PV 102	58012.36	-2916082	2.00	Refusal at 2.00 m on chert boulders
PV 103	58014.62	-2915757	2.20	Refusal at 2.20 m on chert boulders
PV 104	58173.13	-2916309	1.70	Refusal at 1.70 m on chert and sandstone boulders
PV 105	58173.19	-2916635	3.00	No Refusal
PV 106	58175	-2916959	2.40	Refusal at 2.40 m on hardpan ferricrete
PV 107	58172.9	-2917219	2.60	Refusal at 2.60 m on chert boulders
PV 108	58331.85	-2917121	1.80	Refusal at 1.80 m on chert boulders
PV 109	58488.87	2917089	1.80	Refusal at 1.80 m on chert boulders
PV 110	58331.89	-2916796	2.60	Refusal at 2.60 m on chert boulders
PV 111	58486.22	-2916765	2.50	Refusal at 2.50 m on chert boulders
PV 112	58480.76	-2917311	1.10	Refusal at 1.10 m on chert boulders
PV 113	58346.33	-2917399	3.00	No Refusal

Note: No groundwater seepage was encountered in any of the test pits

7.3 DCP Testing

The Dynamic Cone Penetration (DCP) test was conducted to determine the soil consistency of the soils. A total of fifty two (52 No) Dynamic Cone Penetrometer (DCP) tests were done adjacent to selected test pits to a maximum depth of 1.0 m below existing ground level. The test results are presented in Appendix D

7.4 DPSH testing

The Dynamic Probe Super Heavy (DPSH) test is conducted by driving a 60° disposable steel cone, 50 mm diameter, into the ground by a 63,5 kg hammer. The hammer is mechanically lifted and dropped a distance of 762 mm and the number of blows required for each successive 300 mm of penetration (referred to as the N-value) is recorded. The N-value is correlated to the consistency of the soil. DPSH testing was conducted on the proposed two substation sites adjacent to the test pits at these proposed substations. The DPSH was carried out adjacent to selected test pits at a total of ten (10) positions to a maximum depth of 3.60m (see Appendix E for details).

7.5 Resistivity testing

The geophysical investigations comprised earth resistivity measurements at the proposed two substation areas. A total of 16 resistivity measurements was taken at each site, which comprised two perpendicular measurements at each of the 8 test locations. The measurement intervals at a test location were increased at predetermined intervals to a maximum spacing of 16m.

8. Results of Investigation

The detailed descriptions of the soil profiles encountered in the test pits are presented in Appendix B; while the geological profiles are summarised below in Table 2, based on the soil profiles intersected in the test pits.

Table 2: Test pit profile summary

Test Pit No	Topsoil / Hillwash (m)	Transported Layer (m)	Pedogenic Layer (m)	Dolomite residuum (m)	Karoo clays (m)
PV 01	0 - 0.70	0.70 – 3.00			
PV 02	0 - 0.65	0.65 – 3.00			
PV 03	0 - 0.90	0.90 – 3.00			
PV 04	0 - 0.80	0.80 – 3.00			
PV 05	0 - 0.20	0.20 – 3.00			
PV 06	0 – 0.40	0.40 – 1.00		1.00 – 3.00	
PV 07	0 - 0.55	0.55 – 1.50		1.50 – 3.00	
PV 08	0 - 0.30	0.30 – 0.80		0.80 – 3.00	
PV 09	0 - 0.80			0.80 – 3.00	
PV 10	0 - 0.65		0.65 – 3.00		
PV 12	0 - 0.30		1.10 – 3.00	0.30 – 1.10	
PV 13	0 - 0.50	0.50 – 1.90	1.90 – 3.00		
PV14	0 – 0.40		0.70 – 3.00	0.40 – 0.70	
PV 15	0 - 0.40	0.40 – 0.90	0.90 – 3.00		
PV 16	0 – 0.50		0.50 – 1.10	1.10 – 3.00	
PV 17	0 - 0.20		0.20 – 2.20	2.20 – 3.00	
PV 18	0 - 0.50		0.50 – 1.00	1.00 – 3.00	
PV 19	0 - 0.20	0.20 – 0.55	0.55 – 1.20	1.20 – 3.00	
PV 20	0 - 0.30	0.30 – 0.60	0.60 – 1.20	1.20 – 3.00	
PV 21	0 - 0.30	0.30 – 1.30	1.30 – 3.00		
PV 21 1	0 – 0.35	0.35 – 1.20		1.20 – 2.05	
PV 22	0 - 0.30	0.30 – 2.30	2.30 – 3.00		

Test Pit No	Topsoil / Hillwash (m)	Transported Layer (m)	Pedogenic Layer (m)	Dolomite residuum (m)	Karoo clays (m)
PV 23	0 - 0.20	0.20 – 0.95	0.95 – 3.00		
PV 24	0 - 0.40	0.40 – 2.00	2.00 – 3.00		
PV 25	0 - 0.40	0.40 – 2.40	2.40 – 3.00		
PV 26	0 - 0.40	0.40 – 1.10	1.10 – 3.00		
PV 27	0 - 0.30		0.30 – 0.70	0.70 – 3.00	
PV 28	0 - 0.20	0.20 – 1.10	1.10 – 3.00		
PV 29	0 – 0.40	0.40 – 2.05	2.05 – 3.00		
PV 30	0 - 0.50	0.50 – 2.10	2.10 – 3.00		
PV 31	0 - 0.40	0.40 – 0.80	0.80 – 1.50		1.50 – 3.00
PV 32	0 - 0.30		0.30 – 0.50	0.50 – 3.00	
PV 33	0 - 0.25	0.25 – 1.05	1.05 – 3.00		
PV 34	0 - 0.55	0.55 – 1.90	1.90 – 2.00		
PV 35	0 - 0.40	0.40 – 2.50	2.50 – 3.00		
PV36	0 – 0.20	0.20 – 1.20	1.20 – 3.00		
PV 37	0 - 0.40	0.40 – 2.40	2.40 – 3.00		
PV 38	0 – 0.30	0.30 – 2.10	2.10 – 3.00		
PV 39	0 - 0.30	0.30 – 2.30	2.30 – 3.00		
PV 40	0 - 0.40		0.40 – 1.20	1.20 – 3.00	
PV 41	0 - 0.30	0.30 – 0.80	0.80 – 2.00		2.00 – 3.00
PV 42	0 - 0.35	0.35 – 1.70			
PV 43	0 - 0.45		0.45 – 1.70		1.70 – 2.70
PV 44	0 - 0.60	0.60 – 1.30	1.30 – 3.00		
PV 45	0 - 0.35		0.70 – 1.45	0.35 – 0.70	
PV 46	0 - 0.80	0.80 – 2.50	2.50 – 3.00		
PV 47	0 - 0.40	0.40 – 1.50	1.50 – 3.00		
PV 48	0 – 0.35	0.35 – 3.00			
PV 49	0 - 0.20	0.20 – 1.30	1.30 – 3.00		
PV 50	0 - 0.25	0.25 – 2.00	2.00 – 3.00		

