ON BEHALF OF DYASONS KLIP PV 5 (PTY) LTD

PROPOSED DYASONS KLIP 5 PHOTOVOLTAIC PLANT

Stormwater Management Report

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Confidentiality Statement

Declaration

I, RICHARD HIRST, declare that I –

- act as an independent specialist consultant in the field of Stormwater Hydrology for the **Stormwater Management Report for the proposed DYASONS KLIP 5 Photovoltaic Facilities near Upington in the Northern Cape Province**;
- do not have and will not have any financial interest in the undertaking of the activity, other than the agreed remuneration for work performed in terms of the Environmental Impact Assessment Regulations 2006;
- have and will not have any invested interests in the proposed activity proceeding;
- have no, and will not engage in conflicting interests in the undertaking of the activity;
- undertake to disclose to the competent authority, any material information that have or may have the potential to influence the decision of the competent authority or the objectivity of any report, plan or document required in terms of the Environmental Impact Assessment Regulations, 2006;
- will provide the competent authority with access to all information at our disposal regarding the application, whether such information is favourable to the application or not.

Mr. R D Hirst *PrTech*

for **SiVEST Civil Engineering Division Gauteng**

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1 INTRODUCTION

SiVEST SA (Pty) Ltd was appointed by the applicant, **DYASONS KLIP PV 5 (PTY) LTD** to carry out a desktop study of the surface hydrology on a proposed site for the purposes of establishing a new 'Solar Photovoltaic' (PV) Energy facility on the Remainder of the Farm DYASONS KLIP-454, (referred to hereafter as 'Dyasons Klip').

This report therefore serves to outline the related surface, stormwater issues on the proposed site for the purposes of inclusion in the Draft Basic Assessment Report (DBAR) submission. In order to achieve this objective, the following will be assessed and discussed under their relevant headings in this report:-

- Climate
- Surface Hydrology
- Site Stormwater Management
- Conclusions & Recommendations

2 LOCALITY

The proposed site, is located ±21.6Km south, west of the town of UPINGTON in the Northern Cape Province. The farm is bound to the east by the farm, McTaggarts Camp, to the north by the farm Van Roois Vley, to the west by the Farm Bloems Mond and to the south by the Orange River. The N14 National Road which links Upington to Keimoes, is situated ±9 Km to the south of the proposed site and runs parallel to the Orange River.

The farm DYASONS KLIP-454 covers a total area of ±5725.28 ha; however, it is the intention of the developer to develop and utilize ±267.0 ha of the site for the new proposed PV plant.

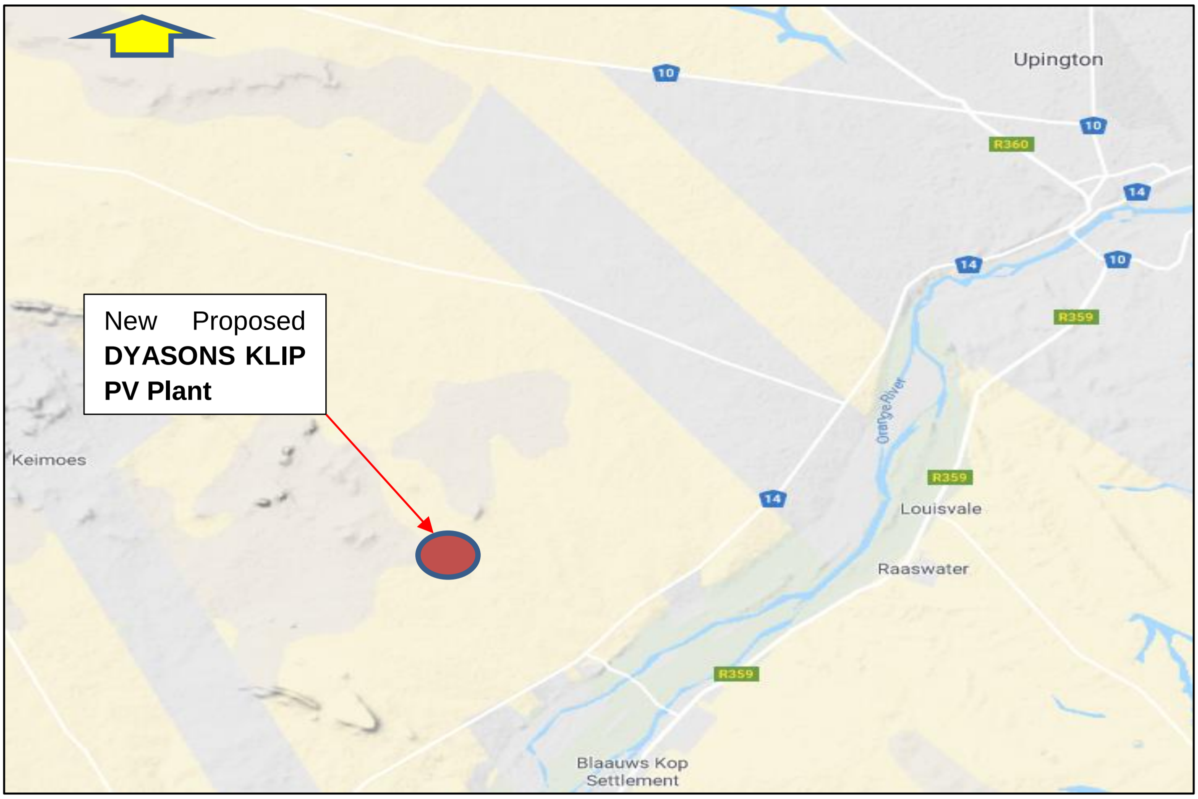

Figure 2:1 Locality Plan

3 GEOTECHNICAL STUDY

A comprehensive geotechnical investigation was carried out in March 2016 by Messrs' JEFFARES & GREEN (Pty) Ltd on the adjacent sites which will form part of the final reference during the detailed design stage of the project.

A summary extract from the Geotechnical Investigation Report confirms the site has the following soil horizons and bedrock:

- Alluvial Horizon (Gordonia Formation)
  Generally loose to medium dense red-brown poorly sorted silty sand to gravelly in places. Trial pits indicate a depth of 0.10m to 0.50m (Diamond rotary boreholes are poor indicators on soil horizon depth as a result of the poor material recovery of unconsolidated material). Alluvial soils may contain reworked calcrete, granite, gneiss, and schist nodules.

- Pedogenic Horizon (Mokolanen Formation)
  Chemically precipitated Calcrete was encountered in the majority of the boreholes and trial pits across the sites. Calcrete is generally well cemented hardpan to nodular with a creamy white to grey colour.

- Granite (Kanoneiland Granite and Louisvale Granite)
  Generally fine to medium grained biotite rich granite containing quartz and feldspar. Granite may be poorly foliated in places and has been described as a 'meta-granite'.

- Gneiss (Dyasonsklip Gneiss):
  Generally fine to medium grained intensely foliated bands of feldspar and biotite rich layers. A strong tendency for the rock to break along foliation planes.

- Schist (Bethesda Formation):
  Generally fine to medium grained intensely to very intensely foliated with a strong tendency to break along foliation.

The following is the Summary Conclusion:

The geology of the area is characterized by a thin, generally less than 1m, reworked calcrete and silty gravelly sand to nodular hard pan calcrete underlain by Gneiss, Schist and Granite of the Areachap Group of the Namaqua-Natal Metamorphic province. Rock outcrops of Gneiss, Schist and Granite may occur at surface.

Excavatability of the material is expected to require *intermediate to hard* excavation techniques across the study area.

4 CLIMATE

4.1 Average Temperature[1]

The Average Maximum temperatures range from 21° to 36° Celsius, with temperatures often exceeding 40° Celsius. (South African Weather service).

January is the warmest month of the year, with an average high-temperature of 35.5°C and an average low-temperature of 4.1°C in month of July.

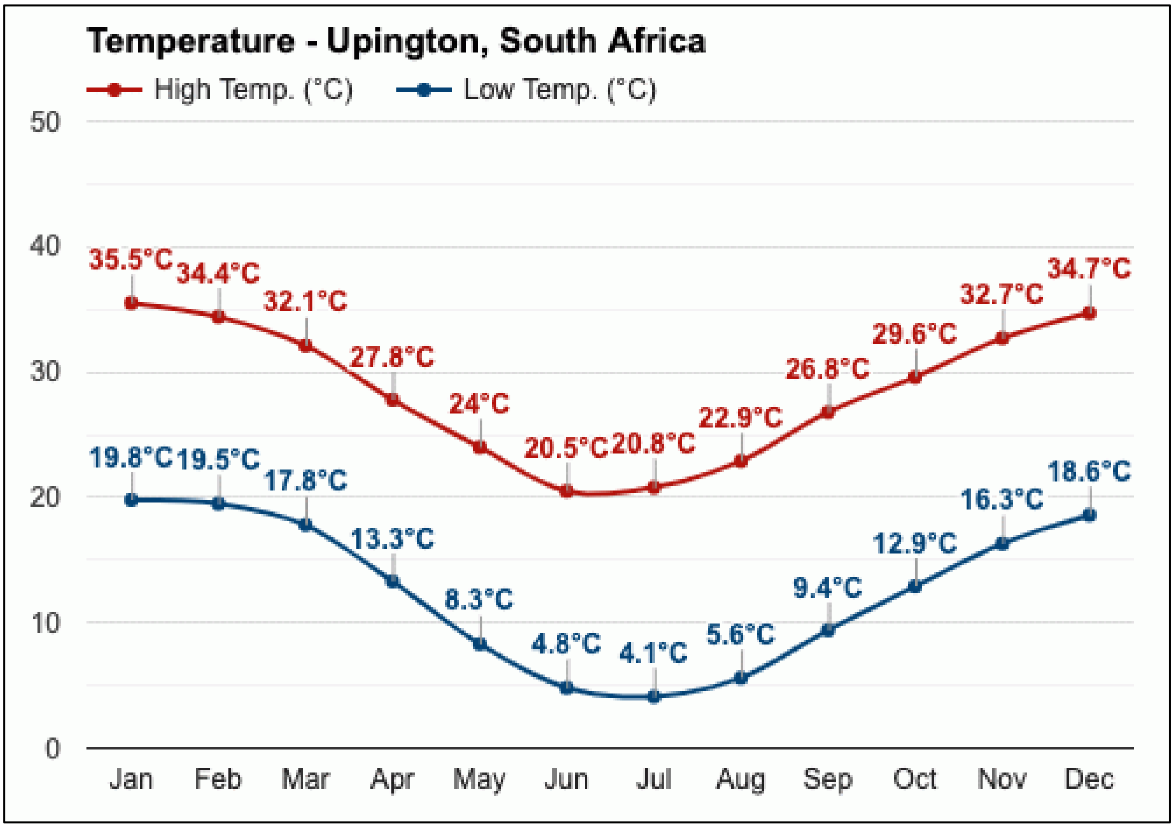

Figure 4:1 Average Temperature

4.2 Mean Annual Precipitation (MAP)[2]

The Upington / Keimoes region is arid and is considered a desert climate with an annual average rainfall of 189mm which falls between the months of October and May. The month of March is on average the wettest month of the year with 37mm accumulated for the month.

The average rainfall days per annum is ±23 days with the highest rainfall days being four, in the months of February & March.

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[1] En-Climate-data

[2] Weather Atlas

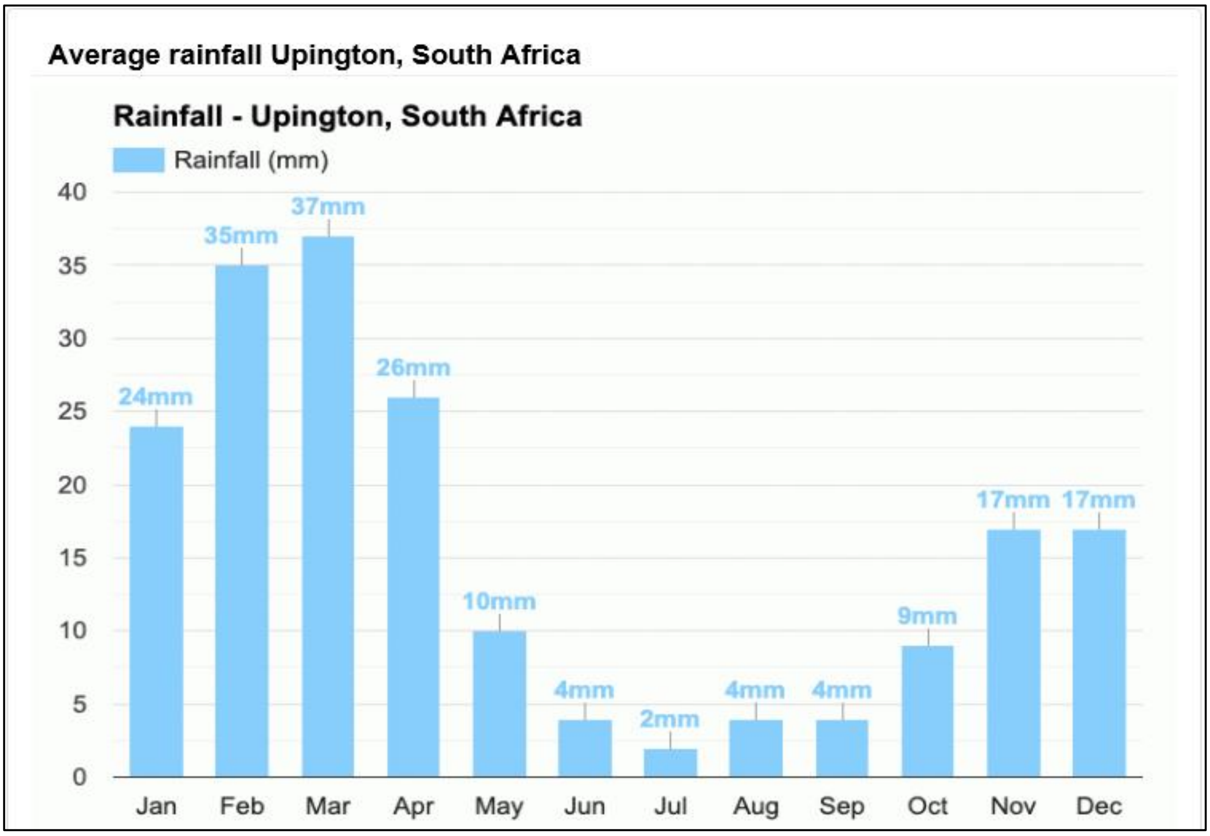

Figure 4:2 Average Rainfall

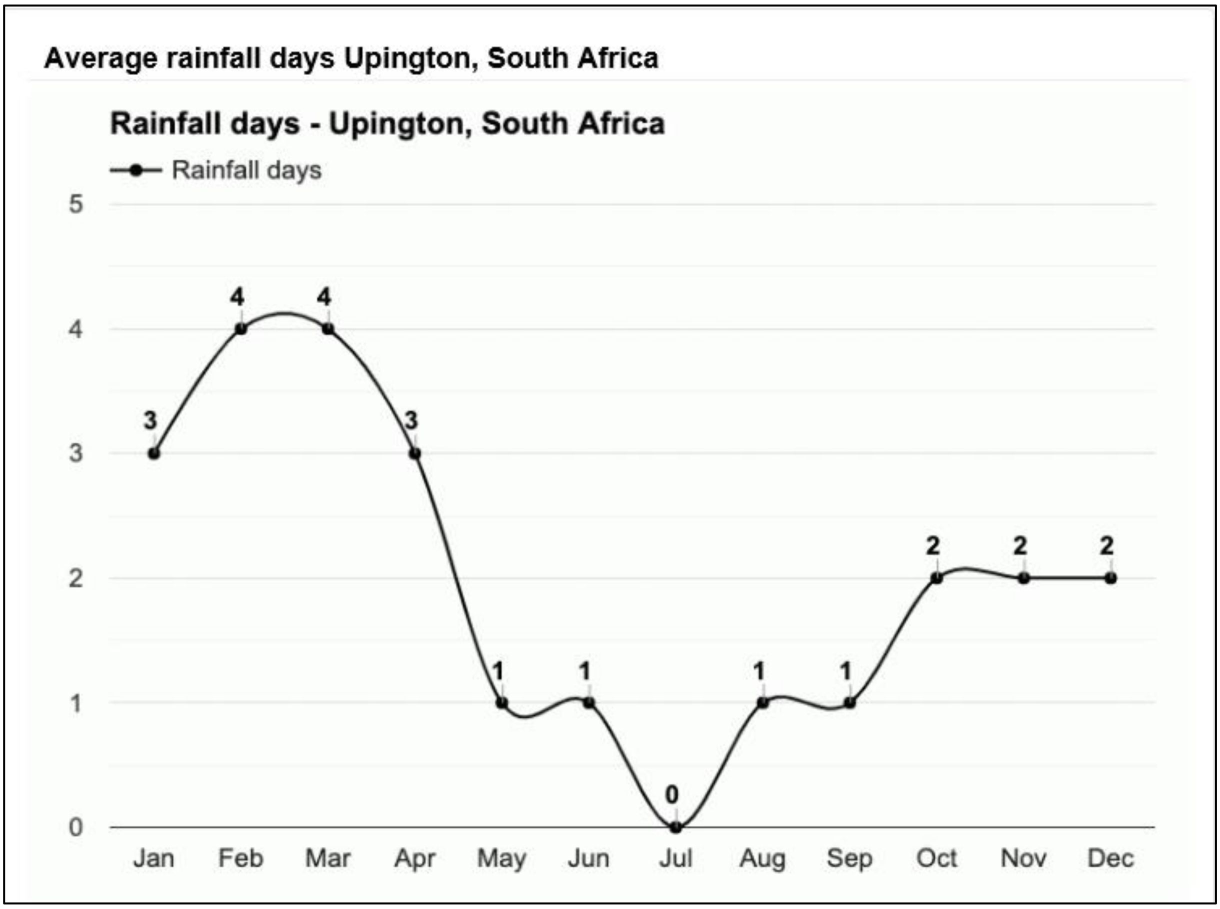

Figure 4:3 Average Rainfall Days

4.3 Humidity

The regions humidity ranges between a maximum of 50% in June, to a minimum of 30% in December.

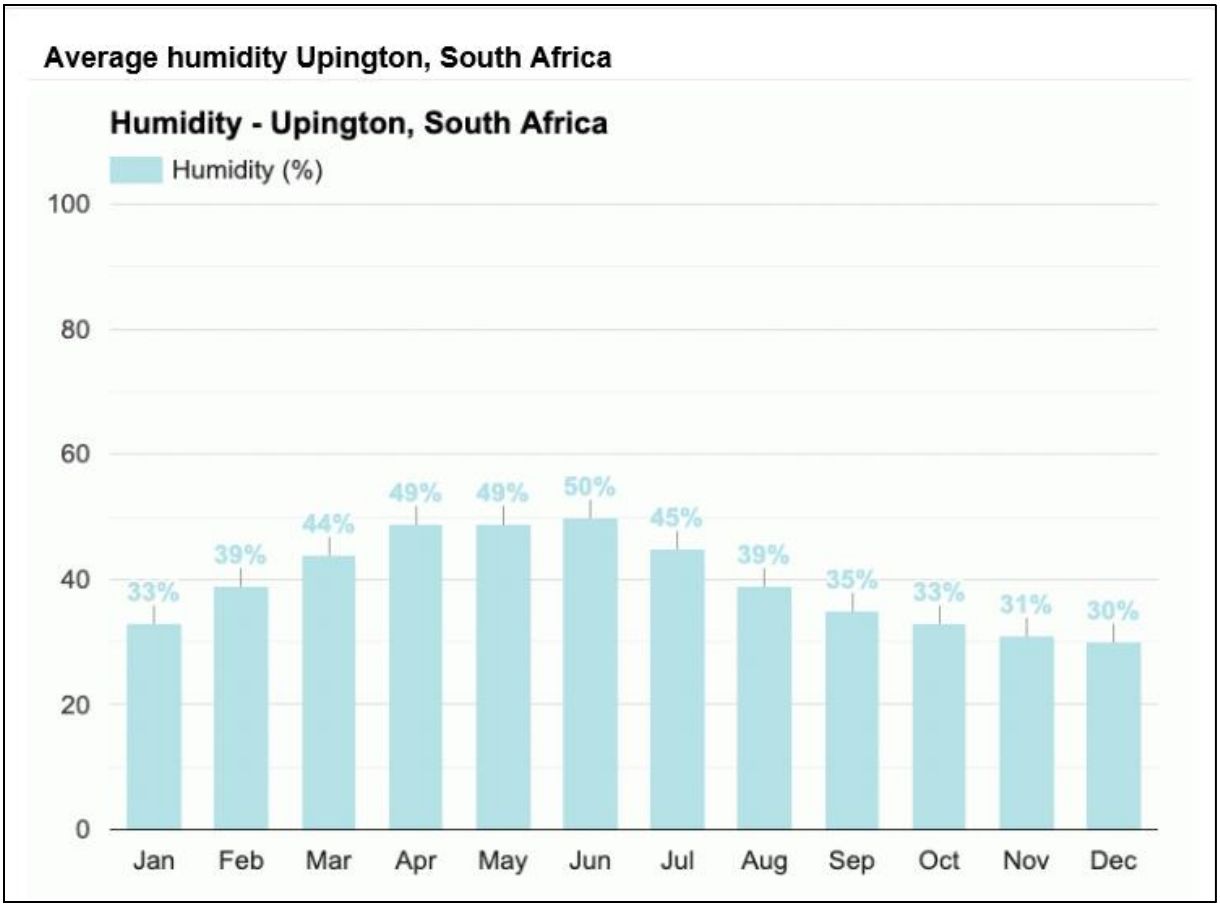

Figure 4:4 Average Humidity

5 SURFACE HYDROLOGY

5.1 Department Water & Sanitation (DWS) Drainage Catchment

5.1.1 Primary Catchment

The site falls within the 'Orange River' drainage catchment (Primary Catchment 'D') which covers an area of ±682 059km² and extends from Lesotho Highlands, Mahikeng and the Karoo as shown in Figure 5:1 below.

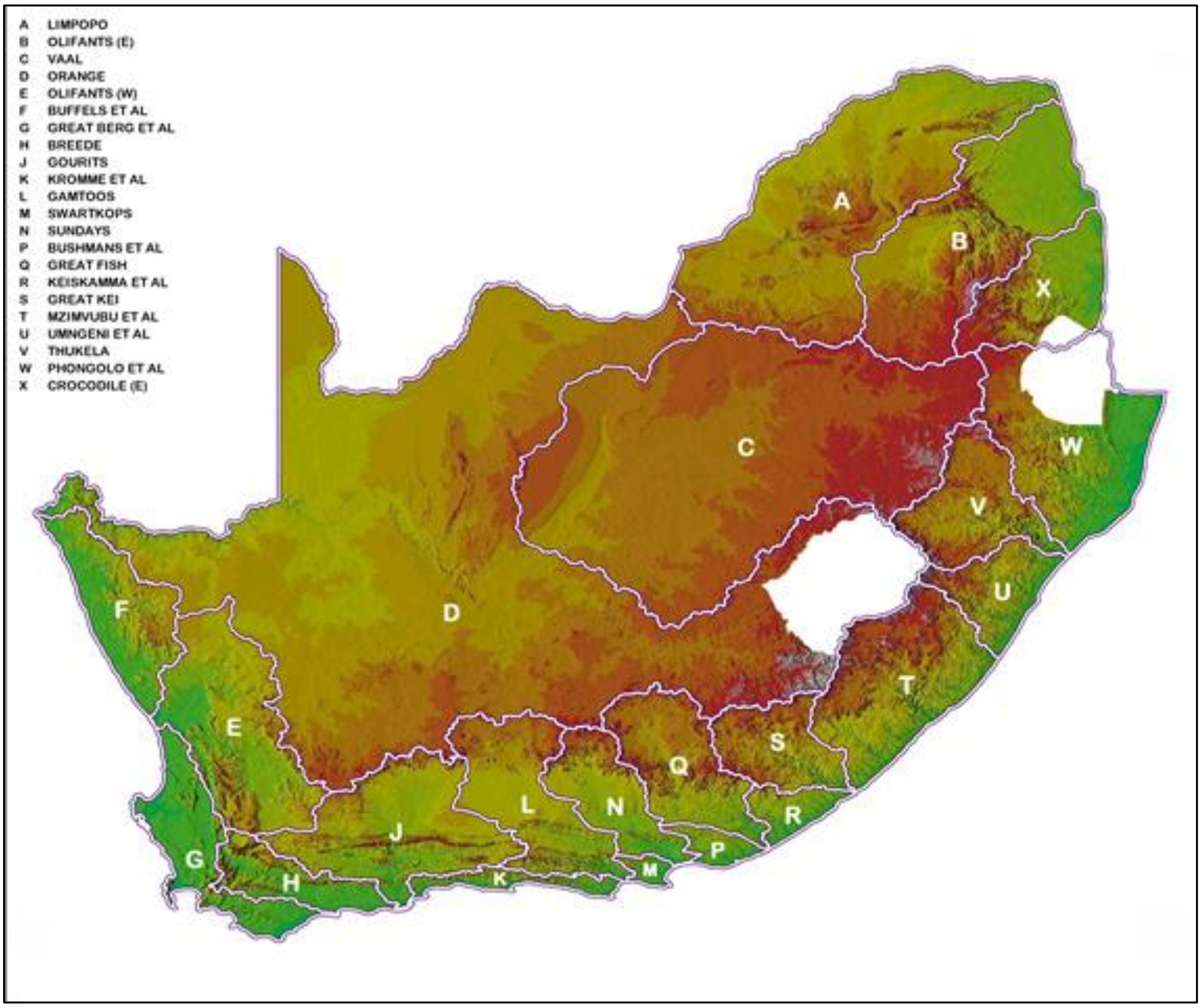

Figure 5:1 Primary Catchments

5.1.2 Quaternary Catchment

The majority of the Farm is located in the D73F Quaternary Catchment. The D73F catchment hosts one major river, namely the ‘Orange River’. It should be noted however, that the ‘Orange River’ will not have an influence on the site as it is situated ±8.0km to the south of the proposed site.

6 CURRENT SITE CHARACTERISTICS

6.1 Site Topography

A natural drainage line runs through the DYASONS KLIP-454 farm in a south easterly direction towards the Orange River. A number of tributaries from other farms link into this drainage line, but for the purposes of this study has only been considered if it impacts the ‘Dyasons Klip PV 5’ development as per Figure 6:1 below.

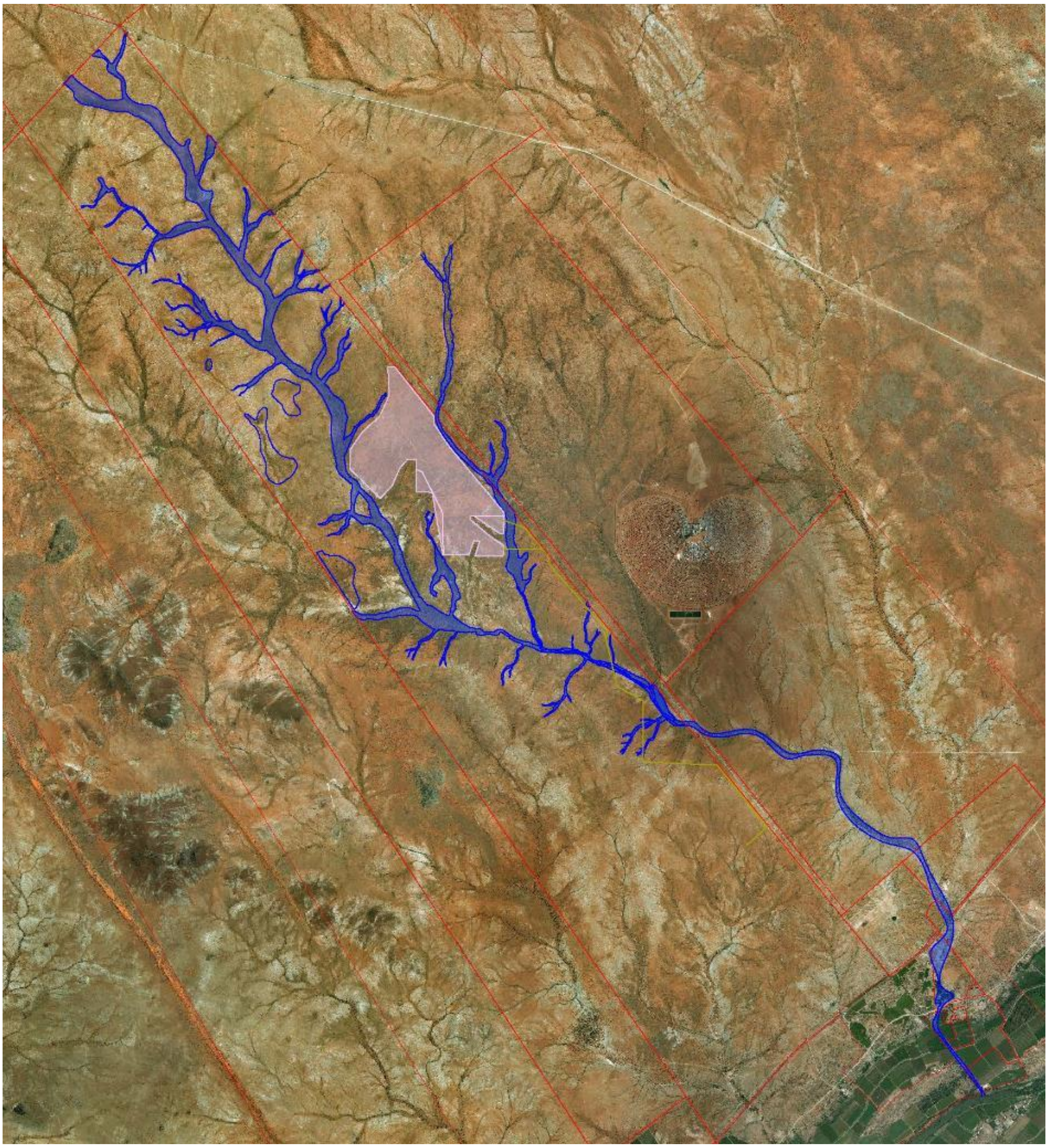

Figure 6:1 Dyasons Klip Farm Drainage Lines

The development site naturally and uniformly slopes from the east towards the west. We confirm that there are many minor natural drainage lines criss-crossing the site, however, larger drainage lines can be found outside the proposed developable area as shown in Figure 6:2 below.

We confirm that these larger drainage lines only become active during the few days of rainfall and therefore no floodline restrictions are applicable. The average slope of the natural ground is extremely flat @ ±1:111 or 0.9% in a south westerly direction.

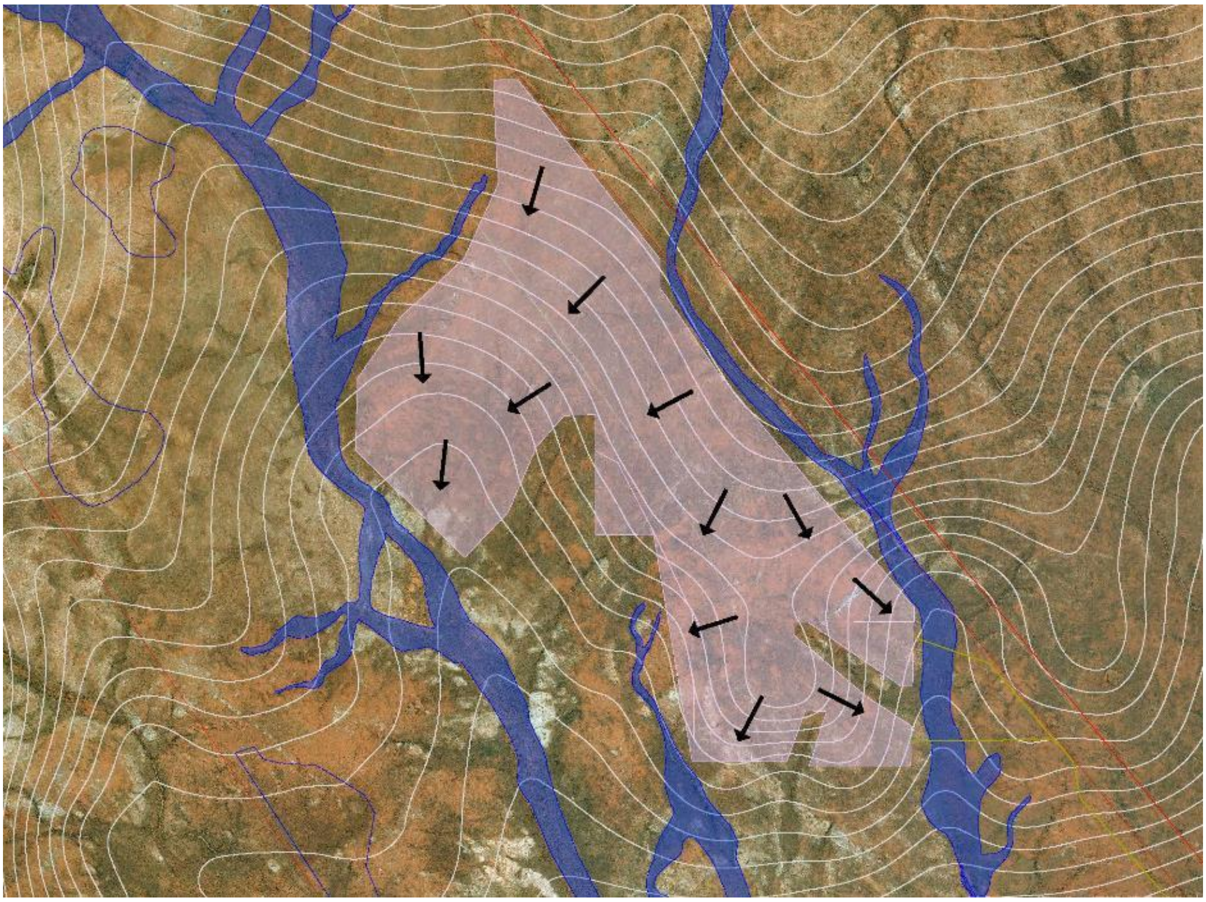

Figure 6:2 Larger Drainage Lines outside the site parameters

Please note that detailed contour data was not available for the broader study area and as such, the National Geo-Spatial Information (NGI)'s 25m DEM was sourced to provide terrain data for this area.

Contours were generated from the Digital Elevation Model (DEM) at one-meter intervals using ESRI's 3D Analyst Extension for ArcGis. We therefore recommend that an updated, detailed Stormwater Management Plan be completed once a more accurate Digital Terrain Model (DTM) of the site is available.

6.2 Site Vegetation

From Figure 6:3 below, we confirm that majority of the site covering is made up of sparse short grass with scattered small shrubs. The surface has been eroded well over time and has many drainage furrows with exposed weathered rock.

Figure 6:3: Current Site Vegetation

7 PRE-DEVELOPMENT RUN-OFF CALCULATION

7.1 Site Topography

From Figure 6:3 above, we confirm a natural slope of ±1:111 or 0.9% and therefore the following percentages: -

- Wetlands & Pans – 15%
- Flat Areas (3% to 10% slope) – 85%

7.2 Site Vegetation

With reference to Figure 6:3 above, being the typical ground cover on the site, the following percentage splits are applicable: -

- Thick Bush & plantations – 0%
- Light Bush & Farm Lands – 5%
- Grasslands – 50%
- No Vegetation – 45%

7.3 Geotechnical Conditions

With reference to Section 3 above, and the geotechnical report, we have assumed the soil conditions to be as follows: -

- Very Permeable – 10%
- Permeable – 15%
- Semi-permeable – 40%
- Impermeable – 35%

7.4 Hardstand Areas

We confirm that the property currently has no areas of hardstand: -

- Hardstand Areas – 0%

7.5 Run-Off Coefficient

Based on *Table 3C.1* of the *Drainage Manual – 6th Edition,* the following run-off coefficients have been assigned for this calculation: -

UN-DEVELOPED Run-off Factor	MAP < 600	MAP 600-900		
Surface Slope - Wetlands & Pans	0.01	0.03	15.0%	0.002
Surface Slope - Flat Areas (3-10%)	0.06	0.08	85.0%	0.051
Surface Slope - Hilly Areas (10-30%)	0.12	0.16	0.0%	0.000
Surface Slope - Steep Areas (>30%)	0.22	0.26	0.0%	0.000
			100.0%	
Soil - Very Permeable	0.03	0.04	10.0%	0.003
Soil - Permeable	0.06	0.08	15.0%	0.009
Soil - Semi-Permeable	0.12	0.16	40.0%	0.048
Soil - Impermeable	0.21	0.26	35.0%	0.074
			100.0%	
Vegetation - Thick Bush/Plantations	0.03	0.04	0.0%	0.000
Vegetation - Light Bush/Farm Lands	0.07	0.11	5.0%	0.004
Vegetation - Grasslands	0.17	0.21	50.0%	0.085
Vegetation - No Vegetation	0.26	0.28	45.0%	0.117
			100.0%	0.392

Table 7:1 Pre-Development Run-Off Coefficient

Based on the foregoing, we have calculated a **PRE-DEVELOPMENT Run-Off Coefficient** of **0.392**.

It should also be noted that no 'Area Reduction Factor' has been applied as we believe the drainage catchment areas are too small.

Refer Appendix 'B' – Pre Development Run-Off Areas (DWG No: 16145-5300-Rev A)

8 POST-DEVELOPMENT RUN-OFF CALCULATION

8.1 Site Development Plan (SDP)

With reference to the SDP, we confirm this proposed PV Plant layout will consist of a series of PV Panels in a structured pattern along with internal access road, sub-station and auxiliary buildings etc. The total areas envisaged footprint to be developed is ±267ha.

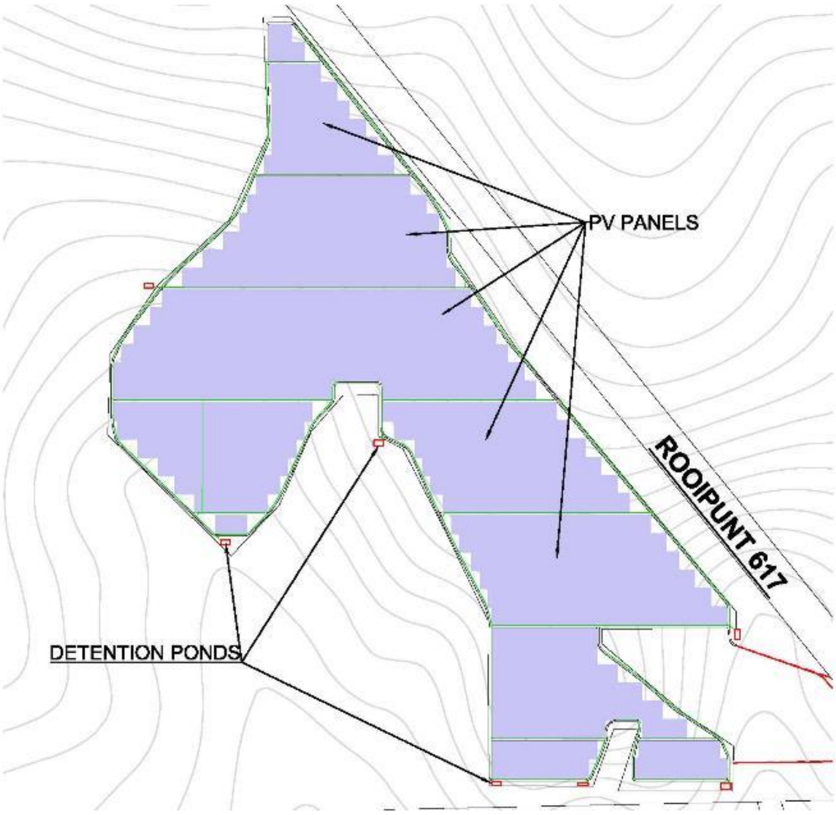

Figure 8:1 Proposed SDP

8.2 Site Vegetation

It is the intention to clear the site void of any large bushes and shrubs. We note however, the scattered grass on the site during construction will over a period of time, rejuvenate to Figure 6:3 above, the following percentage splits are applicable: -

- Thick Bush & plantations – 0%
- Light Bush & Farm Lands – 5%
- Grasslands – 50%
- No Vegetation – 45%

8.3 Geotechnical Conditions

With reference to Section 3 above, and the geotechnical report, we have assumed the percentages used in the ‘pre-development’ run-off coefficient remain unchanged as there would be little or no effect from the PV panel, support structure on the existing ground conditions. The following percentages have been used: -

- Very Permeable – 10%
- Permeable – 15%
- Semi-permeable – 40%
- Impermeable – 35%

8.4 Hardstand Areas

We confirm that the property once developed, will have no significant impervious surfaces in the form of surfaced roads, or surfaced covering other than the natural ground cover. It should be noted however, that gravel roads will be constructed across the site to provide access to the PV panels for maintenance purposes. We have therefore assumed a slight increase in area of imperviousness.

- Hardstand Areas – 5%

8.5 Run-Off Coefficient

Based on *Table 3C.1* of the *Drainage Manual – 6th Edition,* the following run-off coefficients percentages have been assigned for this calculation: -

	MAP < 600	MAP 600-900		
UN-DEVELOPED Run-off Factor				
Surface Slope - Wetlands & Pans	**0.01**	**0.03**	**15.0%**	0.002
Surface Slope - Flat Areas (3-10%)	**0.06**	**0.08**	**85.0%**	0.051
Surface Slope - Hilly Areas (10-30%)	**0.12**	**0.16**	**0.0%**	0.000
Surface Slope - Steep Areas (>30%)	**0.22**	**0.26**	**0.0%**	0.000
			100.0%	
Soil - Very Permeable	**0.03**	**0.04**	**10.0%**	0.003
Soil - Permeable	**0.06**	**0.08**	**15.0%**	0.009
Soil - Semi-Permeable	**0.12**	**0.16**	**40.0%**	0.048
Soil - Impermeable	**0.21**	**0.26**	**35.0%**	0.074
			100.0%	
Vegetation - Thick Bush/Plantations	**0.03**	**0.04**	**0.0%**	0.000
Vegetation - Light Bush/Farm Lands	**0.07**	**0.11**	**5.0%**	0.004
Vegetation - Grasslands	**0.17**	**0.21**	**50.0%**	0.085
Vegetation - No Vegetation	**0.26**	**0.28**	**45.0%**	0.117
			100.0%	**0.392**
DEVELOPED Run-off Factor				
Lawns - Sandy Flat <2%		**0.075**	**0.0%**	0.000
Lawns - Sandy Steep >7%		**0.175**	**0.0%**	0.000
Lawns - Heavy Soil Flat <2%		**0.15**	**0.0%**	0.000
Lawns - Heavy Soil Steep >7%		**0.30**	**0.0%**	0.000
Residential - Houses		**0.40**	**0.0%**	0.000
Residential - Apartments		**0.60**	**0.0%**	0.000
Industrial - Light		**0.70**	**0.0%**	0.000
Industrial - Heavy		**0.75**	**0.0%**	0.000
Business - City Centre		**0.90**	**0.0%**	0.000
Business - Suburban		**0.60**	**0.0%**	0.000
Streets / Roads		**0.80**	**5.0%**	0.040
Impervious Areas		**1.00**	**0.0%**	0.000
				0.040
Precentage UN-DEVELOPED			**95.0%**	**0.392**
Percentage DEVELOPED			**5.0%**	**0.040**
TOTAL Run-Off coefficient				**0.432**

Table 8:1 Post-Development Run-Off Coefficient

Based on the foregoing, we have calculated a **POST-DEVELOPMENT Run-Off Coefficient** of **0.432**.

Refer Appendix 'B' – Post Development Run-Off Areas (DWG No: 16145-5301-Rev A)

9 MODELLING

9.1 Modelling Selection

EMPIRICAL and STATISTICAL METHODS were not considered for this project as not enough hydrological records of the area, along with observed events were available. Therefore a deterministic method has thus been selected to determine the results.

This method comprises of mainly manual, graphic and computer generated spreadsheets. Therefore our selection of the 'UNIT HYDROGRAPH METHOD '(HRU 1972) and the 'RATIONAL METHOD' we believe is appropriate on the bases that the site does not have a varying degree of post development land change and does not have any existing permanent dams and sub-catchments. Computerized spread sheets have been used to assist with iterations and to eliminate manual calculation errors.

10 SURFACE HYDROLOGY

10.1 Catchment Analysis

The site has been divided up into separate sub-catchments as shown in Figure 10:1 below.