Test Pit No	Topsoil / Hillwash (m)	Transported Layer (m)	Pedogenic Layer (m)	Dolomite residuum (m)	Karoo clays (m)
PV 51	0 - 0.60			0.60 – 3.00	
PV 52	0 - 0.35	0.35 – 2.20		2.20 – 3.00	
PV 53	0 - 0.30	0.30 – 2.80	2.80 – 3.00		
PV 54	0 - 0.40	0.40 – 2.40		2.40 – 3.00	
PV55	0 – 0.40	0.40 – 2.60			
PV 56	0 - 0.70	0.70 – 2.60			
PV 57	0 - 0.40			0.40 – 1.70	1.70 – 2.50
PV 58	0 – 0.20	0.20 – 2.30	2.30 – 3.00		
PV 59	0 - 0.20	0.20 – 2.70	2.70 – 3.00		
PV 60	0 - 0.35	0.35 – 2.50		2.50 – 3.00	
PV 61	0 - 0.50			0.50 – 3.00	
PV 62	0 – 0.25	0.25 – 1.35	1.35 – 2.35		
PV 63	0 - 0.40	0.40 – 1.20	1.20 – 1.50		
PV 64	0 - 0.25	0.25 – 0.90	0.90 – 1.10		
PV 65	0 - 0.25	0.25 – 0.95	0.95 – 1.50		
PV 66	0 - 0.40	0.40 – 1.00	1.00 – 3.00		
PV 67	0 - 0.40	0.40 – 0.70			0.70 – 1.50
PV 68	0 - 0.50	0.50 – 3.00			
PV 69	0 - 0.35	0.35 – 3.00			
PV70	0 – 0.30	0.30 – 1.70		1.70 – 3.00	
PV 71	0 - 0.30	0.30 – 1.00	1.00 – 1.70		
PV 72	0 - 0.30	0.30 – 2.05	2.05 – 3.00		
PV 73	0 - 0.40		0.40 – 3.00		
PV 74	0 - 0.40	0.40 – 1.50			
PV 75	0 - 0.30	0.30 – 1.80		1.80 – 3.00	
PV 76	0 - 0.30		0.30 – 2.80		
PV 77	0 – 0.40	0.40 – 1.20	1.20 – 2.20		
PV 79	0 - 0.45		0.45 – 1.00	1.00 – 2.40	

Test Pit No	Topsoil / Hillwash (m)	Transported Layer (m)	Pedogenic Layer (m)	Dolomite residuum (m)	Karoo clays (m)
PV 80	0 - 0.55			0.55 – 2.80	
PV 81	0 - 0.45	0.45 – 0.80			0.80 – 1.10
PV 82	0 - 0.30	0.30 – 1.20		1.20 – 2.00	
PV 83	0 - 0.50	0.50 – 3.00			
PV84	0 – 0.30	0.30 – 3.00			
PV 85	0 - 0.60	0.60 – 3.00			
PV 86	0 - 0.70	0.70 – 2.20		2.20 – 2.40	
PV 87	0 - 0.40	0.40 – 3.00			
PV 88	0 - 0.45	0.45 – 3.00			
PV 89	0 - 0.30	0.30 – 1.30	1.30 – 1.90		
PV 90	0 - 0.10	0.10 – 1.60	1.60 – 3.00		
PV 91	0 – 0.40				0.40 – 1.40
PV 92	0 - 0.35			0.35 – 3.00	
PV 93	0 - 0.45				0.45 – 2.50
PV 94	0 - 0.30	0.30 – 1.00	1.00 – 2.50		
PV 95	0 - 0.40			0.40 - 2.40	
PV 96	0 - 0.60	0.60 – 3.00			
PV 97	0 - 0.80	0.80 – 3.00			
PV98	0 – 0.65	0.65 – 3.00			
PV 99	0 - 0.60	0.60 – 3.00			
PV 100	0 - 0.60			0.60 – 2.40	
PV 101	0 - 0.50			0.50 – 1.60	
PV 102	0 - 0.30	0.30 – 1.40		1.40 – 2.00	
PV 103	0 - 0.55	0.55 – 1.20		1.20 – 2.20	
PV 104	0 - 0.40	0.40 – 0.50	0.50 – 0.80	0.80 – 1.70	
PV 105	0 - 0.30		0.30 – 3.00		
PV106	0 – 0.40	0.40 – 1.80	1.80 – 2.40		
PV 107	0 - 0.20	0.20 – 1.80			1.80 – 2.60

Test Pit No	Topsoil / Hillwash (m)	Transported Layer (m)	Pedogenic Layer (m)	Dolomite residuum (m)	Karoo clays (m)
PV 108	0 - 0.45	0.45 – 1.20			1.20 – 1.80
PV 109	0 – 1.05				1.05 – 1.80
PV 110	0 - 0.25	0.25 – 0.70			0.70 – 2.60
PV 111	0 - 0.25		0.25 – 1.45		1.45 – 2.50
PV 112	0 - 0.30	0.30 – 1.10			
PV 113	0 - 0.60	0.60 – 3.00			

The geotechnical investigation revealed that the profile across the site comprises the following horizons:

- Top soil;
- Transported;
- Pedogenic horizon;
- Dolomite residuum; and
- Karoo clays.

It should be noted that the dolomite residuum was generally absent in test pits where Karoo clays (Ecca Group materials) were encountered.

8.1 Topsoil

This horizon occurs across the entire site and comprises very soft to soft slightly moist, brown, clayey sandy silt containing plant roots. From ground level it and has an average thickness of about 0,40m.

8.2 Transported horizon

The transported layer underlies the topsoil and is up to 2,80m thick with an average thickness of 1,50m. The composition of the layer is generally orange to brownish red, soft to firm, intact, clayey sandy silt. It was profiled as having a consistency ranging from very soft to firm.

8.3 Pedogenic horizon

The pedogenic layer is not present everywhere but it generally underlies the transported horizon (seeTable 2). It varies in thickness between 0.1 m and 2.7 m (average 1.1 m) and in places extends to depths of more than 3 m below the present surface level. This pedogenic layer generally has three horizons.

The first horizon occurs as slightly moist, red brown orange and mottled black, stiff, intact, ferruginised gravel in a sandy silt matrix.

The second pedogenic horizon occurs as slightly moist, reddish brown mottled black, dense, intact, sub-rounded ferricrete nodules in a sandy silt matrix. This layer was encountered in twelve test pits (PV32, PV34, PV38, PV40, PV45, PV46, PV47, PV59, PV69, PV71, PV72, PV79) at the site.

Occasionally a third horizon is encountered, which is reddish brown, very soft rock, hardpan ferricrete, it was encountered in five test pits (PV31, PV41, PV44, PV76, PV94) at the site.

8.4 Dolomite residuum

This layer generally underlies the pedogenic layer and comprises yellowish orange brownish red and black, reddish brown, stiff to very stiff, sandy clay with chert gravels and boulders and it grades to chert and dolomite bedrock with increasing depth. In places cobble sized chert occurs within the layer. It varies in thickness between 0.2 m and 2.65 m (average 1.5 m) and extends to depths in excess of 3 m below the present surface level. It was profiled as having a consistency ranging from firm to very stiff.

8.5 Karoo clays.

These clays occur as small patches throughout the site and not only in areas shown on the geological map. They occur as cream to off-white stiff to very stiff, kaolinite clayey silt. This layer is mined in a nearby quarry close to the site for brickmaking. It varies in thickness between 0.3 m and 2.05 m (average 1.0 m) and extends to depths of 3 m below the present surface level. This layer was profiled as having a very stiff consistency.

8.6 Moisture conditions

Groundwater or seepage was not encountered in any of the test pits excavated at the site. The presence of ferricrete does however indicate that temporary shallow water tables has occurred during the geological history of the site. Shallow perched water tables can therefore be expected in the ground profile after extended rainy periods.

9. Field test results

9.1 DCP Testing

The Dynamic Cone Penetration (DCP) test is conducted by driving a 20 mm diameter, 60° cone into the ground using 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 3 gives a guideline of the correlation between the soil consistency for cohesive soils and the DCP testing.

Table 3: Guideline for probing test relation to soil consistency in clayey materials

Description	Cohesion (kPa)	Dropweight cone penetrometer (DCP) (mm per blow)
Very Soft	<18	>110
Soft	18 – 36	55 – 110
Firm	36 – 72	30 – 55
Stiff	72 – 144	15 – 30
Very Stiff	>144	7 – 15

A total of fifty two (52 No) Dynamic Cone Penetrometer (DCP) tests were conducted adjacent to respective test pit to a maximum depth of 1.0 m below existing ground level to determine the consistency of in-situ materials. A summary of the results appears in Table 4 below, while detailed DCP test results are appended as Appendix D.

Table 4: DCP Counts and Soil Types

Soil Origin	Average Depth (mm) Base of Horizon	Average DCP Counts (mm/blow)	Consistency
Top Soil	300	50 to 60	soft to firm
Top Soil	500	10 to 20	stiff to very stiff
Transported	1000	16 to 50	stiff to very stiff ,occasionally firm
Pedogenic	1000	2 to 36	very stiff to refusal, occasionally stiff
Dolomite Residuum	1000	4 to 10	very stiff to refusal and one firm
Karoo Clays	1000	0	refusal

The DCP results are consistent with the logging carried out at the site, the transported materials at the top is soft to firm. The pedogenic layers according to the DCP results are very stiff to refusal, this is similar to the consistency obtained at site during logging, where refusal of the TLB was encountered on the pedogenic material. Dolomite residuum is very stiff to refusal according to the average DCP results, this again is reflection of the chert boulders which were encountered in this layer.

9.2 DPSH testing

The N-value obtained from the DPSH test can be correlated to the consistency of the soil (see Table 5).

The test pits at the two proposed substation positions have soil profiles up to 3,0m. The DPSH was carried out adjacent to ten (10 No.) test pits. The results from the testing are summarised in Table 6 and included in Appendix E.

Table 5: Relationship of Consistency and N-Value (after Franki, 1995)

Cohesive materials		Non-cohesive materials	
Consistency	**Approximate N-value**	**Consistency**	**Approximate N-value**
Very soft	<2	Very loose	<5
Soft	2 – 4	Loose	5 – 10
Firm	4 – 8	Medium dense	10 – 30
Stiff	8 – 15	Dense	30 – 50
Very stiff	15 - 50	Very dense	>50

Table 6: DPSH test results

Consistency according to the N-value					
Test pit No.	**Loose / soft (m)**	**Medium dense / firm (m)**	**Dense / stiff (m)**	**Very dense / Very stiff (m)**	**Refusal (m)**
PV01	1.50 – 1.80	0.90 – 1.50; 1.80 – 2.10	0 – 0.90; 2.1 – 2.40	2.40 – 2.70	2.40
PV02		0 – 0.60; 0.90 – 2.70	0.60 – 0.90	2.70 – 3.30	3.30
PV03	0.90 – 1.50; 2.10 – 2.40	0 – 0.30; 0.60 – 0.90; 1.50 – 2.10; 2.40 – 2.70; 2.40 – 2.70	0.30 – 0.60; 2.10 – 2.40	2.70 – 3.30	3.30
PV04	2.40 – 2.70	0 – 0.30; 0.60 – 0.90; 1.50 – 2.40; 2.70 – 3.30	0.90 – 1.50	0.30 – 0.60; 3.30 – 3.60	3.60

Consistency according to the N-value					
Test pit No.	**Loose / soft (m)**	**Medium dense / firm (m)**	**Dense / stiff (m)**	**Very dense / Very stiff (m)**	**Refusal (m)**
PV05		0 – 0.90		0.90 – 2.10	2.10
PV06		0 – 0.30; 0.60 – 0.90	0.30 – 0.60; 0.90 – 1.20	1.20 – 2.10	2.10
PV07		0 – 0.30		0.30 – 1.80	1.80
PV08		0 – 0.60	0.60 – 0.90	0.90 – 1.80	1.80
PV09		0 – 0.30	0.30 – 0.60	0.60 – 2.10	2.10
PV98		1.80 – 2.70	0 – 0.30; 0.60 – 1.80	0.30 – 0.60; 2.70 – 3.30	3.30

The consistency as described in the profiles, are consistent with the results from the DPSH tests. Based on the DSPH results, test pits TP01, TP03, TP34, TP36 and TP38 have soft to firm soils up to a depth of 5m and this should be taken into consideration when designing the foundations.

10. Laboratory results

10.1 Foundation Indicators

Representative samples were collected for laboratory testing at specific positions. The test results are attached in Appendix C. A summary of the results is shown in Table 7 below.

Table 7: Foundation Indicator Results

Sample No	Depth (m)	Soil Composition				GM	Atterberg Limits			Activity	Unified Soil Classification
		Clay (%)	Silt (%)	Sand (%)	Gravel (%)		LL (%)	PI (%)	LS (%)		
Topsoil											
PV 09	0 – 0.80	7	13	57	23	1.27	15	5	2.5	0.7	SC
PV 14	0 – 0.40	8	9	60	23	1.25	16	4	2.0	0.5	SC
PV 30	0 – 0.50	19	16	65	0	0.42	17	5	2.5	0.3	CL
PV 33	0 – 0.25	19	14	67	0	0.61	16	6	2.0	0.3	SC
PV 72	0 – 0.30	22	21	57	0	0.54	20	8	3.5	0.4	CL
PV 88	0 – 0.45	27	32	41	0	0.32	23	9	4.0	0.3	CL
Transported Layer											
PV 02	0.65 – 3.00	20	32	39	9	0.59	27	9	6.0	0.5	CL
PV 03	0.90 – 3.00	26	17	34	23	0.94	28	11	6.5	0.4	CL
PV 04	1.00 – 3.00	31	17	31	21	0.87	35	13	6.5	0.4	CL
PV 05	0.20 – 1.00	20	59	18	3	0.16	36	12	6.0	0.6	CL
PV 22	0.30 – 2.30	25	12	63	0	0.49	22	9	5.5	0.4	CL
PV 28	0.20 – 1.10	25	15	54	6	0.59	22	8	5.0	0.3	CL
PV 30	0.50 – 2.10	32	20	46	2	0.45	25	9	5.5	0.3	CL
PV 44	0.60 – 1.30	8	8	24	60	2.07	23	9	5.5	1.1	GC
PV 69	0.35 – 3.00	44	20	35	1	0.29	30	13	6.0	0.3	CL
PV 71	0.30 – 1.00	18	12	47	23	1.15	22	8	4.0	0.4	SC
PV 88	0.45 – 3.00	40	23	37	0	0.2	32	15	7.5	0.4	CL
PV 106	0.40 – 1.80	32	18	36	14	0.71	29	12	5.5	0.4	CL
PV 113	0.60 – 3.00	32	19	37	12	0.57	30	10	4.5	0.3	CL