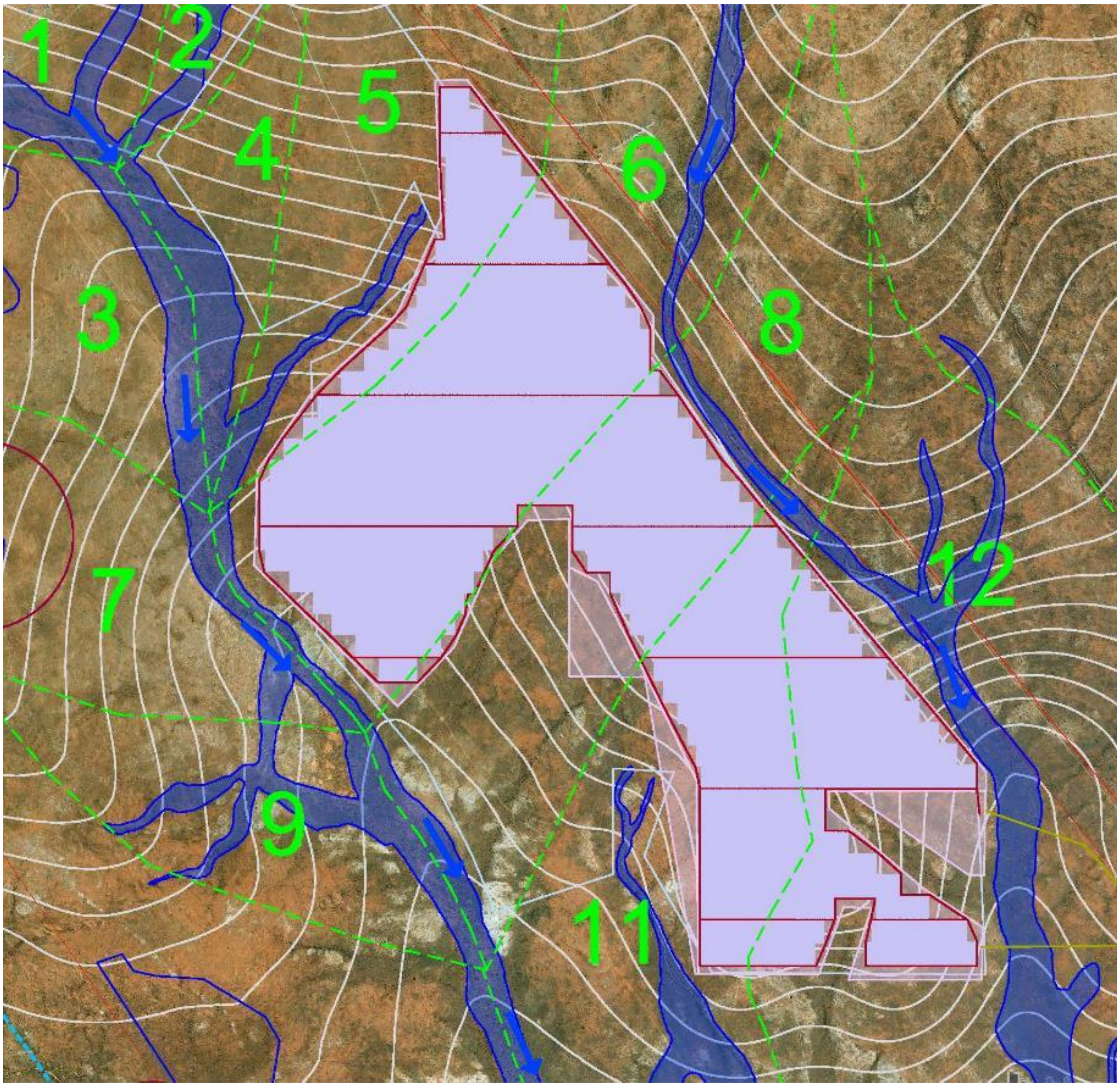

Figure 10:1: Site Run-Off Catchments

From the figure above, we confirm that a 'larger' drainage line runs along the western boundary of the proposed site. This drainage line is fed by numerous, 'minor' drainage lines as can be seen. It should be noted however, that the proposed plant layout (developed area), has been position such that all the drainage lines are not impeded or altered. Based on arial imagery, the drainage line running through 'Catchment 6' flows in a south westerly direction, turning towards the south east, before entering the site. However, the contour data used for this report does not support the same and therefore we suggest that this be investigated at 'Detailed Design' stage.

We confirm that catchments 5, 6, 8, 11 & 12 are the only catchments that will have a direct impact on the proposed site.

10.1.1 CATCHMENT N˚ 5 - Site Criteria

Weather Service Station Number :			283098	GEELKOP		
MAP :			152	mm		
2 / 5 / 10 Year Return Period Daily Rainfall :			26	41	52	mm
20 / 50 / 100 Year Return Period Daily Rainfall :			64	80	93	mm
Ave. Number of Days Thunder was heard :			32			
			m2	Km2	ha	
Catchment Area :		A =	2474882.18	2.475	247.4882	
			m	Km		
Length of Watercourse :		L =	3226.24	3.226		
Centroid of Catchment :		Lc =	2082.74	2.083		
Elevation Difference :		H =	26.5	m		
1085 Height Difference :		1085 =	19.7	m		
1085 Slope :		1085 =	8.14	m/Km		
STORM Re-occurance Period :		T =	20			
Run-Off Coefficient :		C =	0.392	**Pre-Development**		
	Kerby 'r' value :	r' =	0.225			
Run-Off Coefficient :		C =	0.432	**Post-Development**		
	Kerby 'r' value :	r' =	0.215			

Table 10:1 Development Criteria - Catchment N˚ 5

10.1.2 CATCHMENT N˚ 6 – Site Criteria

Weather Service Station Number :			283098	GEELKOP		
MAP :			152	mm		
2 / 5 / 10 Year Return Period Daily Rainfall :			26	41	52	mm
20 / 50 / 100 Year Return Period Daily Rainfall :			64	80	93	mm
Ave. Number of Days Thunder was heard :			32			
			m2	Km2	ha	
Catchment Area :		A =	4744657.61	4.745	474.4658	
			m	Km		
Length of Watercourse :		L =	4371.61	4.372		
Centroid of Catchment :		Lc =	2945.02	2.945		
Elevation Difference :		H =	26	m		
1085 Height Difference :		1085 =	22	m		
1085 Slope :		1085 =	6.71	m/Km		
STORM Re-occurance Period :		T =	20			
Run-Off Coefficient :		C =	0.392	**Pre-Development**		
	Kerby 'r' value :	r' =	0.225			
Run-Off Coefficient :		C =	0.432	**Post-Development**		
	Kerby 'r' value :	r' =	0.215			

Table 10:2 Development Criteria - Catchment N˚ 6

10.1.3 CATCHMENT N˚ 8 – Site Criteria

Weather Service Station Number :		283098	GEELKOP		
MAP :		152	mm		
2 / 5 / 10 Year Return Period Daily Rainfall :		26	41	52	mm
20 / 50 / 100 Year Return Period Daily Rainfall :		64	80	93	mm
Ave. Number of Days Thunder was heard :		32			
		m2	Km2	ha	
Catchment Area :	A =	1495622.6	1.496	149.5623	
		m	Km		
Length of Watercource :	L =	1012.75	1.013		
Centroid of Catchment :	Lc =	1505.69	1.506		
Elevation Difference :	H =	5.8	m		
1085 Height Difference :	1085 =	4	m		
1085 Slope :	1085 =	5.27	m/Km		
STORM Re-occurance Period :	**T =**	20			
Run-Off Coefficient :	**C =**	0.392	**Pre-Development**		
Kerby 'r' value :	**r' =**	0.225			
Run-Off Coefficient :	**C =**	0.432	**Post-Development**		
Kerby 'r' value :	**r' =**	0.215			

Table 10:3 Development Criteria - Catchment N˚ 8

10.1.4 CATCHMENT N˚ 11 – Site Criteria

Weather Service Station Number :		283098	GEELKOP		
MAP :		152	mm		
2 / 5 / 10 Year Return Period Daily Rainfall :		26	41	52	mm
20 / 50 / 100 Year Return Period Daily Rainfall :		64	80	93	mm
Ave. Number of Days Thunder was heard :		32			
		m2	Km2	ha	
Catchment Area :	A =	44563800.35	44.564	4456.3800	
		m	Km		
Length of Watercource :	L =	11333.3	11.333		
Centroid of Catchment :	Lc =	6499.48	6.499		
Elevation Difference :	H =	47	m		
1085 Height Difference :	1085 =	37.4	m		
1085 Slope :	1085 =	4.40	m/Km		
STORM Re-occurance Period :	**T =**	20			
Run-Off Coefficient :	**C =**	0.432	**Pre-Development**		
Kerby 'r' value :	**r' =**	0.215			
Run-Off Coefficient :	**C =**	0.432	**Post-Development**		
Kerby 'r' value :	**r' =**	0.215			

Table 10:4 Development Criteria - Catchment N˚ 11

10.1.5 CATCHMENT N˚ 12 – Site Criteria

Weather Service Station Number :		283098	GEELKOP		
MAP :		152	mm		
2 / 5 / 10 Year Return Period Daily Rainfall :		26	41	52	mm
20 / 50 / 100 Year Return Period Daily Rainfall :		64	80	93	mm
Ave. Number of Days Thunder was heard :		32			
		m2	Km2	ha	
Catchment Area :	A =	1574408.83	1.574	157.4409	
		m	Km		
Length of Watercource :	L =	1917.42	1.917		
Centroid of Catchment :	Lc =	1202.55	1.203		
Elevation Difference :	H =	11	m		
1085 Height Difference :	1085 =	7	m		
1085 Slope :	1085 =	4.87	m/Km		
STORM Re-occurance Period :	T =	20			
Run-Off Coefficient :	C =	0.392	**Pre-Development**		
Kerby 'r' value :	r' =	0.225			
Run-Off Coefficient :	C =	0.432	**Post-Development**		
Kerby 'r' value :	r' =	0.215			

Table 10:5 Development Criteria - Catchment N˚ 12

10.2 Surface Run-Off Results

10.2.1 CATCHMENT N˚ 5 – Pre Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m^3 / s)	(m^3 / s)	(m^3 / s)
1 : 2 year	0.13	2.04	3.74
1 : 5 year	**0.22**	**3.67**	**6.32**
1 : 10 year	0.29	5.10	8.26
1 : 20 year	**0.36**	**6.67**	**10.21**
1 : 50 year	0.45	8.82	12.78
1 : 100 year	0.52	10.69	14.73

Table 10:6 Pre-Development Run-off Results

10.2.2 CATCHMENT N˚ 5 – Post Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m^3 / s)	(m^3 / s)	(m^3 / s)
1 : 2 year	0.15	2.25	4.13
1 : 5 year	**0.24**	**4.04**	**6.96**
1 : 10 year	0.32	5.62	9.11
1 : 20 year	**0.40**	**7.35**	**11.25**
1 : 50 year	0.50	9.72	14.09
1 : 100 year	0.57	11.79	16.23

Table 10:7 Post-Development Run-off Results

10.2.3 CATCHMENT N˚ 6 – Pre Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m³ / s)	(m³ / s)	(m³ / s)
1 : 2 year	0.21	3.39	5.72
1 : 5 year	**0.35**	**6.10**	**9.65**
1 : 10 year	0.45	8.48	12.62
1 : 20 year	**0.56**	**11.10**	**15.60**
1 : 50 year	0.70	14.67	19.53
1 : 100 year	0.81	17.79	22.50

Table 10:8 Pre-Development Run-off Results

10.2.4 CATCHMENT N˚ 6 – Post Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m³ / s)	(m³ / s)	(m³ / s)
1 : 2 year	0.23	3.80	6.31
1 : 5 year	**0.38**	**6.84**	**10.64**
1 : 10 year	0.50	9.51	13.91
1 : 20 year	**0.62**	**12.44**	**17.19**
1 : 50 year	0.77	16.45	21.52
1 : 100 year	0.89	19.95	24.80

Table 10:9 Post-Development Run-off Results

10.2.5 CATCHMENT N˚ 8 – Pre Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m³ / s)	(m³ / s)	(m³ / s)
1 : 2 year	0.11	1.71	3.76
1 : 5 year	**0.18**	**3.07**	**6.34**
1 : 10 year	0.24	4.27	8.29
1 : 20 year	**0.30**	**5.59**	**10.25**
1 : 50 year	0.37	7.39	12.83
1 : 100 year	0.43	8.96	14.78

Table 10:10 Pre-Development Run-off Results

10.2.6 CATCHMENT N˚ 8 – Post Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m^3 / s)	(m^3 / s)	(m^3 / s)
1 : 2 year	0.12	1.91	4.14
1 : 5 year	**0.20**	**3.44**	**6.99**
1 : 10 year	0.27	4.79	9.14
1 : 20 year	**0.33**	**6.26**	**11.29**
1 : 50 year	0.41	8.28	14.14
1 : 100 year	0.47	10.04	16.29

Table 10:11 Post-Development Run-off Results

10.2.7 CATCHMENT N˚ 11 – Pre Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m^3 / s)	(m^3 / s)	(m^3 / s)
1 : 2 year	1.15	20.84	26.80
1 : 5 year	**1.94**	**37.51**	**45.21**
1 : 10 year	2.54	52.13	59.14
1 : 20 year	**3.14**	**68.19**	**73.06**
1 : 50 year	3.93	90.12	91.48
1 : 100 year	4.58	109.30	105.40

Table 10:12 Pre-Development Run-off Results

10.2.8 CATCHMENT N˚ 11 – Post Development Modelling Results

Return Storm Period	Unit Hydrograph Method	Rational Method 'Kerby'	Rational Method 'Emperical'
	(m^3 / s)	(m^3 / s)	(m^3 / s)
1 : 2 year	1.27	23.37	29.54
1 : 5 year	**2.14**	**42.05**	**49.83**
1 : 10 year	2.80	58.44	65.18
1 : 20 year	**3.46**	**76.45**	**80.53**
1 : 50 year	4.33	101.03	100.82
1 : 100 year	5.05	122.54	116.17

Table 10:13 Post-Development Run-off Results

10.2.9 CATCHMENT N˚ 12 – Pre Development Modelling Results

Return Storm Period	Unit Hydrograph Method (m³ / s)	Rational Method 'Kerby' (m³ / s)	Rational Method 'Emperical' (m³ / s)
1 : 2 year	0.10	1.42	2.76
1 : 5 year	**0.18**	**2.56**	**4.66**
1 : 10 year	0.23	3.56	6.10
1 : 20 year	**0.28**	**4.66**	**7.54**
1 : 50 year	0.35	6.15	9.43
1 : 100 year	0.41	7.46	10.87

Table 10:14 Pre-Development Run-off Results

10.2.10 CATCHMENT N˚ 12 – Post Development Modelling Results

Return Storm Period	Unit Hydrograph Method (m³ / s)	Rational Method 'Kerby' (m³ / s)	Rational Method 'Emperical' (m³ / s)
1 : 2 year	0.11	1.60	3.05
1 : 5 year	**0.19**	**2.88**	**5.14**
1 : 10 year	0.25	4.00	6.72
1 : 20 year	**0.31**	**5.23**	**8.31**
1 : 50 year	0.39	6.91	10.40
1 : 100 year	0.45	8.38	11.98

Table 10:15 Post-Development Run-off Results

11 STORMWATER MANAGEMENT

11.1 Proposed Site Drainage

11.1.1 Introduction

The 'Project Specification' makes reference to three methods in which the proposed PV panels are to be fixed to the ground, i.e. 'Fixed Tilt, Single Axis tracking or Dual Axis Mounting Structures. The fixing method will play a role in how the site surface stormwater should be managed. For the purposes of this report, we have assumed the system that will have the most impact on the site stormwater, i.e. Fixed Tilt Mounting Structures.

11.1.2 Site Preparation

Taking into account the average flat slope of the site and the typical vegetation present, the management of the post development stormwater surface flow will generally flow towards the low points of each catchment, adjacent and bordering the site as shown in Figure 10:1 above, however, the possibility of ponding in localised areas as a result of the flat slopes is a possibility.

We therefore propose TWO possible site preparation alternative options for this PV plant:-

11.1.2.1 *OPTION 1: Site Terracing*

This proposed option is to construct multiple sloping terraces across the site to ensure all surface flow run off can be directed effectively and efficiently to the newly proposed on-site detention facilities. In addition, it is proposed that the site is terraced accordingly such that the new terrace surfaces slope between 1% and 1.5% to ensure good surface run-off flows. The use of these slope percentages over large areas will in addition, assist with run-off erosion taking into account that the new terraces will not be covered with any vegetation or hard covering surface.

It should be noted that the terracing of the site will assist with the uniform positioning of the solar panels, site roads and the on-site stormwater system.

As a result of the vegetation being cleared and terraces being formed, the total volume of stormwater run-off emanating off the cleared / terraced area would increase and would therefore require stormwater management in the form of surface stormwater channels and chutes to direct and control the stormwater overland flows definitively to the proposed detention ponds for attenuation and release.

It should be noted that the design extent and slope direction of each terrace should be such that each set of terraces can be drained into individual detention facilities. The management of the surface stormwater from two proposed terraces into one detention pond is not recommended as it will result in a very large pond holding structure.

This option is no doubly the most expensive and could render the project unfeasible. However, the management of any surface stormwater will be effective.

11.1.2.2 *OPTION 2: Natural Site*

This option is considered the most environmentally conscious and cheapest option available, however, it could pose problems insofar as not having a structured surface stormwater management plan. Therefore, particular attenuation must be given at preliminary design stage to incorporate existing and new natural elements to form part of the stormwater management plan.

The natural contouring of the site does allow for surface flow control as proposed however, the natural gradient of the site is extremely flat and can create localised ponding after rain storms which will in all likelihood remain and evaporate as opposed to run-off.

The overland surface run-off will migrate perpendicular to the existing contours towards the adjacent gravel roads where it will be directed and controlled in open channels and chutes towards the low points of each catchment. Located in each low point will be detention facilities to attenuate and release the post development overland flow to contain silt, debris and / or litter.

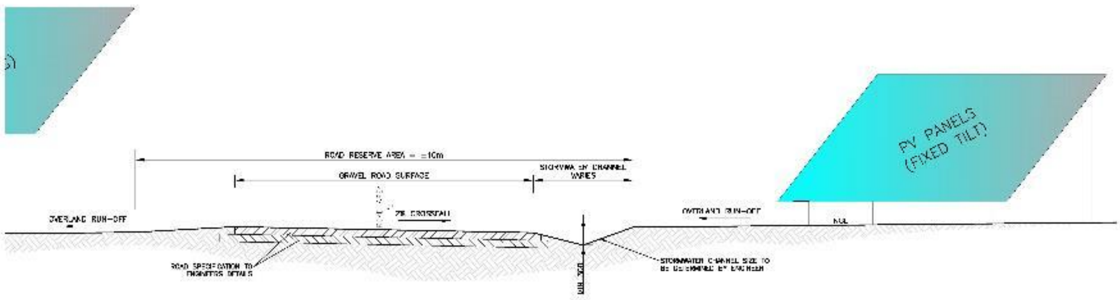

Figure 11:1 Typical Gravel Road & Cut-Off Drain

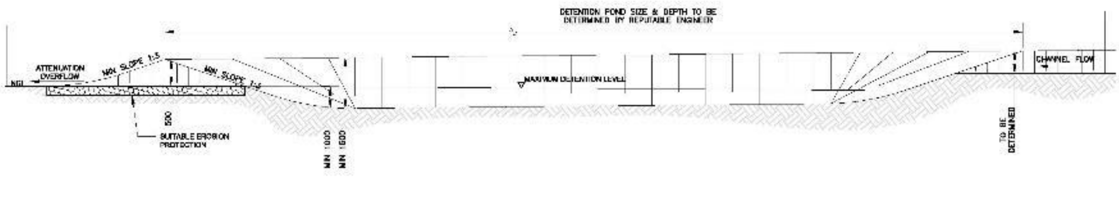

Figure 11:2 Typical Detention Pond Detail

It should be noted that the site left in its natural state, void of impeding vegetation, will require on-going clearing of vegetation in the event the vegetation hampers the movement of the rotating PV panels and the flow of stormwater.

This option remains the most cost effective and cheaper option but could compromise the management of the surface stormwater and hence will require ongoing management as part of the Environmental Management Plan (EMP).

11.2 Terrace Road Drainage

In the event Option 2 above is selected, the proposed road layout will remain as is, however, in the event Option 1, terracing of the site is selected, the positioning of the proposed site roads will be determined by the terraces and their respective levels.

Furthermore, it is proposed that all site roads remain un-surfaced (gravel), with only the section of main access road into the plant from the existing road, surfaced if required. The requirement on whether or not the access road should be surfaced, should be guided by the Transportation Impact Assessment and more particular, the number and type of vehicles expected on the entire development.

To assist with the stormwater run-off, these gravel roads should typically be graded and shaped with a 2% crossfall back into the natural slope, allowing stormwater to be channelled in a controlled manor towards the, natural drainage lines or constructed earth channels and to assist with any sheet flow on the site.

Where any proposed roads, intersect the natural, defined drainage lines, it is suggested that either suitably sized pipe culverts or drive through causeways are installed / constructed and should take into account the hydrology criteria for a selected major storm as outlined in sections 10.2.1 – 10.2.10 above.

11.3 Conveyancing of Site Stormwater

It is the intention to construct earth 'V' drains running parallel to all internal roads to act as a cut-off to the surface flow. We note that all earth drains within a sub-catchment will be linked if required and directed towards the designated point of discharge.

Where new internal gravel roads cross defined drainage lines, it is proposed that pipe culverts and low level causeways be constructed to ensure the migration of stormwater flow across the road and reducing the possibility of erosion.

12 CONCLUSIONS & RECOMMENDATIONS

In summary, the management of the expected surface stormwater for both the proposed Options above can be done with minimal problems.

The only variable to be considered is whether to keep the 'Natural Site' and allow the natural stormwater surface flows to prevail, but managed in a controlled manor or to 'Terrace' each site and definitively control the surface stormwater flows.

It should be noted that the control and effective management of any surface stormwater can only take place if the site areas are prepared accordingly e.g. terracing. However, as a resultant of the gradual slopes of the natural contours and the low MAP, effective management can be achieved without the use of terracing and with inventive use of overland flow management in cut off roads, berms, drains, channels and chutes towards attenuation facilities.

Therefore, it is our recommendation that a Natural Site be considered as far as possible to avoid financial constraints and allow for the effective stormwater management, although to a limited extent.

Furthermore, it should be noted that an indicative PV site layout has been proposed for the process of identifying any possible constraints associated with the management of stormwater on the site and that once the final layout of the site has been concluded, a re-evaluation of this proposal be completed to ensure alignment with the EMP.

The site therefore is not considered problematic, however, it is noted that no provisions have been made to cater for the overland major storm occurrences, i.e. Q50 & Q100 durations.

The future, and on-going management of the detention facilities will be required to ensure the full potential of the facility is available at all times so as to be effective and to ensure the objectives are achieved. This will mostly entail the cleaning of the pond free of silt, debris and / or litter.

Refer Appendix ‘C’ – TYPICAL DETAILS (DWG No: 16145-5302-Rev A)

APPENDIX A: GEOTECHNICAL REPORT

GEOTECHNICAL INVESTIGATION FOR THE PROPOSED DYASONSKLIP AND SIRIUS SOLAR FARMS:

MARCH 2016

Prepared by:

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Form 403.1 Rev 11

VERIFICATION PAGE

GEOTECHNICAL INVESTIGATION FOR THE PROPOSED DYASONSKLIP AND SIRIUS SOLAR FARM SITE			
JOB NO.: 4068	DATE: MARCH 2016		STATUS: FINAL REPORT

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SYNOPSIS:

GEOTECHNICAL INVESTIGATION FOR THE PROPOSED DYASONSKLIP AND SIRIUS SITE SOLAR FARM

KEY WORDS:

Geotechnical, geology, solar farm

QUALITY VERIFICATION

This report has been prepared under the controls established by a quality management system that meets the requirements of ISO9001: 2008 which has been independently certified by DEKRA Certification under certificate number 90906882

Verification	Capacity	Name	Signature	Date
Co-authored by	Geologist	Mr L Whitecross		March 2016
Co-authored by	Geologist	Mr T Hayes		March 2016
Co-authored by	Graduate Engineer	Mr J Olivier		March 2016
Checked by	Senior Eng Geologist	Mr RM Fyvie		March 2016
Authorised by	Executive Associate	Mr RM Fyvie		March 2016

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1 INTRODUCTION

This report presents the available results of the Detailed Geotechnical Investigation carried out for the proposed Dyasonsklip 1, Dyasonsklip 2 and Sirius Solar Farm infrastructure which is situated approximately 20km to 25km along the N14 from Upington towards Keimoes in the Northern Cape province of South Africa.

The scope of work included but was not limited to:

- Site walkover and visual assessment on each site.
- Desktop study focussing on the local topography, regional geology and hydrological conditions.
- The mobilisation of four (4) coring rigs to complete twenty-six (26) 'N' sized 5m boreholes on each site and three (3) 'N' sized boreholes in the location of the O&M Building and HV sub-station.
- The mobilisation of one (1) percussion rigs to complete twenty-six (26) 5m percussion boreholes and four (4) abstraction boreholes.
- The lithological and geotechnical logging of boreholes and collection of core samples and soil samples for analysis.
- Collection of three (3) water samples for analyses.
- Machine excavation of five (5) trial pits on each site to a maximum depth of 3m or refusal.
- Soil profiling and sample collection of disturbed samples from trial pits for analysis according to Jennings *et al* (1973): *Revised guide to soil profiling for Southern African Civil engineering purposes*.
- The mobilisation of one (1) Dynamic Penetrometer Super Heavy (DPSH) rig to conduct thirty (30) DPSH tests.
- Five (5) in-situ Electrical resistivity and five (5) in-situ Continuous Surface Wave (CSW) testing on each site.
- On-site supervision of operations and logging according to Brink & Bruins (2002): *Guidelines for soil and Rock logging in South Africa*.

Analyses and interpretation of results and field investigations are presented on potential geotechnical engineering problems, local and regional geology, suitability for on-site material being used in earthworks and recommendations for foundation design.

The ground conditions described in this report refer specifically to those encountered in the excavated trial pits and drilled boreholes. It is therefore quite possible that conditions

encountered in the proximity of the construction sites may vary to those encountered in the trial pits and boreholes. The information in this report is given in good faith, as an indication of materials and conditions likely to be encountered during construction. Any opinions and interpretations expressed are given as a guide only. There is no warranty that the information is totally representative of the whole investigation area and no responsibility will be accepted for any consequences arising from actual conditions being different from those indicated in this document.

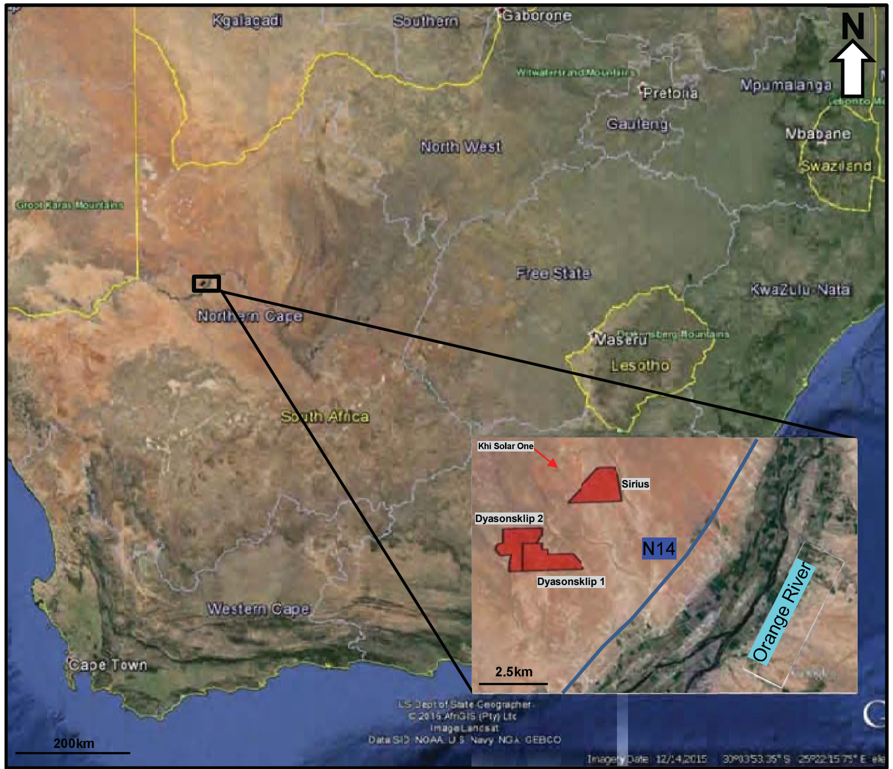
2 TERMS OF REFERENCE

Jeffares & Green (Pty) Ltd was invited by *Scatec Solar* to prepare a proposal and cost estimate for a geotechnical investigation on the proposed Dyasonsklip 1, Dyasonsklip 2 and Sirius Solar Farm sites, situated between the towns of Upington and Keimoes. Appointment was received from Scatec Solar on the 20 October 2015 with field work to commence on the 04 November 2015.

3 SITE DESCRIPTION

3.1 Locality

The Dyasonsklip 1, Dyasonsklip 2 and Sirius sites are located approximately 20 to 25km from Upington along a well maintained N14 road towards Kiemoes. The Sirius solar farm site is accessed by travelling approximately 17km from Upington westwards along the N14 towards a local farm gate and then travelling approximately 4km northwards along a dirt road. The Dyasonsklip 1 and Dyasonsklip 2 sites can be accessed by travelling 22km westwards along N14 and then approximately 3km northwards through a local farm gate and dirt road.



Jeffares & Green	REGIONAL LOCALITY PLAN	SCALE Approximate Scale provided in Image

Figure 3.1: Regional Locality Map for the Dyasonsklip and Sirius Solar farm Geotechnical Investigation

3.2 Topography and Land Use

The topography of the area is gently undulating to flat with a very gentle slope towards the Orange River, minor incisions occur in the vicinity of dry stream beds. Current land use within the area consists of small scale livestock farming.

3.3 Climate

The climate of the area is considered as arid with an annual average rainfall of 189mm predominantly falling between October and May with March being the wettest month. Average Maximum temperatures range from 21° to 36° Celsius, with temperatures often exceeding 40° Celsius however (South African Weather service). There is a high variance to the average means supplied. The vegetation is typical of a dry environment giving rise to Karoo type Orange River veld (Veld types of South Africa).

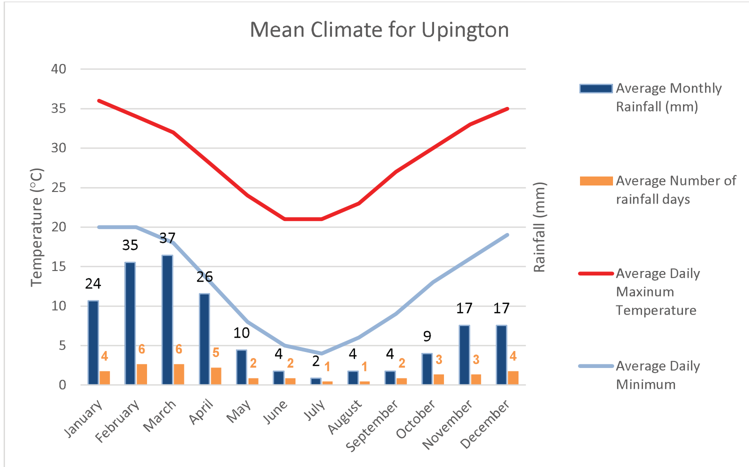


Figure 3.2: Average climate for Upington (South African Weather Service, period 1961 to 1990)

4 GEOLOGY AND GEOHYDROLOGY

The Dyasonsklip 1, Dyasonsklip 2 and Siruis sites fall in an area with a complex geology in the far western portion of the Areachap Group of the Namaqua-Natal Metamorphic province, which is intruded by Keimos Suite granatoids and overlain by Quaternary Kalahari Group sediments and Tertiary calcretes.

Cornell *et al* (1992) and Geringer *et al* (1994) describe the geological history as follows:

1. A period of extensional tectonics resulting in the formation of an oceanic rift basin (1600Ma-1350Ma).
2. Formation of a subduction zone with island arc and back-arc basins (1320Ma – 1270Ma).
3. A period of compressive tectonics resulting in the formation of folds, thrusting, crustal thickening and associated medium-pressure metamorphism (1220Ma – 1210Ma)

4. Crustal melting of the lower crust, the resultant magma intruding as granites which caused compressional forces on the surrounding geology causing further folding and faulting with thermal and hydrothermal granulite metamorphism (~1100Ma)
5. Crustal uplift and erosion of material. Resulting cooling of the crustal material associated with uplift resulted in retrograde medium-pressure metamorphism (1080Ma – 965Ma)

Two types of Keimos Suite Granitoids are recognised in the site area:

- Foliated metamorphosed Granitoids ('Meta-granites' and Gneisses) – these represent older intrusions which were subjected to metamorphic processes related to the deformation history of the area.
- Unfoliated granitoids – these are younger in origin to the foliated granites, post-dating the deformational history of the area.

According to the 1:250 000 scale Geological Map 2820 Upington (Council of Geoscience, 1990) the area has the following formations:

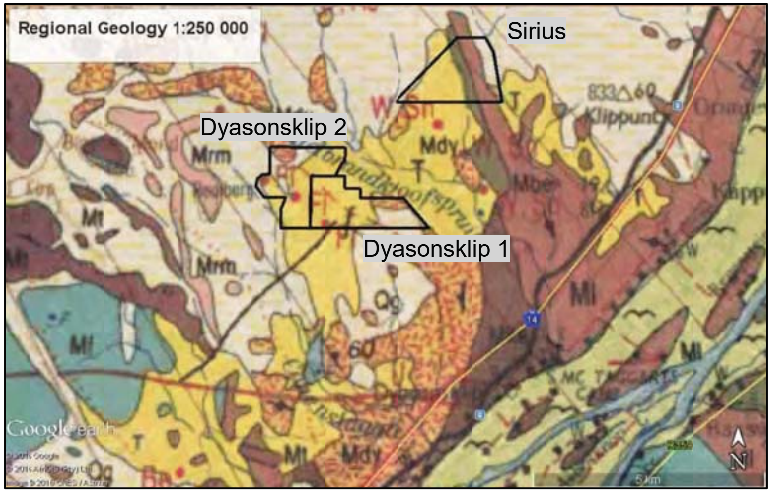


Figure 4.1: Approximate site positions on the 1:250 000 regional geological map.

- **Bethesda Formation** (Mbe) – Biotite rich garnet or metapelitic schists (Cornell et al).
- **Louisvale Granite** (Mi) – Light grey foliated metamorphosed granite.
- **Kanoneiland Granite** (Mka) – Medium grained foliated granite.
- **Dyasons Klip Gneiss** (Mdy) – Porphyroblatic to megacystic gneiss with 'brown-weathering'

- **Gordonia Formation** (Qg) – Aeolian dunes consisting of rounded quartz grains with hematite giving a red brown colour.
- **Mokolanen Formation** (T) – Pedogenic horizon representing chemical precipitate, reworking of material resulted in the formation of calcrete pebbles and cobbles intermixed with sandy material.

Due to the nature of the gneiss and granites described above, with particular reference to the foliation of the granite, the granites in the area are not 'true' granites and have undergone some degree of metamorphism. This adds to the complexity of describing intersected rocks with reference to the regional geology.

The following soil horizons and bedrock were encountered during the drilling program and trial pitting on a local scale:

- Alluvial Horizon (Gordonia Formation)

  Generally loose to medium dense red-brown poorly sorted silty sand to gravelly in places, trial pits indicate a depth of 0.10m to 0.50m (Diamond rotary boreholes are poor indicators on soil horizon depth as a result of the poor material recovery of unconsolidated material). Alluvial soils may contain reworked calcrete, granite, gneiss, and schist nodules.

- Pedogenic Horizon (Mokolanen Formation)

  Chemically precipitated Calcrete was encountered in the majority of the boreholes and trial pits across the sites. Calcrete is generally well cemented hardpan to nodular with a creamy white to grey colour.

- Granite (Kanoneiland Granite and Louisvale Granite)

  Generally fine to medium grained biotite rich granite containing quartz and feldspar. Granite may be poorly foliated in places and has been described as a 'meta-granite'.

- Gneiss (Dyasonsklip Gneiss):

  Generally fine to medium grained intensely foliated bands of feldspar and biotite rich layers. A strong tendency for the rock to break along foliation planes.

- Schist (Bethesda Formation):

  Generally fine to medium grained intensely to very intensely foliated with a strong tendency to break along foliation.

Weathering is variable ranging from moderately to completely weathered along fractures. The trial pits and shallow boreholes did not intersect unweathered material. Rock hardness exhibits a relationship with the weathering and ranges from hard to very soft along completely weathered fractures.

The regional geology is complex with multiple metamorphic and deformation events. Locally the geology is interpreted as follows (overlying reworked sediments and calcrete is not included, reference below is made to the underlying rock and its sub-outcrop):

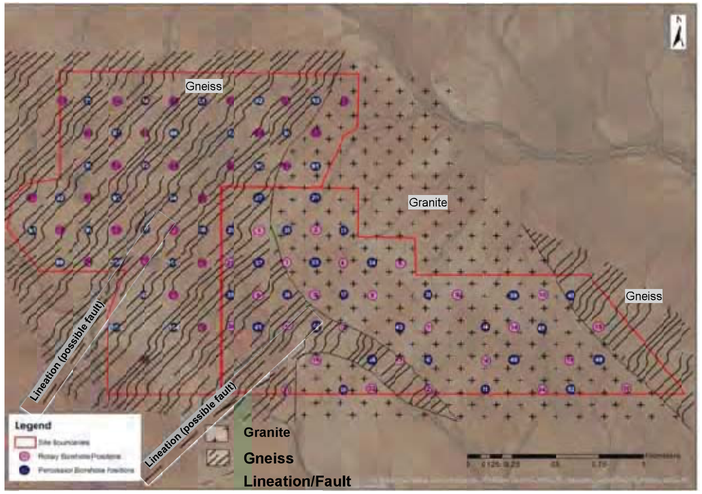


Figure 4.2: Revised local sub-outcrop geological map for Dyasonsklip 1 and 2.

Two dominant geological rock types were intersected on Dyasonsklip 1 and 2, Gneiss possibly from the Dyasonsklip Gneiss Formation and Granite possibly from the Kanoneiland Granite Formation. Two possible faults/lineations were interpreted with reference to the regional geological map and core retrieved from drilling. The possible faults trend roughly NNE/SSW and NE/SW. Their position seems confined to the southern portion of Dyasonsklip 1 and south western portion of Dyasonsklip 2 where the lithology is highly weathered and fractured. Granitic intrusion may occur and are not illustrated on the geological map due to a lack of data.

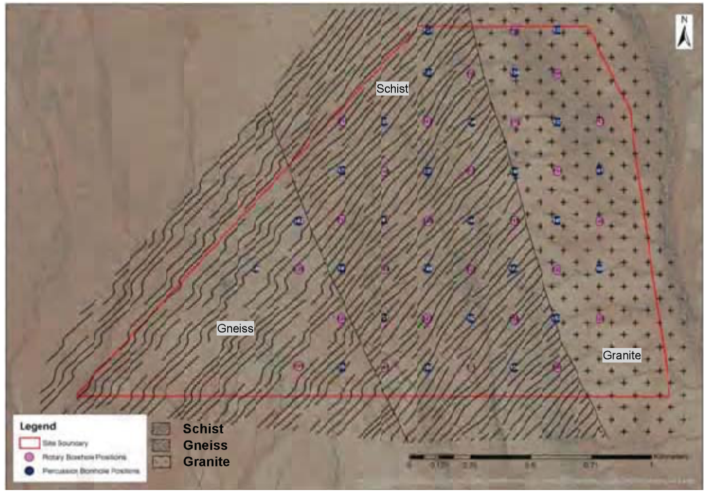


Figure 4.3: Revised local sub-outcrop geological map for Sirius

Three geological formations were intersected on Sirius; Gneiss possibly from the Dyasonsklip Gneiss Formation, Schist possibly from the Bethesda Formation and Granite possibly from the Louisvale Granite formation. Similarly to the Dyasonsklip 1 and 2, granitic veins may occur but are not illustrated on the revised geological map due to limited intersection.

According to the 1:3 000 000 hydrogeological map published by the *Department of Water Affairs and Forestry*, boreholes within the investigation area potentially yield <0.4 litres/second with a maximum volume of 2500 – 4000 m^3/km^2/annum being abstracted while preserving sustained abstraction within the aquifer system.

5 Seismicity

According to the 1:6 000 000 Seismic Hazard Map of Southern Africa, the site falls within a level five area on the Modified Mercalli Scale (MMS). Peak horizontal ground acceleration of <50 cm/s^2 has been recorded, with a 10% probability of this being exceeded at least once in a 50 year period.

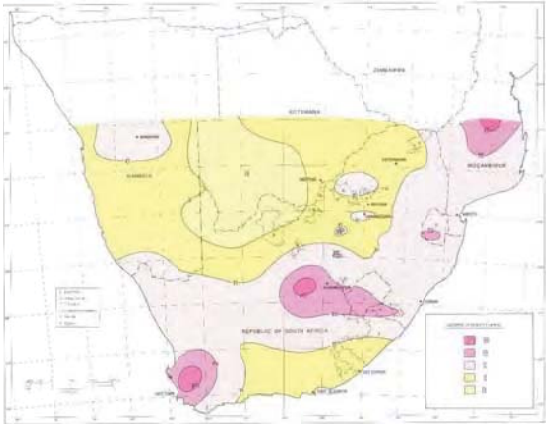


Figure 5.1: Siesmic intensity for Southern Africa

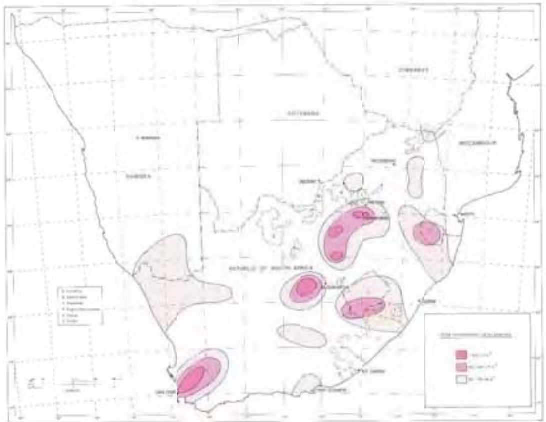


Figure 5.2: Peak horizontal ground acceleration

A detailed seismic normative study on the site was not requested by Scatec Solar, nor required by best practice owing to the low seismic risk for the site.

6 INVESTIGATION METHODOLOGY

The field investigation took place from 05 November 2015 to 11 December 2015 by Jeffares & Green Geologists set out by Jennings *et al* (1973): *Revised guide to Soil profiling for Southern African Civil engineering purposes* and Brink & Bruins (2002): *Guidelines for soil and rock logging in South Africa*. *Diabor Geotechnical and Exploration Drilling* conducted all drilling and DPSH operations under the supervision of a Jeffares & Green geologist. CSW and Electrical resistivity was conducted by G.Heymann CC and ASST (Pty) Ltd respectively. All relevant positions were recorded using a Garmin etrex handheld GPS.

Field investigations were conducted on predetermined locations which were best aligned with the final solar panel locations. For consistency the results and recommendations are presented for each site with reference to geological domains and geotechnical properties.

6.1 Boreholes

Twenty-six (26) shallow ~5m diamond rotary boreholes were drilled on a ‘best fit’ grid to cover the areas on each of the licenses, while three (3) deeper ~25m diamond rotary boreholes were drilled within the vicinity of the O&M Building and HV sub-station.

All boreholes were logged and sampled by *Jeffares & Green* Geologist, soil conditions and rock strata described using standard methods and terminology outlined by Jennings et al. (1973) and the Core Logging Committee of South Africa (1976).

Selected core samples were taken from the diamond rotary boreholes for Point Load, Unconfined Compressive Strength (UCS), Foundation Indicator and Moisture Content tests while one water sample was collected from each site for analysis. Water samples were taken from the deeper ~25m boreholes and sent for analysis to determine the aggressivity of the groundwater.