Sample No	Depth (m)	Soil Composition				GM	Atterberg Limits			Activity	Unified Soil Classification
		Clay (%)	Silt (%)	Sand (%)	Gravel (%)		LL (%)	PI (%)	LS (%)		
Pedogenic Layer											
PV 10	2.00 – 3.00	43	22	28	7	0.43	36	11	6.5	0.3	ML or OL
PV 14	0.70 – 3.00	46	22	29	3	0.33	35	11	6.5	0.2	CL
PV 28	1.10 – 3.00	24	11	23	42	1.47	36	13	6.0	0.5	SC
PV 72	2.10 – 3.00	14	13	27	46	1.65	29	11	6.0	0.8	SC
PV 33	1.05 – 3.00	12	9	36	43	1.59	23	8	4.0	0.7	SC
PV 30	2.05 – 3.00	22	21	46	11	0.67	28	10	5.0	0.5	CL
PV 94	1.00 – 2.50	39	19	34	8	0.5	31	13	6.0	0.3	CL
PV 106	1.80 – 2.40	22	14	37	27	1.18	30	11	5.5	0.5	SC
Dolomite Residuum											
PV 08	0.80 – 3.00	25	20	39	16	0.87	31	13	6.0	0.5	CL
PV 16	1.10 – 3.00	15	10	40	35	1.40	32	9	5.5	0.6	SC
PV 18	2.20 – 3.00	14	8	27	51	1.79	36	13	6.5	0.9	GC
PV 21 1	1.20 – 2.05	10	14	33	43	1.71	25	9	4.5	0.9	SC
PV 52	2.20 – 3.00	31	18	50	1	0.4	26	10	5.5	0.3	CL
PV 80	0.55 – 2.80	32	23	36	9	0.55	34	12	6.0	0.4	CL
PV 82	1.20 – 2.00	15	15	36	34	1.44	23	9	5.0	0.6	SC
PV 86	2.20 – 2.40	11	13	41	35	1.55	30	16	8.0	1.5	SC
PV 92	0.35 – 3.00	17	8	17	58	1.92	26	14	6.5	0.8	GC
PV 103	1.20 – 2.20	27	21	40	12	0.66	32	11	5.5	0.4	CL
Karoo Clays											
PV 31	1.50 – 3.00	28	35	28	9	0.5	35	15	7.5	0.5	CL
PV 43	1.70 – 2.70	39	21	15	25	0.84	39	15	7.0	0.4	CL
PV 57	1.70 – 2.50	15	59	26	0	0.12	32	10	5.0	0.7	CL
PV 67	070 – 1.50	10	13	24	53	1.78	27	7	3.0	0.7	GC-GM
PV 91	0.40 – 1.40	7	22	52	19	1.11	24	11	5.0	1.6	SC

Legend			
	GM	=	Grading modulus
	LL	=	Liquid Limit
	WPI	=	Weighted Plasticity Index
	LS	=	Linear Shrinkage
	Activity*	=	Potential expansiveness of the soil according to Van der Merwe's method (Van der Merwe, 1973)

Table 7 indicates that:

The **topsoil** covering the site consists of clayey sand (SC) and lean clay (CL). The layer has a low (0,32) to very high (1,27) grading moduli. The fine fractions of this material also exhibit low (15,0 %) liquid limits as well as a low (2,0 %) linear shrinkage. The weighted plasticity index (WPI) of the soil is low, indicating that the material has low plasticity characteristics. The material has a low potential expansiveness, according to the method proposed by Van der Merwe (1973).

The **transported layer** generally comprises of clayey sand (SC), lean clay (CL) and clayey gravel (GC). The layer has very low (0,16) to very high (2,07) grading moduli. The fine fractions of this material also exhibit low (22,0 %) to moderate (36%) liquid limits as well as a low (4,5 %) to moderate (7,5%) linear shrinkage. The weighted plasticity index (WPI) of the soil is low to medium, indicating that the material has low plasticity characteristics. According to Van der Merwe (1973) the material has a low potential expansiveness.

The **pedogenic layer** has three layers. The ferruginsed gravel silt layer consists of clayey sand (SC) and lean clay (CL) with gravel fraction up to 46%. The layer has a low (0,33) to very high (1,65) grading moduli. The fine fractions of this material also exhibit low (22,0 %) to moderate (36%) liquid limits as well as a low (4,0 %) to moderate (6,5%) linear shrinkage.

The nodular ferricrete layer consists of gravel and lean clay (CL). The layer has a moderate (0,67) grading modulus. The fine fractions of this material also exhibit moderate (28,0 %) liquid limit as well as a low (5,0 %) linear shrinkage.

The partially hardpan ferricrete layer also consists of gravel and lean clay (CL). The layer has a low (0,50) grading moduli. The fine fractions of this material also exhibit moderate (31,0 %) liquid limits as well as a moderate (6,0%) linear shrinkage.

The weighted plasticity index (WPI) of these three layers is low to medium, indicating that the material has low plasticity characteristics. According to Van der Merwe (1973) these three layers have a low potential expansiveness.

The **dolomite residuum** generally comprises of clayey sand (SC), lean clay (CL) and clayey gravel (GC). The layer has a very low (0,40) to very high (1,92) grading moduli. The fine fractions of this material also exhibit low (23,0 %) to moderate (34%) liquid limits as well as a low (4,5 %) to moderate (8,0 %) linear shrinkage. The weighted plasticity index (WPI) of the soil is low to medium, indicating that the material has low plasticity characteristics. According to Van der Merwe (1973) the material has a low potential expansiveness.

The **Karoo clays** generally comprises of lean clay (CL) and clayey sand (SC). The layer has a very low (0,12) to very high (1,78) grading moduli. The fine fractions of this material also exhibit low (24,0 %) to moderate (39%) liquid limits as well as a low (3,0 %) to moderate (7,5%) linear shrinkage. The weighted plasticity index (WPI) of the soil is low to medium, indicating that the material has low to moderate plasticity characteristics. According to Van der Merwe (1973) the material has a low potential expansiveness.

10.2 Compaction tests

Materials identified as potential sources of construction materials were sampled for laboratory testing. The samples were subjected to compaction tests in which the moisture-density relationship was established. Californian Bearing Ratio tests were carried out to determine the suitability of the soils for use in constructing layer works below paved areas. The test results are summarised below.

Table 8 Compaction Results

Sample No	Depth (m)	OMC (%)	MDD (kg-/m³)	Swell (%)	CBR at various densities				TRH14
					90(%)	93(%)	95(%)	98(%)	
Transported Layer									
PV 44	0.60 – 1.30	10.8	1989	0.5	4.2	7	9	18	G9
PV 88	0.45 – 3.00	15.2	1840	0.2	8	9.2	11	14.5	G9
Pedogenic Layer									
PV 28	1.10 – 3.00	12.4	1984	0.2	6	9.2	12.5	25	G7
PV 33	1.05 – 3.00	13	1966	0.2	16	24	33	43	G7
PV 72	2.05 – 3.00	15.8	1821	0.5	11	15	18	24	G7
Dolomite Residuum									
PV 08	0.80 – 3.00	12.2	1980	0.3	5.2	10	15	23	G8
PV 18	2.20 – 3.00	9.8	2077	0.4	6.4	14	23	38	G8
PV 82	1.20 – 2.00	13	1980	0.1	13	21	28	42	G7
PV 86	2.20 – 2.40	13.3	1871	0.1	7.0	13.1	21	40	G8
PV 92	0.35 – 3.00	10.6	2090	0.1	8	18	29	53	G7
Karoo Clays									
PV 43	1.70 – 2.70	15.6	1798	2.9	3	4	5	8	G10

Legend OMC = Optimum moisture content

MDD	=	Maximum dry density (Mod AASHTO)
Swell	=	Soaked at 100% Mod AASHTO compaction
TRH14	=	Classification according to TRH14 guidelines

From the results in

Table 8 it is evident that:

The **transported layer** has a moderate (1840 kg/m^3) to high (1985 kg/m^3) maximum dry density and moderate to high (10.8 % to 15.2 %) optimum moisture content value. The CBR swell value is very low to low. Low to moderate CBR values were yielded at 93% Mod AASHTO whereas CBR values were yielded at 95 % Mod AASHTO. Material is classified as G10 material, thus suitable for use as a subgrade material and in the construction of engineered fills of low stiffness.

The two **pedogenic layers** that were tested reveal that the **ferruginised gravel layer** has a high (1966 kg/m^3 to 1984 kg/m^3) maximum dry density and high (12.4% to 13.0%) optimum moisture content. The CBR swell value is very low. Low to moderate CBR values were measured at 93% Mod AASHTO, and low to moderate values where attained at 95% Mod AASHTO density.

The ferruginised gravel layer has a moderate (1821 kg/m^3) maximum dry density and high (15.8 %) optimum moisture content. The CBR swell value is low. Moderate CBR values were measured at 93% Mod AASHTO, and moderate values where attained at 95% Mod AASHTO density.

The two pedogenic layers (**ferruginised gravel and nodular ferricrete)** are classified as **G7** according to the TRH 14 (CSIR: 1987) guidelines and considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills.