Twenty-six (26) ~5m percussion boreholes were drilled on a ‘best fit’ grid on each of the licenses, the rotary and percussion grids were positioned to create an alternating sequence of rotary and percussion boreholes. Percussion chips were logged and used to confirm geological interpretations.

The borehole log descriptions and photographic plates are presented in Annexure B and C while the co-ordinate positions of all boreholes are illustrated in Annexure I.

6.2 Trial Pits

Trial pits were machine excavated at predetermined positions towards the outer limits of the license by means of a Tractor-Loader-Backhoe (TLB).

Five (5) trial pits were excavated to a maximum of 3m or refusal on each site, with an attempted 0.6 x 1.5 m maximum excavation width and length, respectively. All trial pits were immediately profiled by *Jeffares & Green* Geologists, soil conditions were described according to standard methods and terminology outlined by Jennings et al. (1973).

Disturbed samples were taken from selected trial pits for Modified AASHTO and California Bearing ratio, Soil Aggresivity, Thermal Conductivity, Falling Head Permeability, Double Hydrometer, Moisture Content and Foundation Indicator testing. Trial pit profiles and sampling summary are included in Annexure D while the co-ordinate positions of all trial pits are illustrated in Annexure D.

6.3 Dynamic Penetrometer Super Heavy (DPSH) Tests

Dynamic Penetrometer Super Heavy (DPSH) testing was conducted using non-propellant trailer mounted rig moved using a LDV vehicle. DPSH testing is useful for establishing *in-situ* density conditions at depths exceeding 2.00m below ground level. Thirty (30) DPSH tests were conducted across the three (3) sites, however showed low levels of penetration and generally shallow refusal.

DPSH testing involves the knocking of a cone (50.5 mm diameter), attached to a rod (32 mm diameter), into the ground from a drop height of 0.75 m using a vertical falling weight (63.5 kg). DPSH testing yields results equitable to SPT ''N'' values.

The ease of cone penetration during DPSH testing can be affected by the moisture content of the soil and therefore is affected by recent climate, heavy rainfall will saturate the soil horizon providing a lower set of results than a similar test in the dry season. Moisture content at the time of testing is therefore recorded together. Potential perched water table conditions, especially where bedrock is close to surface, should not be discounted. DPSH testing can also be affected by obstacles in the soil horizon that may not be representative of the overall area

6.4 Electrical Resistivity Testing

Fifteen (15) traverses were surveyed across the sites with Electrical Resistivity Imaging at predetermined positions to investigate the electrical properties of subsurface profile. The *Wenner Array* method was utilised which requires four equidistant electrodes to be placed into the ground. A known current is applied to the outer electrodes with the voltage recorded between the inner two electrodes.

6.5 Continuous Surface Wave Seismic Testing

Seismic surveys were conducted across five (5) predetermined positions on each site, to establish the likely rock conditions present at the positions.

The results of the CSW survey are important for ascertaining the shear wave velocities of the material, from which a number of geotechnical parameters may be derived.

7 DYASONSKLIP 1 FIELD INVESTIGATION AND DEVELOPMENT RECOMMENDATIONS

7.1 Trial pitting

At TP01 an alluvial horizon was profiled as *dry,* light red brown, *loose to medium dense, intact,* silty gravelly SAND with roots to a depth of 0.15m. The alluvial horizon overlays a pedogenic horizon described as *dry*, creamy white with grey speckles, *medium dense to dense, intact,* CALCRETE **–** nodular to hardpan with cobbles of gneiss intermixed with silty sand. Trial pit was profiled to a termination depth of 0.60m.

At TP02 an alluvial horizon was profiled as *dry,* light reddish brown, *loose, intact,* gravelly silty SAND with nodules of calcretes of cobble size to a depth of 0.30m. This gives way to a pedogenic horizon profiled as *dry,* creamy white, *medium dense becoming dense, intact,* hardpan CALCRETE. The trial pit was profiled to a termination depth of 0.85m.

At TP03 an alluvial horizon was profiled as *dry,* light red brown, *loose, intact,* cobbles of calcretes and granite with silty gravelly sand. This horizon overlies a pedogenic horizon profiled as *dry,* creamy white, *medium dense to dense, intact,* hardpan CALCRETE. The trial pit was profiled to a termination depth of 0.70m.

At TP04 an alluvial horizon was profiled as *dry,* light red brown, *medium dense, intact,* silty sandy GRAVEL to a depth of 0.50m. This overlies a pedogenic horizon profiled as *dry,* creamy white with grey speckling, *medium dense becoming dense with depth, intact,* hardpan CALCRETE with minor silty sand. The trial pit was terminated at a depth of 0.75m.

At TP05 an alluvial horizon was profiled as *dry,* light red brown, *medium dense, intact,* gravelly silty SAND with roots to a depth of 0.25m. The alluvial horizon extends further to 0.45m and was profiled as *dry,* light red brown, *medium dense, intact,* COBBLES of gneiss with silty sand and minor calcretes nodules. The alluvial horizon overlies a pedogenic horizon profiled as *dry,* creamy white, *medium dense to dense, intact,* hardpan CALCRETE with cemented cobbles of gneiss. The trial pit was terminated at a depth of 0.75m.

7.2 Electrical Resistivity Testing

Interpretations are quoted from the report compiled by ASST, reference should be made to the full report attached in Annexure E. ASST note that there is a general relationship with colour variation visible on Google Earth Imagery, with lighter zones generally recording higher resistivities in relation to darker areas. Resistivity profiles across Dyasonsklip 1 indicate varying resistivities both laterally and vertically. An upper higher resisitivity layer with lower resistivity "pockets" were encountered, this is generally underlain by material with lower resistivities which in turn gives way to higher resistivities with depth.

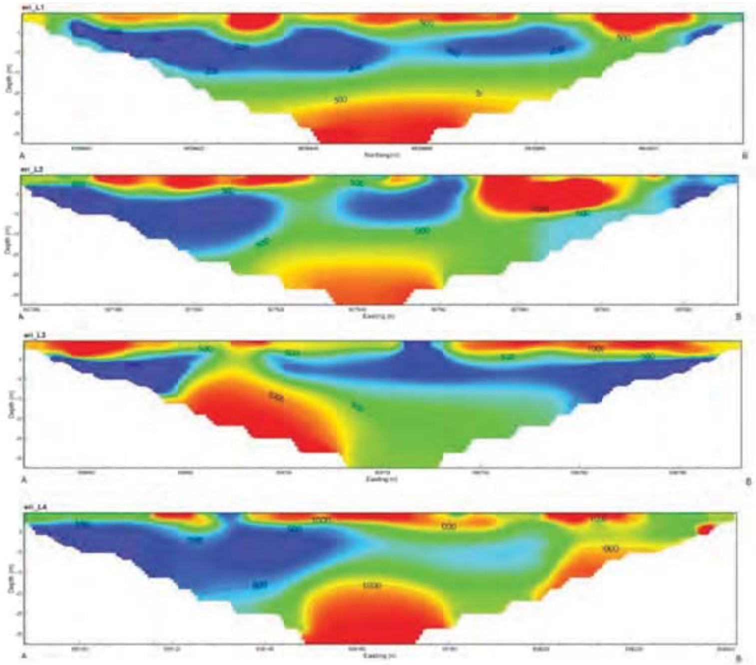


Figure 7.1: Electrical resistivity profiles for the five test posiions across Dyasonsklip 1.

7.3 Continuous Surface Wave Seismic Testing

Five (5) CSW seismic tests were conducted on the Dyasonsklip 1 site. The tests were conducted by G. Heymann CC and the full results are included in Appendix F with results utilised in the recommendation section.

7.4 Laboratory Results

Twenty five (25) samples were taken from Dyasonsklip 1 for point load analysis

Table 7.1: Point Load Test Results

Sample No.	BH No.	Depth	Description	Point Load Index	UCS (Mpa)
BH01A	BH01	3.53 - 3.68	Gneiss	0.34	8.1
BH03A	BH03	3.04 - 3.14	Granite	0.19	4.5
BH04B	BH04	4.52 - 4.68	Gneiss	0.15	3.7
BH05A	BH05	3.10 - 3.23	Gneiss	1.08	26
BH06A	BH06	3.06 - 3.17	Gneiss	0.30	7.2
BH07A	BH07	2.72 - 2.82	Granite	0.17	4
BH08A	BH08	3.99 - 4.12	Granite	0.36	8.5
BH09A	BH09	2.83 - 2.98	Granite	2.86	68.6
BH10A	BH10	1.70 - 1.81	Granite	3.03	72.8
BH11A	BH11	3.50 - 3.63	Gneiss	0.39	9.4
BH12B	BH12	4.18 - 4.30	Gneiss	0.37	8.9
BH13A	BH13	3.38 - 3.57	Granite	0.82	19.8
BH14A	BH14	3.17 - 3.30	Granite	0.35	8.5
BH15B	BH15	3.45 - 3.68	Gneiss	4.29	103
BH16A	BH16	2.04 - 2.16	Granite	0.30	7.3
BH17A	BH17	3.31 - 3.45	Gneiss	0.35	8.4
BH18A	BH18	4.20 - 4.37	Granite	8.16	195.9
BH19A	BH19	3.98 - 4.09	Granite	1.44	34.7
BH20B	BH20	2.84 - 3.00	Granite	6.07	145.8
BH21A	BH21	2.78 - 2.89	Gneiss	0.35	8.5
BH22A	BH22	1.88 - 1.99	Granite	4.69	112.4
BH23A	BH23	4.64 - 4.84	Gneiss	0.44	10.6
BH24A	BH24	3.00 - 3.19	Granite	1.15	27.6
BH25A	BH25	4.94 - 5.12	Gneiss	4.90	117.6
BH26A	BH26	2.67 - 3.00	Gneiss	1.79	42.9
BHDG1A	BHDG1	6.10 – 6.22	Gneiss	1.33	32.0

Results of the Point Load tests on core retrieved from Dyasonsklip 1 indicates a strength range from 3.7 MPa and 195.9 MPa, with the majority of the results below 10 MPa.

Table 7.2: UCS Test Results

Sample No.	Trial pit No.	Depth (m)	Description	Specimen Test Results		
				Failure Load	Strength (UCS)	Failure Code
BH04A	BH04	2.44 - 2.61	Gneiss	40	18.9	3B
BH06B	BH06	4.83 - 4.96	Gneiss	13	6.2	2B
BH09B	BH09	4.83 - 5.05	Granite	79.6	37.5	XA
BH10B	BH10	3.96 - 4.09	Granite	43.5	20.3	2B
BH12A	BH12	3.39 - 3.53	Gneiss	39.5	18.9	0B
BH15A	BH15	4.75 - 4.89	Gneiss	158.3	74.1	XA
BH19B	BH19	4.87 - 5.00	Granite	91.3	42.9	XA
BH22B	BH22	3.27 - 3.50	Granite	73	34.6	XA
BH25B	BH25	3.17 - 3.49	Granite	112.6	52.8	2B
BH26B	BH26	3.00 - 3.28	Gneiss	357	168.3	YA
BHDG1B	BHDG1	7.67 – 7.82	Gneiss	35.6	16.8	0B

Eleven (11) samples were collected on Dyasonsklip 1 for UCS testing with a UCS strength range from 6.2MPa to 168.3 MPa. The majority of the samples exhibited a sliding shear failure, where failure was influenced by discontinuities there was a partial failure from 0° - 30° to the axial plane.

Table 7.3: Atterberg Limit Detemination Test Results on samples collected from rotary boreholes

Sample No.	BH number	Depth (m)	Description	Soil Mortar Analysis (%)					Atterberg Limits (%)			GM	Potential Expansion
				Coarse Sand	Coarse Fine sand	Medium Fine Sand	Fine Fine Sand	Passing 0.075 mm	LL	PI	LS		
BH10C	BH10	0.00 - 0.95	Silty gravelly sand	64	6	6	7	17	24	7	2.5	1.92	LOW
BH20A	BH20	0.75 - 1.50	Silty sand	47	11	12	9	20	-	SP	1.0	1.61	LOW

LL - Liquid Limit GM - Grading Modulus NP - Non Plastic

LS - Linear Shrinkage PI - Plasticity Index SP – Slightly Plastic

According to the Soil Mortar Analysis, Coarse Sand and material passing 0.075 mm generally formed the bulk component of the sampled material. Plasticity Indices (PI) were *Slightly Plastic* to 7 % and Linear Shrinkage (LS) values were 1.0% - 2.5 %, values above 8.0 are considered to be problematic for moisture related heave and shrinkage. Potential expansion predictions indicate that all of the samples are contained within the *LOW* risk category for potential heave.

Table 7.4: Atterberg Limit Detemination Test Results on samples collected from trial pitting

Sample No.	Trial Pit	Depth (m)	Description	Particle size distribution %				Atterberg Limits (%)			Potential Expansion
				Gravel	Sand	Silt	Clay	LL	PI	LS	
TP01	TP01	0.15 - 0.60	Calcrete with nodules of gneiss intermixed with silty sand	71	23	5	1	19	3	1.7	LOW
TP03	TP03	0.00 - 0.45	Nodules of calcrete and granite intermixed with silty gravelly sand	76	21	2	1	22	6	2.9	LOW
TP05	TP05	0.25 - 0.45	Nodules of gneiss with silty sand and minor calcrete nodules	68	27	3	2	18	3	1.7	LOW

LL - Liquid Limit GM - Grading Modulus NP - Non Plastic

LS - Linear Shrinkage PI - Plasticity Index SP – Slightly Plastic

According to the Particle size distribution, Gravel generally formed the bulk component of the sampled material. Plasticity Indices (PI) were between 3% - 6 % and Linear Shrinkage (LS) values were 1.7% - 2.9 %, values above 8.0 are considered to be problematic for moisture related heave and shrinkage. Potential expansion predictions indicate that all of the samples are contained within the *LOW* risk category for potential heave.

Table 7.5: Mod/CBR

Sample No.	Trial Pit No.	Depth (m)	Description	MDD kg/m³	OMC %	CBR					Maximum Swell MAASHTO %	TRH 14 Classification
						90%	93%	95%	98%	100%		
TP01	TP01	0.15 - 0.60	Calcrete with nodules of gneiss intermixed with silty sand	2114	7.3	35	76	103	128	145	0.03	G5
TP03	TP03	0.00 - 0.45	Nodules of calcrete and granite intermixed with silty gravelly sand	2109	4.6	29	75	106	139	159	0.09	G5
TP05	TP05	0.25 - 0.45	Nodules of gneiss with silty sand and minor calcrete nodules	2272	5.2	33	62	81	136	174	5.1	G4

The results from the MOD/CBR tests are fairly conclusive in their representation of in situ materials sampled from the investigation site. The classification of material (G4) - (G5) reflects the composition and strength of the in situ soil. The tested material is of a good quality and may be suitable for use in fill, road / or structural applications depending upon final design requirements. All material classifications were assessed in accordance with TRH 14 specifications (1985).

Sample TP03 (0.00m – 0.45m) recorded a dispersion value of 50%, which represents intermediate to dispersive.

Table 7.6: Moisture Content

Sample No.	Borehole No.	Description	Moisture Content (%)
BH10C	BH10	Silty gravelly sand	6.7
BH20A	BH20	Silty sand	5
TP01	TP01	Calcrete with nodules of gneiss intermixed with silty sand	1.8
TP03	TP03	Nodules of calcrete and granite intermixed with silty gravelly sand	2
TP05	TP05	Nodules of gneiss with silty sand and minor calcrete nodules	1.4

Disturbed samples collected from trial pits and samples collected from the boreholes indicate a low moisture content, elevated moisture contents are expected for samples collected from boreholes as a result of the presence of drilling fluids.

Table 7.7: Falling Head Permeability result.

Sample Number	Depth (m)	Density kg/m^3	W %	H1 (cm)	H2 (cm)	Time			Permeability cm/s
						h	m	s	
TP05	0.25 - 0.45	2045	5.2	60	30.4	0	43	42	2.28E-04

The disturbed sample taken from trial pit TP05 recorded a permeability cm/s value of 2.28E-04.

Table 7.8: Soil Aggresivity

Sample No.	Description	pH	Stability pH at 20°C	Langelier Index at 20°C	Ryznar Stability Index at 20°C	Aggressiveness Index	Corrosivity Ratio	Final agressiveness index at 20°C
TP01	Calcrete with nodules of gneiss intermixed with silty sand	7.9	8.2	-0.3	8.5	10	0.3	319
TP03	Nodules of calcrete and granite intermixed with silty gravelly sand	7.8	8.2	-0.4	8.6	9.9	0.2	358
TP05	Nodules of gneiss with silty sand and minor calcrete nodules	7.9	8.2	-0.3	8.5	10	0.2	321

According to the corrosivity indices, the sample from TP01 is slightly corrosive towards concrete and metals while the Basson Index indicates the sample is non to mildly aggressive towards concrete. The corrosivity indices for TP03 indicate that the sample is slightly corrosive towards concrete but not corrosive towards metals while the Basson Index indicates the sample is mildly to fairly aggressive towards concrete. The corrosivity indices for TP05 indicate that the sample is slightly corrosive towards concrete but not corrosive

towards metals while the Basson Index indicates that the sample is non to mildly aggressive towards concrete.

Table 7.9: Thermal conductivity results for Dyasonsklip 1

Sample Number	Moisture %	Thermal Conductivity (K) W/m.k	Thermal Resistivity (ρ) °C.cm/W	Volumetric specific heat (C) $MJ/(m^3.K)$	Thermal Diffusivity (D) mm^2/s
TP03	0	0.291	344.7	1.201	0.243
	3	0.506	200.8	1.491	0.35
	7	0.775	135	1.551	0.504
TP05	0	0.371	272.2	1.557	0.239
	3	0.94	107	1.413	0.669
	7	1.311	52.99	2.332	0.83

Field based thermal resistivity testing was conducted on two (2) samples retrieved from the Dyasonsklip 1 site. Thermal conductivity ranged from 0.291 – 0.371 W/m.K at a moisture content of 0 %. Thermal conductivity shows an increase with increasing moisture content.

Table 7.10: Analytical report for water sample taken at Dyasonsklip 1.

Jeffares & Green

Deterioration of Concrete in Aggressive Waters (Basson)

Dysansklip and Sirius Solar Farms Farm

Determinant	Units	Method	Method No	Parameter	DG1		DG2		DG3		Screening Guidelines			
					Lab Result	Index	Lab Result	Index	Lab Result	Index	moderately aggressive	high aggressive	Scale Forming Tendency	Corrosive Tendency
Results														
pH at 25°C	pH units	SABS Method 11	1	V1	7.7	360	7.5	400	7.6	380	6-8	5-6		
pH_s at 25°C	pH units	PCI TM 9.28	calculation	V2	7.63	-140	7.63	260	7.66	120				
Langelier Index LI ($pH-pH_s$)		calc			0.07	0.07	-0.13	-0.13	-0.06	-0.06	-0.2 to -0.3	-0.3 to -0.4		
Calcium hardness	mg $CaCO_3$/l	SABS Method 216	calc	V3	52	985.6	80	924	70	946	>200	<200		
Ammonia	mg N/l	SABS Method 217/218	64	V4	0.4	4	3.56	35.6	3.66	36.6	<50	>50		
Dissolved magnesium	mg Mg/l	SABS Method 1071	9A	V5	8	4.8	11	6.6	9	5.4	<500	>500		
Sulphate	mg SO_4/l	SABS Method 212/1056	67	V6	233	69.9	106	31.8	63.7	19.11	<1000	>1000		
Chloride	mg Cl/l	SABS Method 202	16	V7	27	5.4	16	3.2	19	3.8	<1000	>1000		
Total Dissolved Solids at 180°C	mg/l	SABS Method 213	41	V8	1255		285		440					
Total Alkalinity	mg $CaCO_3$/l		10		341		187		219					
Dissolved calcium	mg Ca/l		8A		21		32		28					
Calculations														
Concrete Aggressiveness Method (Basson)		Leaching Corrosion Sub Index LCSI			401.9		528.0		482.0					
		Spalling Corrosion Sub Index SCSI			26.2		24.7		20.4					
		Aggressiveness Index N (standard local conditions)			**428.1**		**552.7**		**502.4**					
Final Index Description					mildy to fairly aggressive		mildy to fairly aggressive		mildy to fairly aggressive		<750	>750		
Recommendations	Leaching Corrosion Counter Measures				Class 1 concrete, inorganic coating		Class 1 concrete, inorganic coating		Class 1 concrete, inorganic coating					
	Spalling Corrosion Counter Measures				Class 3, 4 or 5 concrete, inorganic coating		Class 4 or 5 concrete, inorganic coating		Class 3, 4 or 5 concrete, inorganic coating					
	Chloride Corrosion Counter Measures for rebar				Minimum 25mm concrete cover over rebar		Minimum 25mm concrete cover over rebar		Minimum 25mm concrete cover over rebar					
Domestic Water Use Method		Langelier			0.1		-0.1		-0.1				>0	<0
		Ryznar			7.6		7.8		7.7				<6.5	>6.5
		Corrosion Ratio			5.4		1.7		1.3					>0.1
		Aggressive Index			11.9		11.7		11.8		10 to 12	<10		

	INDEX	DESCRIPTION
Final Index	0-350	non to mildly aggressive
	350-750	mildy to fairly aggressive
	750-1000	highly aggressive
	1000	very highly aggressive

The results of all three samples indicate mildly to fairly aggressive conditions, based on concrete aggressiveness method (Basson).

Finally based on the chloride corrosion index, the recommended concrete thickness around all rebar is a minimum of 25mm.

Resting water level at BHDG1 on Dyasonsklip 1 was 13.28m, this is below the recommended piling depth.

7.5 Foundation and Development Recommendations – Dyasonsklip 1

The pile installation methods considered for Dyasonsklip 1 include displacement steel piles, displacement ductile iron piles and pre-bored steel piles. Exosun is proposing to implement a Exotrack® HZ V2 solar panel system with the following pier configuration:

- Central pile:
  - 160x80x4 Rectangular hollow steel section,
  - IPE 200 (Equivalent to W8 x 10) I-section,
- End piles:
  - C170x60, or
  - IPE 160 (Equivalent to W6 x 9) I-section,

Due to the presence of calcrete, gravel, shallow bedrock, and abundant pebbles, cobbles and boulders, steel displacement piles will likely reach shallow refusal and could deform as a result. The DPSH probes reached refusal at shallow depths across the sites of Dyasonklip1. The SPT N-value readings generally exceed 20 before reaching refusal. Displacement steel hollow piles are generally unsuccessful in soils with a SPT N-value exceeding 20.