The **dolomite residuum** has a moderate (1871 kg/m^3) to very high (2090 kg/m^3) maximum dry density and moderate to high (9.3% to 13.3%) optimum moisture content. The CBR swell value is very low to low. Moderate CBR values were measured at 93% Mod AASHTO, while moderate values where attained at 95% Mod AASHTO density.

At PV82 and PV92 the material is classified as **G7** according to the TRH 14 (CSIR: 1987) guidelines and considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills. At PV8, PV18 and PV86, the material is classified as **G8** material (TRH 14

Guidelines, 1987), thus also considered suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills.

The **Karoo clay** has a moderate (1840 kg/m^3) to high (1985 kg/m^3) maximum dry density and moderate to high (10.8 % to 15.2 %) optimum moisture content value. The CBR swell value is very low to low. Low to moderate CBR values were yielded at 93% Mod AASHTO whereas CBR values were yielded at 95 % Mod AASHTO. Material is classified as G10 material, thus suitable for use as a subgrade material and in the construction of engineered fills of low stiffness.

10.3 Chemical tests

Several soil properties have an effect on buried metals. These properties are:

- Electrical conductivity of the soil
- Chemical properties of the soil
- Ability of the soil to support sulphide reducing bacteria
- Heterogeneity of the soil (long-line currents)
- Differential aeration
- Stray currents in the soil, and
- Bacteria attack.

The conductivity of the soil has a profound influence on the rate of corrosion of buried metallic objects. Based on significance of soil resistivity on corrosivity, Duligal (1996) provides the following table for evaluation of the conductivity of soil.

Table 9: Guideline values for interpretation of soil conductivity (Duligal, 1996)

Soil conductivity		
Soil conductivity (mS/m)	**Soil resistivity (Ohm.cm)**	**Corrosivity classification**
More than 50	0 – 2000	Extremely corrosive
25 – 50	2000 – 4000	Very corrosive
20 – 25	4000 – 5000	Corrosive
10 – 20	5000 – 10000	Mildly corrosive
Less than 10	>10000	Not generally corrosive

Disturbed samples of the residual soil horizons were taken and subjected to chemical (pH and conductivity) tests. The test results are summarised as follows. Based on Evans guideline (1977) a soil pH less than 6 indicates serious corrosion potential.

Table 10: Summarised chemical test results

Chemical tests				
Hole No	**Depth (m)**	**Horizon**	**pH**	**Conductivity (mS/m)**
PV 02	0.65 – 3.00	Transported	5.97	12.5
PV 03	0.90 – 3.00	Transported	6.14	9.7
PV 05	1.00 – 3.00	Karoo Clay	4.63	3.5
PV 14	0.70 – 3.00	Pedogenic	4.89	8.0
PV 21 1	1.20 – 2.05	Dolomite Residuum	5.09	4.2
PV 22	0.30 – 2.30	Transported	4.98	15.1
PV 30	2.10 – 3.00	Pedogenic	4.27	14.1
PV 31	1.50 – 3.00	Karoo Clay	4.33	8.0
PV 69	0.35 – 3.00	Transported	5.17	32.6
PV 71	0.30 – 1.00	Transported	5.93	25.3
PV 80	0.55 – 2.80	Pedogenic	4.66	4.6
PV 88	0 - .45	Topsoil	6.13	35.3
PV 91	0.40 – 1.40	Dolomite Residuum	4.65	7.0

According to the soil conductivity guideline values (Duligal, 1996) and the results in Table 10, the following can be seen:

The **topsoil** is very corrosive to buried steel elements and therefore special consideration should be given in the design against the deterioration of buried steel in soil. The **transported layer** is generally mildly corrosive to buried steel elements and therefore special consideration should be given in the design against the deterioration of buried steel in soil. The **pedogenic material, dolomite residuum and Karoo clays** are not corrosive to buried steel elements.

11. Resistivity

11.1 Overview

The electrical work included the following:

i) Perform comprehensive earth resistivity measurements of the proposed areas,

ii) Evaluation of measurements and results,

iii) Document initial findings with conclusions and recommendations.

11.2 Methodology

A total of 16 measurements of the resistivity at each site was taken, with each set of measurements taken perpendicular to each other. The spikes were inserted at predetermined intervals for each measurement and up to a 16m spacing, from where the next set of measurements are taken until the whole area the proposed substation was complete. The geophysical investigations comprised earth resistivity measurements of at the proposed two substation areas. A total of 16 resistivity measurements were taken at each site, which comprised two perpendicular measurements at each of the 8 test locations. The measurement intervals at a test location were increased at predetermined intervals up to a maximum spacing of 16m.

11.3 Results

The results from the field measurements are tabulated below:

Table 11: Soil resistivity test results for test pit PV3 and PV4

Electrode Spacing (m)	Depth (m)	Geometric Factor	PV3 & PV4							
			Test - 1		Test - 2		Test - 3		Test - 4	
			Rmeas (Ω)	ρMeas (Ω-m)	Rmeas (Ω)	ρMeas (Ω-m)	Rmeas (Ω)	ρMeas (Ω-m)	Rmeas (Ω)	ρMeas (Ω-m)
1	0.75	6.28	23.11	145.20	20.40	128.18	22.10	138.86	21.87	137.41
2	1.50	12.57	13.92	174.92	12.39	155.70	14.10	177.19	14.30	179.70
3	2.25	18.85	10.10	190.38	8.84	166.63	9.93	187.18	9.72	183.22
4	3.00	25.13	7.10	178.44	6.51	163.61	7.20	180.96	7.12	178.95
6	4.50	37.70	4.87	183.59	4.72	177.94	4.51	170.02	4.81	181.33
8	6.00	50.27	3.74	187.99	3.45	173.42	3.44	172.91	3.51	176.43
16	12.00	100.53	1.20	120.64	1.01	101.54	1.32	132.70	1.23	123.65

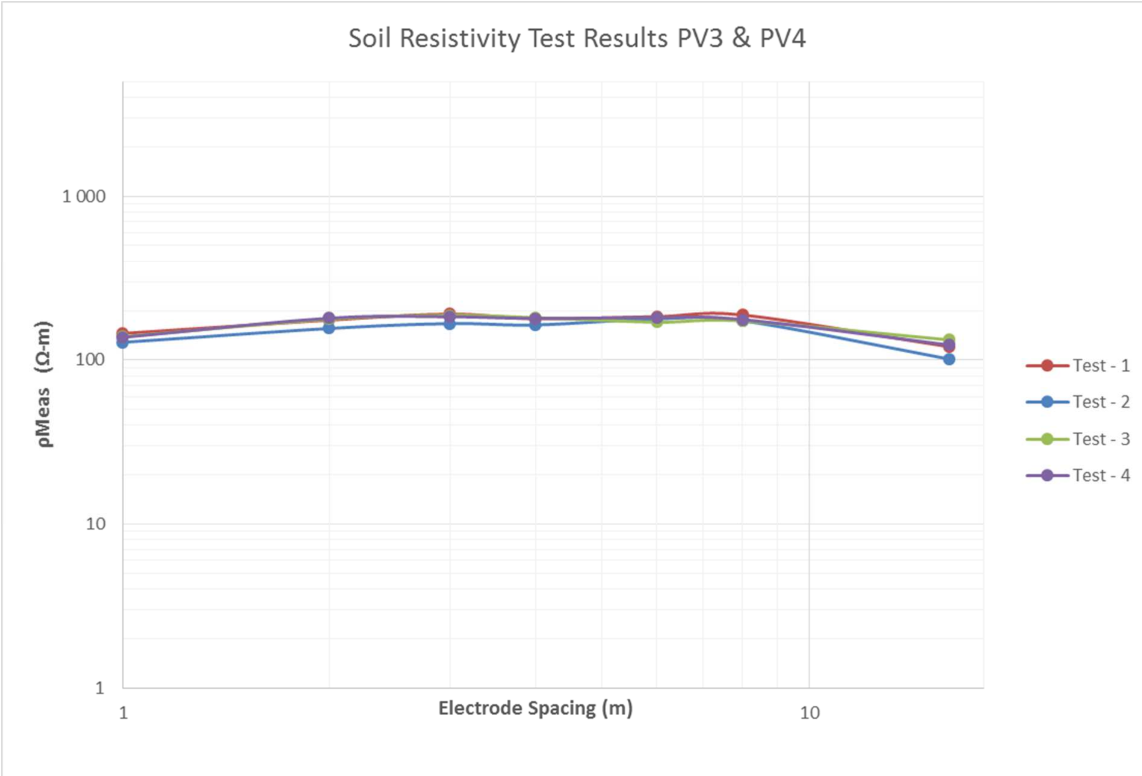

Figure 7: Graph for soil resistivity test results at test pits PV3 and PV4

Table 12: Soil resistivity test results for test pit PV7 and PV8

Electrode Spacing (m)	Depth (m)	Geometric Factor	PV7 & PV8							
			Test - 1		Test - 2		Test - 3		Test - 4	
			Rmeas (Ω)	pMeas (Ω-m)	Rmeas (Ω)	pMeas (Ω-m)	Rmeas (Ω)	pMeas (Ω-m)	Rmeas (Ω)	pMeas (Ω-m)
1	0.75	6.28	23.03	144.70	35.30	221.80	47.50	298.45	35.27	221.61
2	1.50	12.57	15.04	189.00	24.03	301.97	28.04	352.36	22.37	281.11
3	2.25	18.85	11.55	217.71	18.38	346.45	21.38	403.00	17.11	322.52
4	3.00	25.13	7.00	175.93	13.72	344.82	16.03	402.88	12.40	311.65
6	4.50	37.70	3.67	138.36	9.04	340.80	9.97	375.86	7.57	285.38
8	6.00	50.27	2.88	144.76	6.40	321.70	6.52	327.73	5.75	289.03
16	12.00	100.53	1.65	165.88	2.62	263.39	2.75	276.46	2.23	224.18

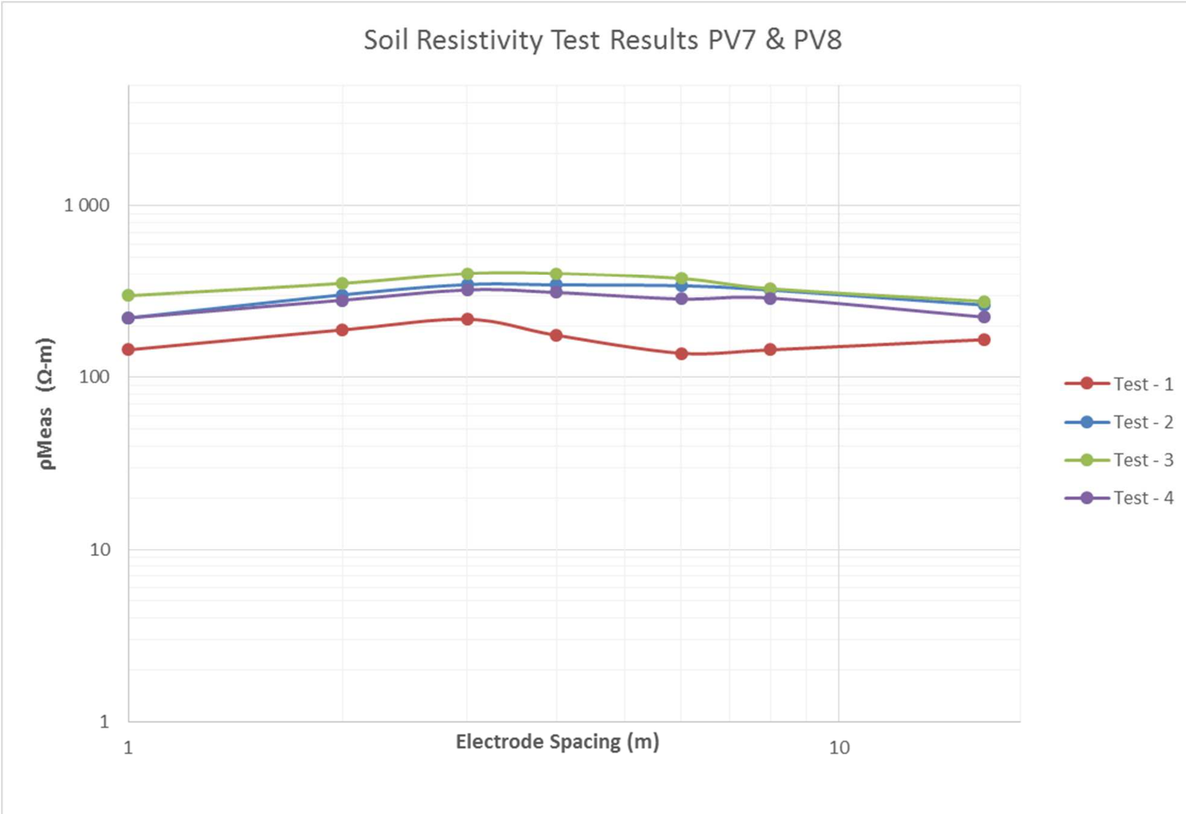

Figure 8: Graph for soil resistivity test results at test pits PV7 and PV8

11.4 Assessment of Results

The resistivity survey was carried out at the two proposed substation positions, site A (PV3 & PV4) and site B (PV7 & PV8). The profile of the resistivity measurements at the two proposed substation positions, indicated the layer of soil material is relatively uniform, with no hard layers.

12. Geotechnical considerations

The purpose of the geotechnical investigation was to provide a broad overview of the suitability of the land for the proposed development and outline obvious constraints. The following issues generally have to be considered for the site to be developed:

- Collapsible / compressible or expansive soil profile;
- Shallow seepage or groundwater level;
- Erodibility of the soil profile;
- Excavatibility;
- Undermined ground;
- Instability of areas of soluble rock;
- Steep slopes;
- Unstable natural slopes;
- Seismic activity;
- Areas subject to flooding

Each of the above-mentioned constraints and its applicability to this specific site is discussed in the sections that follow.

12.1 Collapsible / compressible soil

Indications of compressible soil conditions were observed in the topsoil, transported material and dolomite residuum encountered at the site. A predominantly very soft to soft consistency of the topsoil and transported material profile indicates that the in-situ materials may be compressible. No indication of a potential collapsible soil structure (pinhole structure) was observed in the profile. Problems with in-situ compressible soils are therefore expected at the site.

12.2 Shallow seepage or groundwater level

At the time of investigation, no groundwater or seepage was encountered within the depth limits afforded by the TLB. The presence of pedogenic material in the soil profile does however suggest the seasonal presence of shallow perched ground water tables. The development of temporary perched water tables as a consequence of sustained or intense rainfall periods must therefore be expected.

12.3 Expansive soil profile

Based on the field observations and laboratory testing, the material on site has a low potential expansiveness. Heave associated with expansive soils are therefore not anticipated for this site.

12.4 Erodibility of the soil profile

Due to its sandy nature, the topsoil is prone to erosion. However no significant erosion channels were observed on the site.

12.5 Undermined ground

The Libanon mine is located less than 1km from the north eastern site boundary. According to the general mining plan from Sibanye Gold, the site is undermined. The Department of Mineral Resources requirement is that an in-depth, detailed geotechnical risk assessment be carried if the proposed structures are within a horizontal distance of 100 metres from underground workings. This is defined in the Amended Chapter 17 of the Mine Health and Safety Act (MHSA) regulation 17.8 and 17.9 as amended in Government Gazette, No 34308 (No. 447) dated 27 May 2011 and as set out below.