The alternatives are pre-drilling steel piles or circular hollow ductile iron displacement piles (TRM or similar approved). The latter has been used successfully at the Khi Solar One plant, approximately 2km north of the sites, where circular hollow ductile iron displacement piles (TRM piles) were driven, open ended, through the alluvial and "socketed" into the underlying bedrock. Similarly, pre-drilled steel piles could be founded in the underlying bedrock resulting in minor vertical settlement.

7.5.1 Loading

The loading imposed on the piles include the self-weight of the panels and piles, wind load imposed on the panels and the load applied by the daily tracking unit. The typical maximum loads applied on the piles were obtained from a document provided by the supplier titled "Exotrack HZ® foundations loads According to SANS 10160". The loads include the following:

- Vertical Direction:
  - $Fv_{central\_down}$ = 7000 N (downward load on central pile),
  - $Fv_{central\_uplift}$ = -2500 N (uplift load on central pile),
  - Fv_{end_down} = 6000 N (downward load on end pile),
  - Fv_{end_uplift} = -2500 N (uplift load on end pile),
  - $Fv_{motor\ support}$ = 400 N (downward on motor foundation).
- Horizontal Direction:
  - $Fh_{central}$ = 11 000 N (central pile),
  - $Mth_{central}$ = 2400 N.m (moment of central pile),
  - $Fh_{end\ pile}$ = 3000 N (end pile),
  - Fh_{motor} = 900 N (motor foundation).

These loads were applied to theoretical formulae in order to estimate the vertical settlement horizontal displacement rotation and minimum embedment depth of the piles. All calculations were carried out according to the Eurocode 7 Design Approach 1b standards.

7.5.2 Material Classification and Excavation Conditions

Excavation classes and rock classifications for the subsurface materials occurring across the investigated area have been estimated using the definitions provided in the SANS 1200D (1988) and SANS 1200F (1988) standards. The excavatability and rippability of the underlying material were assessed by evaluating borehole logs and the shear wave velocities obtained from the CSW tests. The S-wave velocities for the CSW tests conducted at Dyasonsklip1 is shown in Figure 7.2.

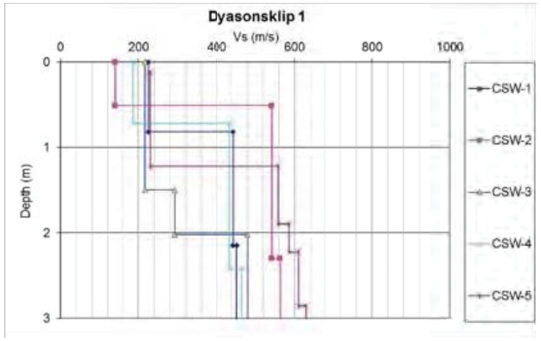


Figure 7.2: CSW test results at Dyasonsklip 1

Table 7.11 shows the excavation classes and rock classifications expected for the subsurface material across the site.

Table 7.11: Excavation Classes for the subsurface material at Dyasonsklip 1

Site	Depth (m)	S-wave Velocity (m/s)	Class of Excavation (SANS 1200D)	Rock Classification (SANS 1200F)
Dyasonsklip 1	0.0m – 0.5m	200m/s	Soft to Intermediate Excavation with Class A boulder excavation	-
	0.5m – 3.0m	400m/s< Vs< 600m/s	Intermediate Excavation with Class A boulder excavation between 0.5-1.5m	R2

7.5.3 Material Parameters and Assumptions

The particle size analysis of the alluvial material show that the horizon consists predominantly of a non-plastic sand, with abundant gravel, calcrete, cobbles and boulders in a dense consistency. According to the work done by Gibson (1953) a typical material with these characteristics will exhibit an internal friction angle (ϕ') of 30°.

The following assumptions were made in order to conduct further calculations:

- A unit weight of 18 kN/m^3 was assumed for the alluvium.
- The Poisson's Ratio (*v*) of the subsoils is in the order of 0.3

- The interface friction angle (δ') were assumed to equal to 0.75ϕ',
- The lateral earth pressure coefficient was assumed to be equal to the lateral earth pressure coefficient of earth pressure at rest, K/K_0=1.
- The piles were assumed to be open ended.

7.5.4 Bearing Resistance

The minimum pile embedment depth was calculated for the central and corner pile as a function of the base and shaft resistance. The shear strength of the subsurface material, the effective normal stress acting on the piles and the pile properties contribute to the overall bearing resistance of the subsurface material. The bearing resistance formulae as given by Knappet and Craig (2012) are as follow:

Base resistance:

$$Q_{bu} = A_p(N_q\sigma_q')$$

Shaft resistance:

$$Q_{su} = P_p\int_0^{L_p}\tau_{int}\,(z)dz$$

Where: A_p is the area of the pile cross-section,

N_q is a bearing capacity factor depended on the slenderness and friction angle of the subsoils,

σ_q' is surcharge above the pile base,

P_p is the pile perimeter,

L_p is the length of the pile.

τ_{int} is the shear strength along the interface of the pile and the soil.

7.5.5 Embedment Depth

The minimum embedment depth was calculated by assessing two conditions. For the first condition it is assumed that the bedrock is situated well below the pile base and the second condition when the pile base is situated within competent weathered bedrock.

Table 7.12 indicates the condition that the bedrock is situated well below the pile base, the minimum embedment depth ranges between 2.55m and 2.90m for limiting vertical loads of 7kN in the case of a central pile and 6kN in the case of end piles.

Table 7.12: Minimum embedment depth of pile founded in alluvium

Pile Section	Minimum Embedment Depth (m)	Compression Bearing Resistance ($Q_{bu}+Q_{su}$) (kN)	Tension Bearing Resistance (uplift wind load) (kN)
Central Pile			
Steel Hollow Pile 160 x 80 x 4mm	2.90	7.2	4.8
IPE 200	2.55	7.2	4.5
170 x 9 Circular Hollow (TRM)	2.60	7.0	3.6
End Pile			
IPE 160	2.60	6.0	3.9
C160 x 65	2.70	6.0	4.0
118 x 7.5 Circular Hollow Pile (TRM)	2.90	6.0	4.7

Table 7.13 indicates the second conditions where the pile base is founded within the competent bedrock the pull-out tension load will govern the embedment depth. The minimum embedment depth was found to range between 1.9m and 2.2m at a pull-out load limit of 2.5 kN.

Table 7.13: Minimum embedment depth of piles founded on weathered bedrock

Pile Section	Minimum Embedment Depth (m)	Compression Bearing Resistance ($Q_{bu}+Q_{su}$) (kN)	Tension Bearing Resistance (uplift wind load) (kN)
Central Pile			
Steel Hollow Pile 160 x 80 x 4mm	2.1	-	2.5
IPE 200	1.9	-	2.5
170 x 9 Circular Hollow (TRM)	2.2	-	2.5
End Pile			
IPE 160	2.1	-	2.5
C160	2.1	-	2.5
118 x 7.5 Circular Hollow Pile (TRM)	2.1	-	2.5

It is recommended that pile load tests are conducted at the onset of the construction phase in order to assess and verify the recommended embedment depths.

7.5.6 Settlement

The maximum estimated settlement was calculated in the event that the bedrock is situated well below the pile base level. The settlements were determined by applying the analytical techniques by Randolph and Wroth (1978) method and the T-z Finite Differences method. These methods require the follow input parameters:

- Young's Modulus (E) of the pile,
- Shear Modulus ($G_{0.1\%}$) distribution of the subsoils,
- Poison's Ratio of the surrounding soils (*v*)
- Dimensions of the piles

- Pile Force.

Young's Modulus (E) of a typical steel pile was selected to be 200 x 10^3 GPa, as per the Southern African Steel Construction Handbook (Sixth Edition, 2008). The maximum amount of vertical settlement will occur at the central pile as a result of a vertical load of 7000N.

The operative level of strain for a foundation structure is 0.1%. The shear modulus ($G_{0.1\%}$) values at 0.1% strain were determined by applying a softening function proposed by Rollins et. Al. (1998) to the shear modulus data, Figure 7.3, overleaf, shows the minimum estimated shear modulus ($G_{0.1\%}$) to a depth of 3m.

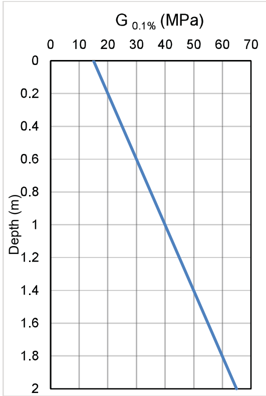


Figure 7.3: Minimum Shear Modulus of Subsoils

Figure 7.4 shows the pile load and settlement response to a depth of 3m. The total vertical settlement was estimated to be 1mm. The settlement of the piles is not sensitive to the marginal dimensional changes of the pile sections.

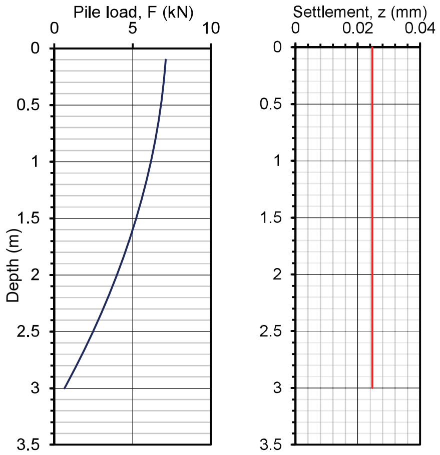


Figure 7.4: Settlement response for a pile to a depth of 3m

7.5.7 Horizontal Displacement and Rotation

The horizontal displacement was estimated by applying elasticity theory solutions. The horizontal displacement and rotation at the pile head, as given by Knappet and Craig (2012), are as follow:

Horizontal Displacement:

$$h = \frac{\left(\frac{E_p}{G_c}\right)^{1/7}}{p_c G_c}\left[\frac{H}{1.85L_c} + \frac{M}{0.83L_c^2}\right]$$

Rotation:

$$\theta = \frac{\left(\frac{E_p}{G_c}\right)^{1/7}}{p_c G_c}\left[\frac{H}{0.83L_c^2} + \frac{M}{0.16p_c^{-0.5}L_c^3}\right]$$

Where: E_p is Young's Modulus of the pile,

G_c is the median shear modulus,

p_c is the homogeneity factor describing shear modulus with depth,

L_c is the pile embedment depth,

H is the horizontal load, and

M is the moment applied around the head.

The horizontal displacement (h) at the pile head was calculated to be a maximum of 2mm with a rotation (θ) of 2°.

7.5.8 Motor support

The motor support foundation is required to carry a vertical dead load of approximately 400N and a horizontal wind load of approximately 900N. The load combination applies an eccentric stress to the subsurface soil. It was assumed that the foundation is situated on the alluvium with a horizon thickness of approximately 1.5m. A square foundation was assumed with a width (B) and length (L) of 1m.

The operative level of strain for a foundation structure is 0.1%. The settlement of the foundation is dependent on Young's Modulus (E) of the subsoils at 0.1% strain. From the CSW results the lowest measured Young's Modulus ($E_{0.1\%}$) at 0.1% strain was selected for the settlement calculations. The Young's Modulus ($E_{0.1\%}$) was taken 50 MPa. The maximum amount of settlement is estimated to be 0.121mm. The amount of settlement across the width of the foundation is shown in Figure 7.5. The settlement will occur immediately during construction.

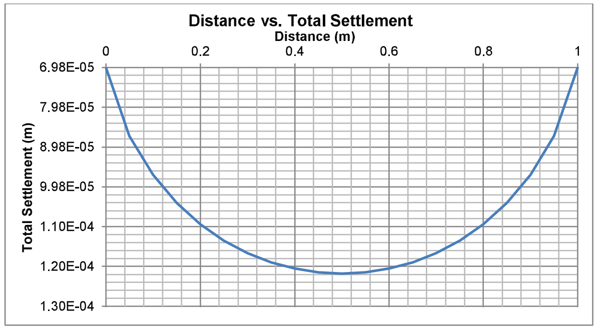


Figure 7.5: Total Settlement of the motor support foundation

7.5.9 Pile Installation

Displacement (Driven/rammed) steel hollow piles will not be feasible for over 90% of the development areas, owing to the presence of calcrete, cobbles and boulders. Furthermore, displacement piles will likely refuse before reaching a satisfactory embedment depth of 2m, "socketed" into weathered bedrock, in order to withstand uplift loads imposed by the wind. Small portions (about 10%) of Dyasonsklip 1 could be suitable for the use of displacement piles, but this has to be confirmed by pile load test at the onset of the construction phase.

The pier configuration recommended by Exosun is feasible provided that the holes are pre-drilled to a minimum embedment depth of 2m, "socketed" into weathered bedrock. A Rectangular 160x80x4mm section or a IPE 200 (W8 x 10) section is feasible for the use as a central pile and the C170 section is feasible for the use as an end pile provided that the above conditions are adhered to.

It is recommended that a pre-drilled displacement pile installation method is implemented, where the holes are pre-drilled to a minimum embedment depth of 2m with the pile base founded in weathered bedrock. After a hole has been drilled, a well graded fill material is placed and compacted from the base to the existing ground level. The pile section is then driven, open ended, to the minimum embedment depth. The diameter (D_0) of the boreholes should fall into the range of $1.00>L/D_0>1.25$, where L is the largest cross-sectional dimension of the pile. The well graded soil mixture used to refill the hole should be placed and compacted to 93% modified AASHTO at an optimum moisture content of -1 to +1 in, not exceeding, 300 mm thick layers. The grading of the fill material must be verified on site by a qualified geotechnical professional. It is essential that quality testing is carried out on site to ensure that the specified compaction is achieved. Typically a commercially-sourced G5 material is utilised, with a measure of fine-grained, alluvial onsite material mixed in to provide as a binder. It is recommended that, should the appointed contractor select this installation method that a series of varying soil mixes with differing percentages of G5/alluvial fines be submitted for laboratory testing to ascertain the most favourable density combination. A wet mix will always lead to greater compaction than dry mixing, and should be considered for the on-site installation.

Alternatively, circular ductile iron piles, open ended, should be driven to "socket" into the weathered bedrock. This method has been proven to be successful at the Khi Solar One plant, situated about 2km north of the sites. A typical 170 x 9mm section will be suitable to use as a central pile and a 118 x 7mm section for an end pile.

It is recommended that pile load tests are conducted at the onset of the construction phase in order to assess and verify the recommended embedment depths and to finalize the selections of the most effective pile types and pile installation procedure.

The interpretation of geotechnical conditions across the site is based upon the point information provided by the various investigation positions. Conditions intermediate to these have been inferred by interpolation, extrapolation and the use of professional judgement, to gain an overall evaluation of the geotechnical conditions likely to be encountered during construction. In the event, during the pre-drilling of the piles, of significant deviations from the inferred conditions being encountered, then it is recommended that the services of a

geotechnical professional are sought, as the recommended embedment depth may need to be exceeded.

8 DYASONSKLIP 2 FIELD INVESTIGATION AND DEVELOPMENT RECOMMENDATIONS

8.1 Trial Pitting

At TP06 an alluvial horizon was profiled as *dry*, red brown with creamy white, *loose to medium dense, intact,* poorly sorted coarse sand with nodules of CALCRETE of cobble and boulder size to 0.50m. This gives way to a pedogenic horizon profiled as *dry,* creamy white, *dense, intact,* CALCRETE hardpan. The trial pit was terminated at 0.55m.

At TP07 an alluvial horizon was profiled as *dry*, red brown with creamy white, *loose to medium dense, intact,* silty gravelly sand with CALCRETE nodules of cobble to boulder size down to 0.25m. This gives way to a pedogenic horizon profiled as *dry,* creamy white with brown staining, *medium dense to dense, intact,* CALCRETE hardpan. Trail pit was terminated at a depth of 0.30m.

At TP08 an alluvial horizon was profiled as *dry,* red brown with creamy white, *loose, intact,* silty gravelly SAND with roots and nodules of calcretes and gneiss down to a depth of 0.50m. This overlies a pedogenic horizon profiled as *dry,* cream white with grey, *medium dense to dense, intact,* hardpan CALCRETE. Trial pit terminated at 0.85m.

At TP09 an alluvial horizon was profiled as *dry,* red brown, *loose, intact,* silty gravelly SAND with root material to a depth of 0.15m. The alluvial horizon extends into a horizon profiled as *dry,* red brown with creamy white grey, *medium dense, intact,* poorly sorted medium GRAVEL with silty sand and cobbles of gneiss down to 0.50m. This gives way to a residual horizon profiled as *dry,* creamy white with grey, *medium dense to dense, intact,* highly to completely weathered cobbles and boulders of GNEISS intermixed with gravel. Trial pit was terminated at 0.80m.

At TP10 an alluvial horizon was profiled as *dry,* red brown, *medium dense, intact,* silty gravelly SAND with root material to a depth of 0.35m. The alluvial horizon gives way to a pedogenic horizon described as *dry,* creamy white with brown staining, *medium dense to dense, intact,* fractured CALCRETE with intermixed silty gravelly sand to a depth of 0.75m. The pedogenic horizon continues and was profiled as a *dry,* creamy white grey, *medium dense to dense, intact,* CALCRETE hardpan. Trial pit was terminated at 1.25m.

8.2 Electrical Resistivity Testing

Interpretations are quoted from the report compiled by ASST, reference should be made to the full report attached in Annexure E. ASST note that there is a general relationship with colour variation visible on Google Earth Imagery, with lighter zones generally recording higher

Resistivities in relation to darker areas. Resistivity profiles across Dyasonsklip 2 indicate varying resistivities both laterally and vertically with "pockets" of higher electrical resistivity.

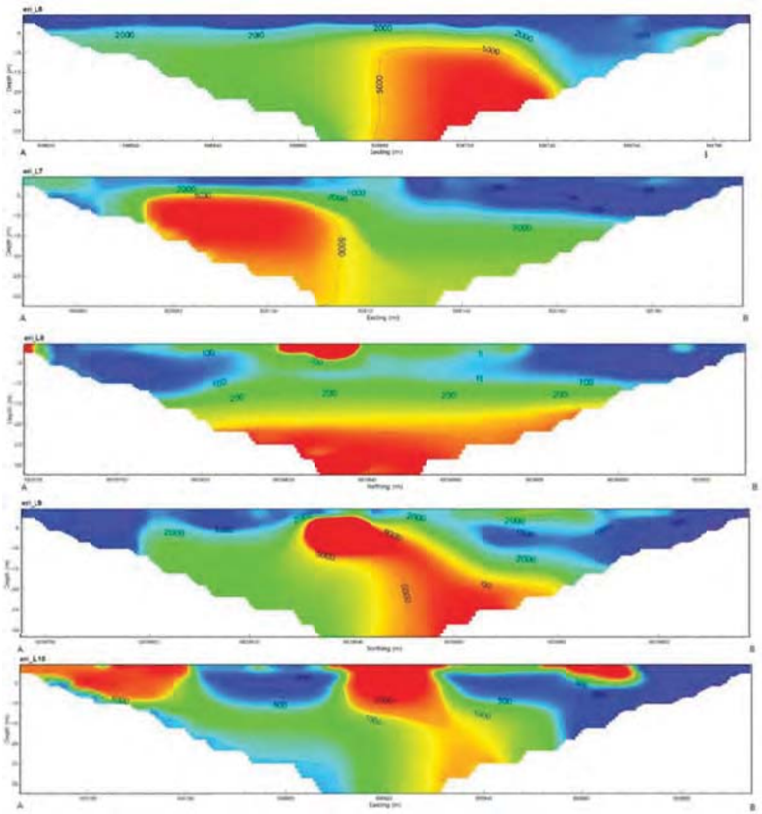


Figure 8.1: Electrical resistivity profiles for the five test posiions across Dyasonsklip 2.

8.3 Continuous Surface Wave Seismic Testing

Five (5) CSW seismic tests were conducted on the Dyasonsklip 2 site. The tests were conducted by G. Heymann CC and the full results are included in Appendix F with results utilised in the recommendation section.

8.4 Laboratory Results

Table 8.1: Point Load Test Results

Sample No.	BH No.	Depth	Description	Point Load Index	UCS (Mpa)
BH53A	BH53	2.09 - 2.23	Gneiss	2.14	51.4
BH54A	BH54	2.16 - 2.32	Gneiss	5.97	143.3
BH55A	BH55	4.00 - 4.11	Gneiss	0.18	4.4
BH56B	BH56	4.26 - 4.41	Gneiss	0.48	11.6
BH57A	BH57	2.52 - 2.64	Gneiss	0.08	2
BH58A	BH58	4.75 - 5.01	Granite	2.72	65.4
BH59A	BH59	4.38 - 4.50	Granite	2.72	65.3
BH60A	BH60	3.36 - 3.48	Gneiss	2.91	69.8
BH61A	BH61	3.09 - 3.21	Gneiss	14.50	348
BH62A	BH62	4.70 - 4.82	Gneiss	0.13	3.2
BH63A	BH63	4.34 - 4.50	Gneiss	0.38	9
BH64A	BH64	2.48 - 2.63	Gneiss	2.16	51.9
BH65A	BH65	3.25 - 3.40	Gneiss	1.42	34
BH66A	BH66	3.61 - 3.75	Granite	1.36	32.7
BH68B	BH68	1.50 - 1.76	Granite	5.23	125.4
BH70B	BH70	3.85 - 4.04	Gneiss	1.10	26.5
BH71A	BH71	3.85 - 4.04	Gneiss	5.05	121.2
BH73A	BH73	2.85 - 2.95	Gneiss	0.39	9.3
BH74A	BH74	4.07 - 4.35	Gneiss	9.23	221.5
BH77A	BH77	3.73 - 4.00	Gneiss	5.82	139.7
BHDG2B	BHDG2	4.50 - 4.69	Gneiss	13.89	333.4

Results of the Point Load tests on core retrieved from Dyasonsklip 2 indicates a strength range from 2.0 MPa and 348 MPa.