Regulation 17(8) of this Act states that:

> *"No person may erect, establish or construct any buildings, roads, railways, dams, waste dumps, reserve land, excavations or any other structures whatsoever within a horizontal distance of 100 (one hundred) metres from workings, unless a lesser distance has been determined safe -*
>
> *(a) in the case of the employer, by risk assessment and all restrictions and conditions determined in terms of the risk assessment are complied with; or*
>
> *(b) in the case of any other person, by a professional geotechnical specialist and all restrictions and conditions determined by him or her or by the Chief Inspector of Mines are complied with;*

The gold mining within the rock of the Witwatersrand Group occurs at depths of more than 800m. This is far beyond the 100 metres stated in the legislation, and these undermined areas therefore do not need to be considered in a detailed geotechnical risk assessment. Furthermore, owing to its depth, it is unlikely that mine openings will affect the proposed development.

12.6 Steep slopes

The sites are characterised by a generally flat surface. No indication of the presence of unstable natural slopes was found during the desk study or field investigation.

12.7 Unstable natural slopes

The site is characterised by a flat topography. No indication of the presence of unstable natural slopes was found during the desk study or the field investigation. Stability of slopes such as in trench excavations or slopes on the cut faces for the building platforms has to be considered.

12.8 Seismic activity

According to the the published seismic hazard map of South Africa (Kijko, *et al.* 2003), a peak ground acceleration of less than 0.20g with a 10 % probability of exceedance in 50 years is predicted for the area. The site therefore has a high risk for seismic activity. The final design of buildings on this site has to account for the high expected seismic hazard levels.

12.9 Areas subject to flooding

The site is reasonably flat and the draining of surface water would need proper consideration. A flood line study falls outside of Aurecon's current scope of work. It is recommended that a formal flood line study be conducted if deemed necessary.

12.10 Soil corrosivity

According to the significance of soil resistivity on corrosivity by Duligal (1996) and guideline by Evans (1977) for acidity of soil, the soils topsoil and transported layer are categorised as aggressive to buried steel elements and therefore special consideration should be given in the design against the deterioration of buried steel in soil.

12.11 Suitability of construction materials

The **transported** material is classified as G10 material. Thus it is only suitable for use as a subgrade material and in the construction of engineered fills of low stiffness.

The **pedogenic** material encountered on site is classified as **G7** according to the TRH 14 (CSIR: 1987) guidelines and considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills. It should be noted that pedogenic material suitable for construction occurs at depths exceeding 1m in places on the site, (see table 8).

The **dolomite residuum** is classified as **G7** and **G8.** It is considered to be suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills (**G7**) also considered suitable for the construction of selected subgrade layer material and in moderate stiffness engineered fills (**G8**). G7 type dolomite residuum material suitable for construction occurs at depths exceeding 1m in places on the site (see Table 2).

The **kaolinitic (Karoo) clay** encountered at the site is classified as **G10** material, thus at best it is suitable for use as a subgrade material and in the construction of engineered fills of low stiffness.

The distribution of the various materials encountered on site is shown on drawing 112286-G01-02 in Appendix F. The interpretation has been made from point data and variances can be expected depending on the ground conditions.

13. Recommendations

13.1 Founding conditions for key infrastructure

The infrastructure for the solar park is expected to be the following:

- **Photovoltaic installations** (movement sensitive, single pole structure; PV panel with compression, tension, shear and moment forces.
- **Substation with transformers** (light and heavy)
- **Single storey buildings** typically comprising control rooms, gate houses etc.

It is also important to consider the following philosophies:

- Numerous PV panels will need to be installed. This will favour a founding solution that is robust in relation to varying ground conditions
- The PV panels will need to be installed rapidly, with minimal reliance on complex methodologies (e.g driving the pile may be preferred rather than pre-drilling, grouting and then driving the pile).

Additional founding precautions may have to be taken, depending on the outcome of the dolomitic stability investigation of the site.

13.1.1 PV panels

The loads exerted by the PV panels are minimal. Lateral and uplift forces have a much greater effect on the stability of the structure. Design forces provided by the client are shown in Table 13 below.

Table 13: Wind and uplift forces acting on a PV panel

Applied Loads		
Vertical force	**Horizontal force**	**Moment**
6 kN	6kN	7kNm

The PV panels need a simple connection system to the foundation where the pre-manufactured superstructure could rapidly connected to the substructure. In the areas with shallow bedrock, one could consider founding the structures by means of pad foundations. There is however constraints regarding pad foundations such as:

- Excavation to competent material, which is expected to be present at varying depths
- Anchoring the footing to avoid uplift
- Installing the footing and connection facilities for the PV panel base
- Allowing for time for concrete of footing to cure

Alternatively the PV panels can be founded by means of piled foundations. Two alternatives, driven and predrilled grouted piles were investigated.

A driven pile foundation would provide a possible foundation solution as construction programme requirements would be achieved as well as a desired uniform foundation design for the site. Ideally in soft soils, a single pile foundation could be driven into the soil to a predetermined depth where firm material is encountered. Due to the presence of the chert boulders and hardpan ferricrete in the soil profile, driving of piles could be problematic as it will only allow the pile to proceed to the top of the boulders, hard pan ferricrete or bedrock and anchoring may be minimal. If soft or firm material occurs in the zone above the bedrock lateral support for the pile may not be adequate. Driven piles that are generally not installed in predrilled boreholes are therefore not the preferred solution.

Alternatively, the PV panels can be founded by means of steel tube piles or precast concrete piles installed in pre-drilled grout filled holes. This entails the following:

- Pre-drilling holes with a percussion drill to the required depth. The diameter of the holes must be slightly larger than the diameter of the steel pile (i.e approximately 175 mm holes for 150 mm steel piles)
- Filling the holes with a cement grout
- Installing the open ended steel tube piles or precast concrete piles to the required depth. The tube piles can be manufactured to allow easy connection of the PV panels and connecting cabling.

The investigation revealed that foundation conditions are variable at the site. The extent of variability is illustrated on drawing 112286-G01-03 in Appendix F.

- In view of the above, the following founding solution is proposed for the PV panels: Pre-drilling holes with a percussion drill to a depth of 3 m. The diameter of the holes must be slightly larger than the diameter of the steel pile (i.e approximately 175 mm holes for 150 mm diameter piles)
- Filling the holes with a cement grout.
- Installing the open ended steel tube piles or precast concrete piles to a depth of 3 m. The tube piles can be manufactured to allow easy connection of the PV panels and connecting cabling.

Horizontal and vertical displacements for PV panels subjected to forces described in Table 14 above have been calculated for 150 mm diameter piles and for 200 mm diameter piles. With the aid of computer software (Repute) the different displacements have been calculated for piles founded in areas where firm to stiff or medium dense to very dense material occurs at depth exceeding 1m (competent profile) and in areas where soft to firm or loose to medium dense material occurs to depths exceeding 3m (soft profile). The results are shown in Table 14.

Table 14: Foundation pile displacements

Material	Horizontal displacement (mm)	
	150 mm pile	200 mm pile
Soft profile to a depth of 3 m	17.47	11.04
Soft material to a depth of 2m underlain by a competent profile	9.32	4.58

13.1.2 Substation and transformers

Each of the transformers and inverters to be erected at the substation are in the order of 5 m long and 2.5 m wide and weigh between 40 and 85 tonnes.

Owing to the flat topography of the site it is anticipated that major cut and fill operations will not be necessary to establish a level platform for the proposed sub-station at either of the two substation locations. It should be noted that the recommendations below do not take any dolomitic stability situations regarding possible sinkhole or doline formations into account. Two sets of recommendations (one for each substation location) are provided.