Table 8.2: UCS Test Results

Sample No.	Trial pit No.	Depth (m)	Description	Specimen Test Results		
				Failure Load	Strength (UCS)	Failure Code
BH53B	BH53	3.19 - 3.33	Gneiss	473.1	167	XA
BH56C	BH56	4.53 - 4.73	Gneiss	27.5	13	3B
BH57B	BH57	4.29 - 4.42	Gneiss	292.8	104.6	XB
BH61B	BH61	4.28 - 4.43	Gneiss	458.6	163.9	YA
BH65B	BH65	4.76 - 4.95	Gneiss	142.5	51.3	3B
BH68A	BH68	3.43 - 3.61	Granite	346.3	164.2	XB

Sample No.	Trial pit No.	Depth (m)	Description	Specimen Test Results		
				Failure Load	Strength (UCS)	Failure Code
BH71B	BH71	4.10 - 4.23	Gneiss	339	159.7	YA
BH77B	BH77	4.30 - 4.53	Gneiss	107.9	38	3B
BHDG2A	BHDG2	7.96 – 8.14	Gneiss	29.0	10.3	YA

Nine (9) samples were collected on Dyasonsklip 2 for UCS testing with a UCS strength range from 10.3MPa to 164.2MPa. The was a general split between samples failing as a result of discontinuities and samples not influenced by discontinuities, where samples failed from discontinuities there was complete failure at 21° to 30° from the axis.

Table 8.3: Atterberg Limit Determination Test Results on samples collected from rotary boreholes.

Sample No.	BH number	Depth (m)	Description	Soil Mortar Analysis (%)					Atterberg Limits (%)			GM	Potential Expansion
				Coarse Sand	Coarse Fine sand	Medium Fine Sand	Fine Fine Sand	Passing 0.075 mm	LL	PI	LS		
BH56A	BH56	0.95 - 1.50	Silty sandy Gravel	34	10	12	11	33	39	9	4.0	2.02	LOW
BH67A	BH67	2.40 - 3.00	Silty sand	34	11	36	10	9	-	NP	0.0	1.25	LOW
BH67B	BH67	5.45 - 6.50	Silty sand	42	21	17	13	7	-	NP	0.0	1.35	LOW
BH69A	BH69	4.50 - 5.00	Completely weathered Gneiss	42	18	18	11	11	-	NP	0.0	2.16	LOW
BH70A	BH70	0.00 - 0.64	Silty sand	20	15	20	20	24	-	NP	0.0	1.35	LOW
BH75A	BH75	4.69 - 5.00	Completely weathered granitic material	38	20	19	13	10	-	NP	0.0	1.54	LOW
BH77C	BH77	1.90 - 2.44	Silty sand	40	11	12	12	25	22	6	2.0	1.46	LOW

LL - Liquid Limit GM - Grading Modulus NP - Non Plastic

LS - Linear Shrinkage PI - Plasticity Index SP – Slightly Plastic

According to the Soil Mortar Analysis, Coarse Sand formed the bulk component of the sampled material with even spread between a 0.075mm passing and coarse fine sand. Plasticity Indices (PI) are mainly *Non-Plastic* with two samples recording values of 6% – 9%. Linear Shrinkage (LS) values were mainly 0% with two values recording values of 2% - 4%, values above 8% are considered to be problematic for moisture related heave and shrinkage. Potential expansion predictions indicate that all of the samples are contained within the *LOW* risk category for potential heave.

Table 8.4: Atterberg Limit Determination Test Results on samples collected from trial pitting

Sample No.	Trial Pit	Depth (m)	Description	Particle size distribution %				Atterberg Limits (%)			Potential Expansion
				Gravel	Sand	Silt	Clay	LL	PI	LS	
TP06	TP06	0.00 - 0.50	Silty sand with nodules of calcrete	43	48	4	5	21	1	0.8	LOW
TP08	TP08	0.00 - 0.50	Silty Gravelly Sand with nodules of calcrete	53	38	4	5	22	2	1.0	LOW
TP10	TP10	0.35 - 1.25	Calcrete with Silty sand	61	35	2	2	22	5	2.7	LOW

LL - Liquid Limit GM - Grading Modulus NP - Non Plastic

LS - Linear Shrinkage PI - Plasticity Index SP – Slightly Plastic

According to the Particle size distribution, Gravel and Sand formed the bulk component of the disturbed trial pit samples. Plasticity Indices (PI) were between 1% - 5 % and Linear Shrinkage (LS) values were 0.8% - 2.7 %, values above 8% are considered to be problematic for moisture related heave and shrinkage. Potential expansion predictions indicate that all of the samples are contained within the *LOW* risk category for potential heave.

Table 8.5: MOD/CBR

Sample No.	Trial Pit No.	Depth (m)	Description	MDD kg/m³	OMC %	CBR					Maximum Swell MAASHTO %	TRH 14 Classification
						90%	93%	95%	98%	100%		
TP06	TP06	0.00 - 0.50	Silty sand with nodules of calcrete	2028	5.7	21	50	70	97	116	0.02	G5
TP08	TP08	0.00 - 0.50	Silty Gravelly Sand with nodules of calcrete	2067	7.4	30	68	93	105	111	0.0	G5
TP10	TP10	0.35 - 1.25	Calcrete with Silty sand	2048	6.3	26	58	79	147	194	0.1	G5

The results from the MOD/CBR tests are fairly conclusive in their representation of in situ materials sampled from the investigation site. The classification of material (G5) reflects the composition and strength of the in situ soil. The tested material is of a good quality and may be suitable for use in fill, road / or structural applications depending upon final design requirements. All material classifications were assessed in accordance with TRH 14 specifications (1985).

TP08 (0.00m – 0.50m) recorded a dispersion percentage of 20%, non-dispersive.

Table 8.6: Moisture Content

Sample No.	Borehole No.	Description	Moisture Content (%)
BH56A	BH56	Silty sandy gravel	8.4
BH67A	BH67	Silty sand	9.9
BH67B	BH67	Silty sand	9.9
BH69A	BH69	Completely weathered gneiss	6.4
BH70A	BH70	Silty sand	0.6
BH75A	BH75	Completely weathered granitic material	5.9
BH77C	BH77	Silty sand	12.9
TP06	TP06	Silty sand with nodules of calcrete	1.6
TP08	TP08	Silty Gravelly Sand with nodules of calcrete	1.8
TP10	TP10	Calcrete with Silty sand	2.3

Disturbed samples collected from trial pits and samples collected from the boreholes indicate a low moisture contents below 10%, elevated moisture contents are expected for samples collected from boreholes as a result of the presence of drilling fluids.

Table 8.7: Falling head permeability result.

Sample Number	Depth (m)	Density kg/m³	W %	H1 (cm)	H2 (cm)	Time h	Time m	Time s	Permeability cm/s
TP08	0.00 - 0.5	1860	7.4	60	8	0	20	51	1.42E-04

The disturbed sample taken from trial pit TP08 recorded a permeability cm/s value of 1.42E-04.

Table 8.8: Soil aggresivity

Sample No.	Description	pH	Stability pH at 20°C	Langelier Index at 20°C	Ryznar Stability Index at 20°C	Aggressiveness Index	Corrosivity Ratio	Final agressivity index at 20°C
TP06	Silty sand with nodules of calcrete	8.1	8	0.1	7.9	10.3	0.3	176
TP08	Silty Gravelly Sand with nodules of calcrete	7.9	7.9	0	8	10.1	0.2	214
TP10	Calcrete with Silty sand	8	8.2	-0.2	8.4	10.1	0.3	283

According to the corrosivity indices for the sample taken at TP06 the sample is not corrosive towards concrete but slightly corrosive towards metals while the Basson Index indicates that the soil is non to mildly aggressive towards concrete. The corrosivity indices at TP08 indicates that the sample is not corrosive towards concrete and metals while the Basson Index indicates that the sample is non to mildly aggressive towards concrete. The corrosivity indices for TP10 indicates that the sample is slightly corrosive towards concrete

and metals while the Basson Index indicates that the sample is non to mildly aggressive towards concrete.

Table 8.9: Thermal conductivity results.

Sample Number	Moisture %	Thermal Conductivity (K) W/m.k	Thermal Resistivity (p) °C.cm/W	Volumetric specific heat (C) MJ/(m³.K)	Thermal Diffusivity (D) mm²/s
TP06	0	0.364	277.3	1.528	0.238
	3	0.83	120.6	1.36	0.661
	7	1.118	90.05	1.794	0.64

Field based thermal resistivity testing was conducted on one (1) sample retrieved from the Dyasonsklip 2 site. Thermal conductivity was 0.364 W/m.K at a moisture content of 0 %. Thermal conductivity shows an increase with increasing moisture content.

Table 8.10: Analytical report for water sample taken at Dyasonsklip 2.

Deterioration of Concrete in Aggressive Waters (Basson)

Dysansklip and Sirius Solar Farms Farm

Determinant	Units	Method	Method No	Parameter	DG1 Lab Result	DG1 Index	DG2 Lab Result	DG2 Index	DG3 Lab Result	DG3 Index	Screening Guidelines: moderately aggressive	high aggressive	Scale Forming Tendency	Corrosive Tendency
Results														
pH at 25°C	pH units	SABS Method 11	1	V1	7.7	360	7.5	400	7.6	380	6-8	5-6		
pH_s at 25°C	pH units	PCI TM 9.28	calculation	V2	7.63	-140	7.63	260	7.66	120				
Langelier Index LI ($pH-pH_s$)		calc			0.07	0.07	-0.13	-0.13	-0.06	-0.06	-0.2 to -0.3	-0.3 to -0.4		
Calcium hardness	mg $CaCO_3$/l	SABS Method 216	calc	V3	52	985.6	80	924	70	946	>200	<200		
Ammonia	mg N/l	SABS Method 217/218	64	V4	0.4	4	3.56	35.6	3.66	36.6	<50	>50		
Dissolved magnesium	mg Mg/l	SABS Method 1071	9A	V5	8	4.8	11	6.6	9	5.4	<500	>500		
Sulphate	mg SO_4/l	SABS Method 212/1056	67	V6	233	69.9	106	31.8	63.7	19.11	<1000	>1000		
Chloride	mg Cl/l	SABS Method 202	16	V7	27	5.4	16	3.2	19	3.8	<1000	>1000		
Total Dissolved Solids at 180°C	mg/l	SABS Method 213	41	V8	1255		285		440					
Total Alkalinity	mg $CaCO_3$/l		10		341		187		219					
Dissolved calcium	mg Ca/l		8A		21		32		28					
Calculations														
Concrete Aggressiveness Method (Basson)		Leaching Corrosion Sub Index LCSI			401.9		528.0		482.0					
		Spalling Corrosion Sub Index SCSI			26.2		24.7		20.4					
		Aggressiveness Index N (standard local conditions)			**428.1**		**552.7**		**502.4**					
Final Index Description					mildy to fairly aggressive		mildy to fairly aggressive		mildy to fairly aggressive		<750	>750		
Recommendations	Leaching Corrosion Counter Measures				Class 1 concrete, inorganic coating		Class 1 concrete, inorganic coating		Class 1 concrete, inorganic coating					
	Spalling Corrosion Counter Measures				Class 3, 4 or 5 concrete, inorganic coating		Class 4 or 5 concrete, inorganic coating		Class 3, 4 or 5 concrete, inorganic coating					
	Chloride Corrosion Counter Measures for rebar				Minimum 25mm concrete cover over rebar		Minimum 25mm concrete cover over rebar		Minimum 25mm concrete cover over rebar					
Domestic Water Use Method		Langelier			0.1		-0.1		-0.1				>0	<0
		Ryznar			7.6		7.8		7.7				<6.5	>6.5
		Corrosion Ratio			5.4		1.7		1.3					>0.1
		Aggressive Index			11.9		11.7		11.8		10 to 12	<10		

	INDEX	DESCRIPTION
Final Index	0-350	non to mildly aggressive
	350-750	mildy to fairly aggressive
	750-1000	highly aggressive
	1000	very highly aggressive

The results of all three samples indicate mildly to fairly aggressive conditions, based on concrete aggressiveness method (Basson).

Finally based on the chloride corrosion index, the recommended concrete thickness around all rebar is a minimum of 25mm.

The resting water level at the deep geotechnical hole was 16.16m, this is below the recommended piling depth.

8.5 Foundation and Development Recommendations – Dyasonsklip 2

The pile installation methods considered for Dyasonsklip 2 include displacement steel piles, displacement ductile iron piles and pre-bored steel piles. Exosun is proposing to implement a Exotrack® HZ V2 solar panel system with the following pier configuration:

- Central pile:
  - 160x80x4 Rectangular hollow steel section,
  - IPE 200 (Equivalent to W8 x 10) I-section,

- End piles:
  - C170x60, or
  - IPE 160 (Equivalent to W6 x 9) I-section,

Due to the presence of calcrete, gravel, shallow bedrock, and abundant pebbles, cobbles and boulders, steel displacement piles will likely reach shallow refusal and could deform as a result. The DPSH probes reached refusal at shallow depths across the sites of Dyasonklip 2. The SPT N-value readings generally exceed 20 before reaching refusal. Displacement steel hollow piles are generally unsuccessful in soils with a SPT N-value exceeding 20.

The alternatives are pre-drilling steel piles or circular hollow ductile iron displacement piles (TRM or similar approved). The latter has been used successfully at the Khi Solar One plant, approximately 2km north of the sites, where circular hollow ductile iron displacement piles (TRM piles) were driven, open ended, through the alluvial and “socketed” into the underlying bedrock. Similarly, pre-drilled steel piles could be founded in the underlying bedrock resulting in minor vertical settlement.

8.5.1 Loading

The loading imposed on the piles include the self-weight of the panels and piles, wind load imposed on the panels and the load applied by the daily tracking unit. The typical maximum loads applied on the piles were obtained from a document provided by the supplier titled “Exotrack HZ® foundations loads According to SANS 10160”. The loads include the following:

- Vertical Direction:
  - $Fv_{central\_down}$ = 7000 N (downward load on central pile),
  - $Fv_{central\_uplift}$ = -2500 N (uplift load on central pile),
  - Fv_{end_down} = 6000 N (downward load on end pile),
  - Fv_{end_uplift} = -2500 N (uplift load on end pile),
  - $Fv_{motor\ support}$ = 400 N (downward on motor foundation).
- Horizontal Direction:
  - $Fh_{central}$ = 11 000 N (central pile),
  - $Mth_{central}$ = 2400 N.m (moment of central pile),
  - $Fh_{end\ pile}$ = 3000 N (end pile),
  - Fh_{motor} = 900 N (motor foundation).

These loads were applied to theoretical formulae in order to estimate the vertical settlement horizontal displacement rotation and minimum embedment depth of the piles. All calculations were carried out according to the Eurocode 7 Design Approach 1b standards.

8.5.2 Material Classification and Excavation Conditions

Excavation classes and rock classifications for the subsurface materials occurring across the investigated area have been estimated using the definitions provided in the SANS 1200D (1988) and SANS 1200F (1988) standards. The excavatability and rippability of the underlying material were assessed by evaluating borehole logs and the shear wave velocities obtained from the CSW tests. The S-wave velocities for the CSW tests conducted at Dyasonsklip1 is shown in Figure 8.2.

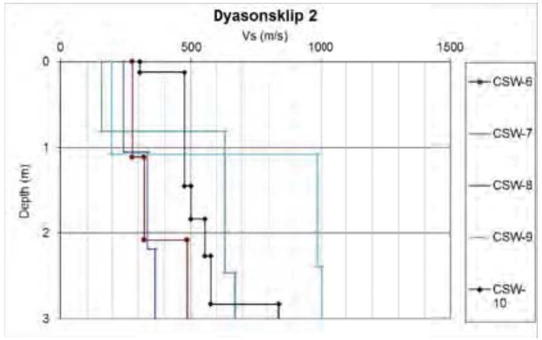


Figure 8.2: CSW test results at Dyasonsklip 2

Table 8.11 shows the excavation classes and rock classifications expected for the subsurface material across the site.

Table 8.11: Excavation Classes for the subsurface material at Dyasonsklip 2

Site	Depth (m)	S-wave Velocity (m/s)	Class of Excavation (SANS 1200D)	Rock Classification (SANS 1200F)
Dyasonsklip 2	0.0m – 0.8m	200m/s< Vs< 400m/s	Soft Excavation to Intermediate with Class A boulder excavation	-
	0.8m – 3.0m	400m/s< Vs< 1000m/s	Intermediate to Hard Excavation with Class A boulder excavation between 0.8-1.5m	R3

8.5.3 Material Parameters and Assumptions

The particle size analysis of the alluvial material show that the horizon consists predominantly of a non-plastic sand, with abundant gravel, calcrete, cobbles and boulders in a dense consistency. According to the work done by Gibson (1953) a typical material with these characteristics will exhibit an internal friction angle (ϕ') of 30°.

The following assumptions were made in order to conduct further calculations:

- A unit weight of 18 kN/m^3 was assumed for the alluvium.
- The Poisson's Ratio (*v*) of the subsoils is in the order of 0.3
- The interface friction angle (δ') were assumed to equal to 0.75ϕ',

- The lateral earth pressure coefficient was assumed to be equal to the lateral earth pressure coefficient of earth pressure at rest, $K/K_0=1$.
- The piles were assumed to be open ended.

8.5.4 Bearing Resistance

The minimum pile embedment depth was calculated for the central and corner pile as a function of the base and shaft resistance. The shear strength of the subsurface material, the effective normal stress acting on the piles and the pile properties contribute to the overall bearing resistance of the subsurface material. The bearing resistance formulae as given by Knappet and Craig (2012) are as follow:

Base resistance:

$$Q_{bu} = A_p(N_q \sigma'_q)$$

Shaft resistance:

$$Q_{su} = P_p \int_0^{L_p} \tau_{int}\,(z)dz$$

Where: A_p is the area of the pile cross-section,

N_q is a bearing capacity factor depended on the slenderness and friction angle of the subsoils,

σ'_q is surcharge above the pile base,

P_p is the pile perimeter,

L_p is the length of the pile.

τ_{int} is the shear strength along the interface of the pile and the soil.

8.5.5 Embedment Depth

The minimum embedment depth was calculated by assessing two conditions. For the first condition it is assumed that the bedrock is situated well below the pile base and the second condition when the pile base is situated within competent weathered bedrock.

Table 8.12 indicates the condition that the bedrock is situated well below the pile base, the minimum embedment depth ranges between 2.55m and 2.90m for limiting vertical loads of 7kN in the case of a central pile and 6kN in the case of end piles.

Table 8.12: Minimum embedment depth of pile founded in alluvium

Pile Section	Minimum Embedment Depth (m)	Compression Bearing Resistance ($Q_{bu}+Q_{su}$) (kN)	Tension Bearing Resistance (uplift wind load) (kN)
Central Pile			
Steel Hollow Pile 160 x 80 x 4mm	2.90	7.2	4.8
IPE 200	2.55	7.2	4.5
170 x 9 Circular Hollow (TRM)	2.60	7.0	3.6
End Pile			
IPE 160	2.60	6.0	3.9
C160 x 65	2.70	6.0	4.0
118 x 7.5 Circular Hollow Pile (TRM)	2.90	6.0	4.7

Table 8.13 indicates the second conditions where the pile base is founded within the competent bedrock the pull-out tension load will govern the embedment depth. The minimum embedment depth was found to range between 1.9m and 2.2m at a pull-out load limit of 2.5 kN.

Table 8.13: Minimum embedment depth of piles founded on weathered bedrock

Pile Section	Minimum Embedment Depth (m)	Compression Bearing Resistance ($Q_{bu}+Q_{su}$) (kN)	Tension Bearing Resistance (uplift wind load) (kN)
Central Pile			
Steel Hollow Pile 160 x 80 x 4mm	2.1	-	2.5
IPE 200	1.9	-	2.5
170 x 9 Circular Hollow (TRM)	2.2	-	2.5
End Pile			
IPE 160	2.1	-	2.5
C160	2.1	-	2.5
118 x 7.5 Circular Hollow Pile (TRM)	2.1	-	2.5

It is recommended that pile load tests are conducted at the onset of the construction phase in order to assess and verify the recommended embedment depths.

8.5.6 Settlement

The maximum estimated settlement was calculated in the event that the bedrock is situated well below the pile base level. The settlements were determined by applying the analytical techniques by Randolph and Wroth (1978) method and the T-z Finite Differences method. These methods require the follow input parameters:

- Young's Modulus (E) of the pile,
- Shear Modulus ($G_{0.1\%}$) distribution of the subsoils,
- Poison's Ratio of the surrounding soils (*v*)
- Dimensions of the piles

- Pile Force.

Young's Modulus (E) of a typical steel pile was selected to be 200 x 10^3 GPa, as per the Southern African Steel Construction Handbook (Sixth Edition, 2008). The maximum amount of vertical settlement will occur at the central pile as a result of a vertical load of 7000N.

The operative level of strain for a foundation structure is 0.1%. The shear modulus ($G_{0.1\%}$) values at 0.1% strain were determined by applying a softening function proposed by Rollins et. Al. (1998) to the shear modulus data, Figure 8.3, overleaf, shows the minimum estimated shear modulus ($G_{0.1\%}$) to a depth of 3m.

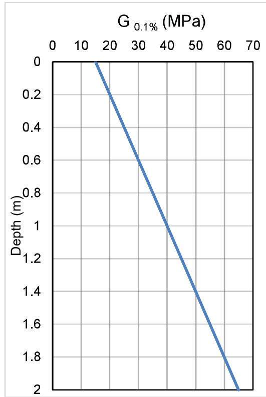


Figure 8.3: Minimum Shear Modulus of Subsoils

Figure 8.4 shows the pile load and settlement response to a depth of 3m. The total vertical settlement was estimated to be 1mm. The settlement of the piles is not sensitive to the marginal dimensional changes of the pile sections.

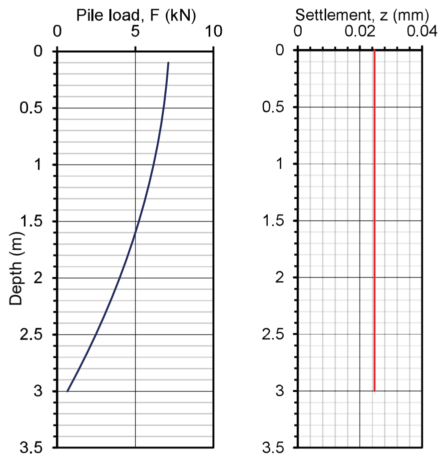


Figure 8.4: Settlement response for a pile to a depth of 3m

8.5.7 Horizontal Displacement and Rotation

The horizontal displacement was estimated by applying elasticity theory solutions. The horizontal displacement and rotation at the pile head, as given by Knappet and Craig (2012), are as follow:

Horizontal Displacement:

$$h = \frac{\left(\frac{E_p}{G_c}\right)^{1/7}}{p_c G_c}\left[\frac{H}{1.85L_c} + \frac{M}{0.83L_c^2}\right]$$

Rotation:

$$\theta = \frac{\left(\frac{E_p}{G_c}\right)^{1/7}}{p_c G_c}\left[\frac{H}{0.83L_c^2} + \frac{M}{0.16p_c^{-0.5}L_c^3}\right]$$

Where: E_p is Young's Modulus of the pile,

G_c is the median shear modulus,

p_c is the homogeneity factor describing shear modulus with depth,

L_c is the pile embedment depth,

H is the horizontal load, and

M is the moment applied around the head.