13.1.2.1 Southeastern substation location

The south eastern site is underlain by very soft to firm sandy silt and sandy clayey silt that extends to depth exceeding 3.0 m. To avoid excessive settlements caused by consolidation of these mainly cohesive materials it is recommended that transformers and inverters be founded on soil rafts constructed of imported G7 material.

In order to support ancillary components it is recommended that they be founded on a 2.0 m thick engineered soil raft. The transformers shall be founded on a concrete plinth on the soil raft. The thickness of the imported fill below the proposed transformer foundations shall not be less than 2.0m.

The materials at the base of the excavation shall be ripped to a depth of 150 mm and subsequently compacted to 90 % of Modified AASHTO density or better, prior to importation of approved backfill.

Backfill between the base of the excavation and the proposed founding depth of the plinth shall consists of imported G7 (or better) material (according to TRH14) placed in 150 mm thick layers, each compacted to at least 93 % Mod AASHTO density.

Following receipt and consideration of laboratory test results it is concluded that the following limitations are imposed on the founding methodology of ancillary structures:

- For a 2m thick engineered fill the foundations for the transformers and circuit breakers will be placed at surface of the soil raft. Should wider foundations be required the thickness of the engineered fill shall be appropriately increased.
- Foundation loads exerted by the foundations supported on the G7 engineered fill shall not exceed 100kPa.
- Excavations shall be battered at angles no steeper than 45° from the horizontal. Alternatively excavations shall be shored in the instance where vertical excavations are required.

13.1.2.2 Southwestern substation location

The south western site is underlain by very soft to firm sandy silt and sandy clayey silt that extends to 1.0 m depths and occasionally to 1,5 m depth. To avoid excessive differential settlements caused by consolidation of these materials it is recommended that transformers and circuit breakers as described above be founded on soil rafts constructed of imported G7 material.

In order to support ancillary components it is recommended that they be founded on an engineered soil raft extending to 1 m depth where stiff sandy silt occurs below 1,0m and to 1,5m depth where stiff sandy silt is encountered below 1,5 m depth.

The materials at the base of the excavation shall be ripped to a depth of 150 mm and subsequently compacted to 90 % of Modified AASHTO density or better, prior to importation of engineer approved backfill.

Backfill between the base of the excavation and the proposed founding level of the plinth shall consists of imported G7 (or better) material (according to TRH14) placed in 150 mm thick layers, each compacted to at least 93 % Mod AASHTO density.

Following receipt and consideration of laboratory test results it is concluded that the following limitations are imposed on the founding methodology of ancillary structures:

- It is recommended that the strip foundations will be placed at a depth of not more than 0.5m below the final platform level.
- Foundation loads exerted by the foundations supported on the G7 engineered fill shall not exceed 100 kPa.

- Excavations shall be battered at angles no steeper than 45° (from the horizontal). Alternatively excavations shall be shored in the instance where vertical excavations are required.

13.1.2.3 Drainage measures

The following drainage measures shall be implemented in conjunction with a formal drainage design.

- No accumulation of surface water is permitted and the entire development shall be properly drained.
- All trenches and excavation works shall be properly backfilled and compacted in order prevent infiltration of surface run-off into these excavations. Backfilling should be done at optimum moisture content, in layers not exceeding 150 mm to at least 93 % of modified AASHTO density.

13.1.3 Light building structures (control rooms, etc.)

From information received form the client, loads of 50 kPa are expected to be exerted by the footings and plinths that will support the various substation components.

It is understood that the light building structures will be located in the vicinity of the substations. It should be noted that the recommendations below do not take any dolomitic stability situations regarding possible sinkhole or doline formations into account. Two sets of recommendations (one for each substation location) are provided.

13.1.3.1 Southeastern substation location

The south eastern site is underlain by very soft to firm sandy silt and sandy clayey silt that extends to depth exceeding 3.0 m. To avoid excessive differential settlements caused by consolidation of these materials it is recommended that light buildings be founded on soil rafts constructed of imported G7 material.

In order to these light single storey buildings it is recommended that they be founded on a 1.5 m thick engineered soil raft. The thickness of the imported fill below the proposed ancillary foundations shall not be less than 1.5 times the proposed foundation width of the structure (e.g.: engineered fill below a 0.6 m wide strip footing will be required to extend to a depth of not less than 0.9 m below the proposed founding level of the strip footing). Consequently, for a structure founded at a depth of 0.5m into the 1.5m thick engineered fill, the foundations shall not exceed a width of 0.6 m

The materials at the base of the excavation shall be ripped to a depth of 150 mm and subsequently compacted to 90 % of Modified AASHTO density or better, prior to importation of engineer approved backfill.

Backfill between the base of the excavation and the proposed platform level shall consists of imported, engineer approved G7 (or better) material (according to TRH14) placed in 150 mm thick layers, each compacted to at least 93 % Mod AASHTO density. The structures can then be founded by means of conventional pad and strip footings at a shallow depth in the soil raft.

For a 1.5 m thick engineered fill the foundations will be placed at a depth of not more than 0.5m below the final platform level. The width of such foundations shall not exceed 0.6 m. Should wider foundations be required (assuming greater loads) the thickness of the engineered fill shall be appropriately increased. Foundation loads exerted by the foundations supported on the G7 engineered fill shall not exceed 100 kPa

Excavations shall be battered at angles no steeper than 45° (from the horizontal). Alternatively excavations shall be shored in the instance where vertical excavations are required.

13.1.3.2 Southwestern substation location

The south western site is underlain by very soft to firm sandy silt and sandy clayey silt that extends to 1.0 m depths and occasionally to 1, 5 m depth. To avoid excessive settlements (exceeding 5 mm) caused by consolidation of these materials it is recommended that light buildings be founded on soil rafts constructed of imported G7 material.

In order to support ancillary components it is recommended that they be founded on an engineered soil raft extending to 1 m depth where stiff sandy silt occurs below 1,0m and to 1,5m depth where stiff sandy silt is encountered below 1,5 m depth.

The materials at the base of the excavation shall be ripped to a depth of 150 mm and subsequently compacted to 90 % of Modified AASHTO density or better, prior to importation of engineer approved backfill.

Backfill between the base of the excavation and the proposed founding depth of the strip footing, the platform level shall consists of imported, engineer approved G7 (or better) material (according to TRH14) placed in 150 mm thick layers, each compacted to at least 93 % Mod AASHTO density. The structures can then be founded by means of conventional pad and strip footings at a shallow depth in the soil raft.

Foundation loads exerted by the foundations supported on the G7 engineered fill shall not exceed 100 kPa.

Excavations shall be battered at angles no steeper than 45° (from the horizontal). Alternatively excavations shall be shored in the instance where vertical excavations are required.

13.1.3.3 Drainage measures

The following drainage measures shall be implemented in conjunction with a formal drainage design:

- No accumulation of surface water is permitted and the entire development shall be properly drained.
- All trenches and excavation works shall be properly backfilled and compacted in order prevent infiltration of surface run-off into these excavations. Backfilling should be done at optimum moisture content, in layers not exceeding 150 mm to at least 93 % of modified AASHTO density.

All structures should be provided with a paved apron of 1.0 m wide around the building to prevent water ingress next to foundations and to avoid erosion from surface runoff from roofs. The ground surface shall be suitably sloped to prevent ponding of water in the vicinity of the structures.

All exposed cut slopes and fill batters shall be vegetated to prevent long-term erosion. Stormwater shall be collected in drainage channels and conveyed down slopes in a controlled manner.

13.2 Resistivity

The resistivity results obtained during this investigation must be considered during the detail design. The results revealed that the profile for the resistivity measurements indicated the layer of soil material is relatively uniform, with no hard layers at the proposed substation positions. No exceptional earthing mat designs are expected on this site.

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