The horizontal displacement (h) at the pile head was calculated to be a maximum of 2mm with a rotation (θ) of 2°.

8.5.8 Motor support

The motor support foundation is required to carry a vertical dead load of approximately 400N and a horizontal wind load of approximately 900N. The load combination applies an eccentric stress to the subsurface soil. It was assumed that the foundation is situated on the alluvium with a horizon thickness of approximately 1.5m. A square foundation was assumed with a width (B) and length (L) of 1m.

The operative level of strain for a foundation structure is 0.1%. The settlement of the foundation is dependent on Young's Modulus (E) of the subsoils at 0.1% strain. From the CSW results the lowest measured Young's Modulus ($E_{0.1\%}$) at 0.1% strain was selected for the settlement calculations. The Young's Modulus ($E_{0.1\%}$) was taken 50 MPa. The maximum amount of settlement is estimated to be 0.121mm. The amount of settlement across the width of the foundation is shown in Figure 8.5. The settlement will occur immediately during construction.

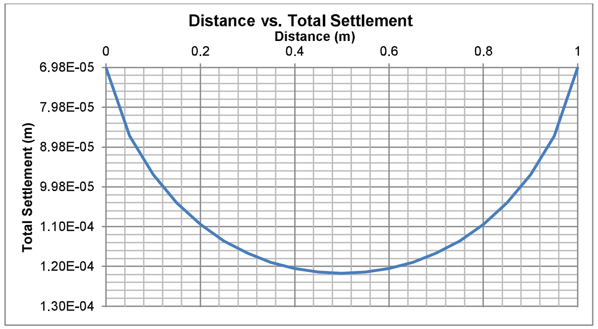


Figure 8.5: Total Settlement of the motor support foundation

8.5.9 Pile Installation

Displacement (Driven/rammed) steel hollow piles will not be feasible for over 90% of the development areas, owing to the presence of calcrete, cobbles and boulders. Furthermore, displacement piles will likely refuse before reaching a satisfactory embedment depth of 2m, "socketed" into weathered bedrock, in order to withstand uplift loads imposed by the wind. Small portions (about 10%) of Dyasonsklip 2 could be suitable for the use of displacement piles, but this has to be confirmed by pile load test at the onset of the construction phase.

The pier configuration recommended by Exosun is feasible provided that the holes are pre-drilled to a minimum embedment depth of 2m, "socketed" into weathered bedrock. A Rectangular 160x80x4mm section or a IPE 200 (W8 x 10) section is feasible for the use as a central pile and the C170 section is feasible for the use as an end pile provided that the above conditions are adhered to.

It is recommended that a pre-drilled displacement pile installation method is implemented, where the holes are pre-drilled to a minimum embedment depth of 2m with the pile base founded in weathered bedrock. After a hole has been drilled, a well graded fill material is placed and compacted from the base to the existing ground level. The pile section is then driven, open ended, to the minimum embedment depth. The diameter (D0) of the boreholes should fall into the range of 1.00>L/D0>1.25, where L is the largest cross-sectional dimension of the pile. The well graded soil mixture used to refill the hole should be placed and compacted to 93% modified AASHTO at an optimum moisture content of -1 to +1 in, not exceeding, 300 mm thick layers. The grading of the fill material must be verified on site by a qualified geotechnical professional. It is essential that quality testing is carried out on site to ensure that the specified compaction is achieved. Typically a commercially-sourced G5 material is utilised, with a measure of fine-grained, alluvial onsite material mixed in to provide as a binder. It is recommended that, should the appointed contractor select this installation method that a series of varying soil mixes with differing percentages of G5/alluvial fines be submitted for laboratory testing to ascertain the most favourable density combination. A wet mix will always lead to greater compaction than dry mixing, and should be considered for the on-site installation. An alluvial soil profile, devoid of the calcrete hardpan that typifies the majority of the site, was encountered at borehole position BH67. At this position, and immediate surrounds of <200m, it is envisioned that no pre-drilling would be required and steel piles may be rammed by way of conventional methods.

Alternatively, circular ductile iron piles, open ended, should be driven to "socket" into the weathered bedrock. This method has been proven to be successful at the Khi Solar One plant, situated about 2km north of the sites. A typical 170 x 9mm section will be suitable to use as a central pile and a 118 x 7mm section for an end pile.

It is recommended that pile load tests are conducted at the onset of the construction phase in order to assess and verify the recommended embedment depths and to finalize the selections of the most effective pile types and pile installation procedure.

The interpretation of geotechnical conditions across the site is based upon the point information provided by the various investigation positions. Conditions intermediate to these have been inferred by interpolation, extrapolation and the use of professional judgement, to gain an overall evaluation of the geotechnical conditions likely to be encountered during

construction. In the event, during the pre-drilling of the piles, of significant deviations from the inferred conditions being encountered, then it is recommended that the services of a geotechnical professional are sought, as the recommended embedment depth may need to be exceeded.

9 SIRIUS FIELD INVESTIGATION AND DEVELOPMENT RECOMMENDATIONS

9.1 Trial pitting

At TP11 an alluvial horizon was profiled as *dry,* red brown, *loose, intact,* silty gravelly SAND with minor roots to a depth of 0.15m. This overlies a pedogenic horizon profiled as *dry,* creamy white with grey, *medium dense, intact,* CALCRETE hardpan with minor silty gravelly sand *to a depth of 0.60m.* This gives way to bed rock profiled as creamy white with grey speckles, moderately weathered alkali poor GRANITE. Trial pit was terminated at 0.65m.

At TP12 an alluvial horizon was profiled as *dry,* red brown, *loose to medium dense, intact,* silty sandy GRAVEL with quartz, calcretes and granite clasts to a depth of 0.10m. This gives way to a pedogenic horizon profiled as *dry,* creamy white with grey, *dense, intact,* CALCRETE hardpan with cemented in situ material. Trial pit was terminated at 0.50m.

At TP13 an alluvial horizon was profiled as *dry,* red brown, *medium dense, intact,* silty sandy GRAVEL with quartz, calcretes and gneiss clasts of cobble size and minor roots down to 0.15m. This gives way to a pedogenic horizon described as *dry,* creamy white with grey, *dense, intact,* CALCRETE hardpan with cemented in situ clasts. Trial pit was terminated at 0.55m.

At TP14 an alluvial horizon was profiled as *dry,* red brown, *loose, intact,* silty sandy GRAVEL with roots and nodules of calcrete, quartz and schist to a depth of 0.45m. This overlies a bed rock horizon profiled as light grey, soft to medium hard, completely to highly weathered SCHIST. Trial pit was terminated at 1.40m.

At TP015 an alluvial deposit was profiled as *dry,* red brown, *loose, intact,* silty gravelly SAND with nodules of calcretes to a depth of 0.25m. This overlies a pedogenic horizon profiled as *dry,* creamy white with grey, *medium dense, intact,* CALCRETE hard pan with minor silty sand to a depth of 0.70m. This gives way to bedrock profiled as creamy white with grey speckles, soft rock, highly to completely weathered GNEISS. Trial pit was terminated at a depth of 1.50m.

9.2 Former Artisanal Mining Works

Old work trenches have been noted when encountered in the field and during desktop studies. All trenches will require infilling with an engineered fill to natural ground level. It is recommended that the infilling material should be of G7 or better quality, compacted in 200mm layers at 93% MDD. Approximate infill volumes are provided in table 9.1, it is recommended that a comprehensive survey is conducted on all trenches to determine actual volumes required. All calculated volumes in table 9.1 are based on basic field observations and measurements.

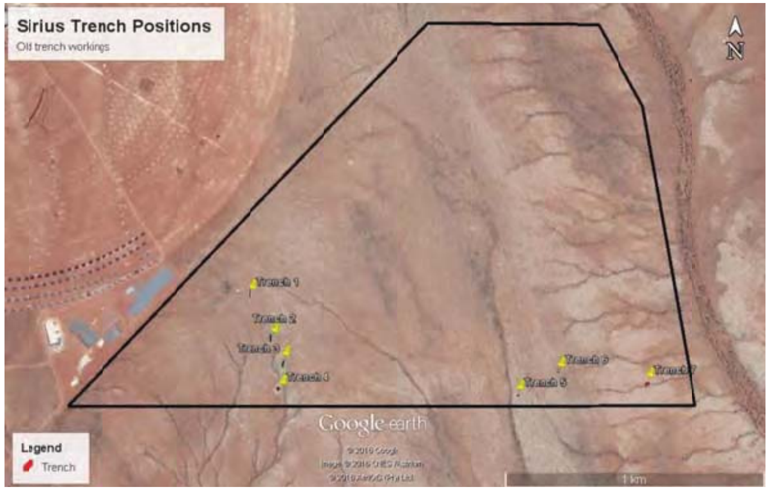


Figure 9.1: Location map for old trench workings encountered while conducting field work.

Trenches identified are based on field observations and Google Earth imagery and may not represent all old artisanal workings across the investigation areas.

Table 9.1: Approximate trench positions on Sirius site.

ID	Southing	Easting	Comment	Approximate Volume
Trench 1	28 32 48.30 S	21 05 16.60 E	Long rectangular with multiple excavations along the long axis, deepest trench ~ 2.48m	Volume 265.36m^2
Trench 2	28 32 54.27 S	21 05 19.65 E	Identified on Google Earth	Approximate volume not available
Trench 3	28 32 57.68 S	21 05 21.41 E	Identified on Google Earth	Approximate volume not available
Trench 4	28 33 00.67 S	21 05 20.74 E	Identified on Google Earth	Approximate volume not available
Trench 5	28 33 01.60 S	21 05 55.20 E	Tabular, ~0.20m deep	Volume 2.92m^2
Trench 6a	28 32 58.80 S	21 06 01.00 E	Shallow circular trench	Volume 2.055m^2
Trench 6b	28 32 58.50 S	21 06 01.10 E	Circular, ~0.25m deep	Volume 1.9875m^2
Trench 7	28 33 00.20 S	21 06 14.20 E	Circular with deepest portion at ~ 2.10m. Muscovite rich quartz vein	Volume 281.4m^2

9.3 Electrical Resistivity Testing

Interpretations are quoted from the report compiled by ASST, reference should be made to the full report attached in Annexure E. ASST note that there is a general relationship with colour variation visible on Google Earth Imagery, with lighter zones generally recording higher resistivities in relation to darker areas. Resistivity profiles across Sirius generally indicate an upper lower resisitivity layer which in turn gives way to higher resistivities with depth.

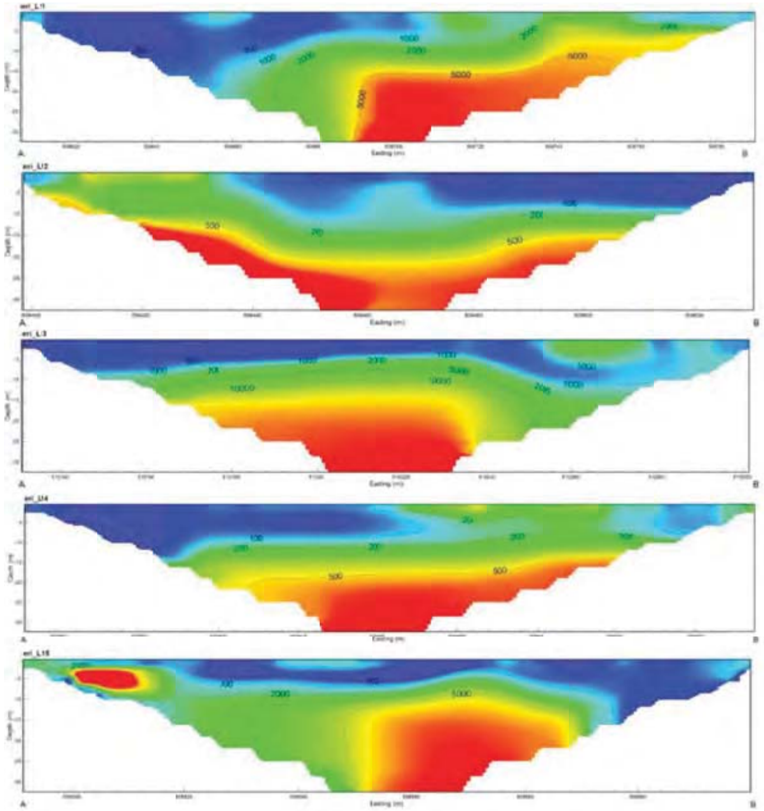


Figure 9.2: Electrical resistivity profiles for the five test posiions across Sirius.

9.4 Continuous Surface Wave Seismic Testing

Five (5) CSW seismic tests were conducted on the Sirius site. The tests were conducted by G. Heymann CC and the full results are included in Appendix F with results utilised in the recommendation section.

9.5 Laboratory Results

Table 9.2: Point Load Test Results

Sample No.	BH No.	Depth	Description	Point Load Index	UCS (Mpa)
BH105B	BH105	4.88 - 5.00	Granite	2.53	60.7
BH105C	BH105	3.00 - 3.13	Sheared Granite	1.38	33.1
BH106B	BH106	4.87 - 5.05	Granite	10.24	245.7
BH107A	BH107	4.50 - 4.73	Veined Schist	5.17	124.1
BH108A	BH108	2.88 - 3.00	Schist	1.78	42.8
BH109A	BH109	2.88 - 3.00	Granite	4.87	117
BH110A	BH110	3.94 - 4.08	Granite	5.81	139.5
BH111A	BH111	2.50 - 2.61	Schist	0.64	15.3
BH112B	BH112	3.77 - 3.90	Granite	4.15	99.6
BH113A	BH113	3.74 - 3.95	Granite	13.22	317.2
BH114A	BH114	2.85 - 3.00	Granite	5.88	141
BH115A	BH115	3.79 - 3.92	Schist	0.95	22.8
BH116A	BH116	4.56 - 4.84	Schist	0.96	23.1
BH117B	BH117	4.90 - 5.00	Schist	5.81	139.4
BH118A	BH118	3.18 - 3.37	Schist	5.05	121.2
BH119A	BH119	3.00 - 3.27	Schist with veined	2.97	71.2
BH120A	BH120	3.00 - 3.13	Gneiss	2.78	66.8
BH121A	BH121	2.28 - 2.44	Schist	5.38	129.2
BH122A	BH122A	2.50 - 2.66	Gneiss	0.33	8
BH123B	BH123	1.77 - 2.07	Schist	1.33	31.9
BH124A	BH124	3.72 - 3.86	Granite	2.64	63.4
BH125B	BH125	4.50 - 4.71	Gneiss	9.12	218.9
BH126A	BH126	4.29 - 4.50	Gneiss	3.97	95.3
BH127A	BH127	4.20 - 4.40	Schist	0.35	8.5
BH128A	BH128	2.16 - 2.25	Schist	0.47	11.3
BH129A	BH129	3.90 - 4.00	Pegmatite vein	0.40	9.6
BHDG3B	BHDG3	3.00 - 3.12	Granite	1.48	35.5

Results of the Point Load tests on core retrieved from Sirius indicates a strength range from 8 MPa and 317.2 MPa with the majority of the values below 100 MPa.

Table 9.3: UCS testing results

Sample No.	Trial pit No.	Depth (m)	Description	Specimen Test Results		
				Failure Load	Strength (UCS)	Failure Code
BH105A	BH105	4.64 - 4.88	Granite	139.9	66.1	XA
BH106A	BH106	3.86 - 4.15	Granite	380.1	180.8	YA
BH110B	BH110	4.70 - 4.90	Gneiss	188.3	87.1	2B
BH111B	BH111	4.00 - 4.19	Schist	44.9	21.1	2B
BH112A	BH112	4.50 - 4.67	Granite	293.4	104.4	XB
BH114B	BH114	3.00 - 3.15	Granite	308.3	143.6	XB

Sample No.	Trial pit No.	Depth (m)	Description	Specimen Test Results		
				Failure Load	Strength (UCS)	Failure Code
BH118B	BH118	3.86 - 4.00	Schist	132.4	63.2	1B
BH121B	BH121	2.44 - 2.70	Schist	67.6	31	1B
BH122B	BH122	4.89 - 5.00	Gneiss	137.6	64.7	1B
BH124B	BH124	4.76 - 4.96	Granite	177	83.8	2B
BH125A	BH125	3.00 - 3.27	Gneiss	164.8	58.5	2B
BH129B	BH129	4.93 - 5.05	Pegmatite	44.8	21	0B
BHDG1B	BHDG1	7.67 - 7.82	Gneiss	35.6	16.8	0B
BHDG2A	BHDG2	7.96 - 8.14	Gneiss	29	10.3	YA
BHDG3A	BHDG3	4.50 - 4.72	Granite	146.4	69.5	4B

Fifteen (15) samples were collected on Sirius for UCS testing with a UCS strength range from 10.3MPa to 180.8MPa. The majority of the samples exhibited a sliding shear failure to splitting, where failure was influenced by discontinuities there was a complete failure generally from 0° - 20° with one sample failing at 31° to 40° with two samples failing along multiple discontinuities. This is directly related to the foliation and recrystallization of mica minerals along the foliation planes.

Table 9.4: Atterberg Limit Determination Test Results on samples collected from rotary boreholes.

Sample No.	BH number	Depth (m)	Description	Soil Mortar Analysis (%)					Atterberg Limits (%)			GM	Potential Expansion
				Coarse Sand	Coarse Fine sand	Medium Fine Sand	Fine Fine Sand	Passing 0.075 mm	LL	PI	LS		
BH113B	BH113	0.00 - 0.53	Silty sand	32	15	21	17	15	-	NP	0.0	1.18	LOW
BH117A	BH117	0.70 - 1.50	Silty sand	28	14	18	17	0	-	NP	0.0	1.72	LOW
BH123A	BH123	0.00 - 0.20	Silty sand	24	11	22	23	20	17	3	1.5	1.29	LOW
BH124C	BH124	0.10 - 0.70	Silty sand	28	11	19	20	23	-	NP	0.0	1.22	LOW

LL - Liquid Limit GM - Grading Modulus NP - Non Plastic

LS - Linear Shrinkage PI - Plasticity Index SP – Slightly Plastic

According to the Soil Mortar Analysis, Coarse Sand formed the bulk component of the sampled material with the remaining material spread between the 0.075mm passing and coarse fine sand. Plasticity Indices (PI) were generally *Non-Plastic* with one sample recording a value of 3% while Linear Shrinkage (LS) values were 0.0% - 1.5 %, values above 8.0 are considered to be problematic for moisture related heave and shrinkage. Potential expansion predictions indicate that all of the samples are contained within the *LOW* risk category for potential heave.

Table 9.5: Atterberg Limit Determination Test Results on samples collected from trial pitting

Sample No.	Trial Pit	Depth (m)	Description	Particle size distribution %				Atterberg Limits (%)			Potential Expansion
				Gravel	Sand	Silt	Clay	LL	PI	LS	
TP11	TP11	0.15 - 0.60	Calcrete with minor silty gravelly sand	65	28	6	1	20	2	1.0	LOW
TP12	TP12	0.00 - 0.10	Silty sandy gravel	81	13	5	1	18	4	2.2	LOW
TP14	TP14	0.45 - 1.40	Highly to completely weathered schist	65	27	7	1	29	5	2.5	LOW

LL - Liquid Limit GM - Grading Modulus NP - Non Plastic

LS - Linear Shrinkage PI - Plasticity Index SP – Slightly Plastic

According to the Particle size distribution, Gravel and Sand formed the bulk component of the sampled material. Plasticity Indices (PI) were between 2% - 5% while Linear Shrinkage (LS) values were 1.0% - 2.5 %, values above 8.0 are considered to be problematic for moisture related heave and shrinkage. Potential expansion predictions indicate that all of the samples are contained within the *LOW* risk category for potential heave.

Table 9.6: MOD/CBR

Sample No.	Trial Pit No.	Depth (m)	Description	MDD kg/m³	OMC %	CBR					Maximum Swell MAASHTO %	TRH 14 Classification
						90%	93%	95%	98%	100%		
TP11	TP11	0.15 - 0.60	Calcrete with minor silty gravelly sand	2147	6.7	59	72	82	91	97	0.02	G5
TP12	TP12	0.00 - 0.10	Silty sandy gravel	2192	3.6	26	76	110	195	253	0.05	G6
TP14	TP14	0.45 - 1.40	Highly to completely weathered schist	2107	7.4	20	29	35	47	55	0.07	G6

The results from the MOD/CBR tests are fairly conclusive in their representation of in situ materials sampled from the investigation site. The classification of material (G5) - (G6) reflects the composition and strength of the in situ soil. The tested material is of a good to moderate quality and may be suitable for use in fill, road / or structural applications depending upon final design requirements. All material classifications were assessed in accordance with TRH 14 specifications (1985).

TP13 (0.00m – 0.15m) recorded a dispersive percentage of 60%, which is regarded as dispersive.

Table 9.7: Moisture content

Sample No.	Borehole No.	Description	Moisture Content (%)
BH113B	BH113	Soil	13.5
BH117A	BH117	Soil	0.9
BH123A	BH123	Silty sand	17
BH124C	BH124	Soil	15
TP11	TP11	Calcrete with minor silty gravelly sand	0.8
TP12	TP12	Silty sandy gravel	0.6
TP14	TP14	Highly to completely weathered schist	1.4

Disturbed samples collected from trial pits and samples collected from the boreholes indicate a low moisture content with trial pit values below 2%, elevated moisture contents are expected for samples collected from boreholes as a result of the presence of drilling fluids.

Table 9.8: Falling head permeability result.

Sample Number	Depth (m)	Density kg/m^3	W %	H1 (cm)	H2 (cm)	Time			Permeability cm/s
						h	m	s	
TP12	0.00 - 0.1	1973	3.6	60	9.8	0	21	49	1.20E-04

The disturbed sample taken from trial pit TP12 recorded a permeability cm/s value of 1.20E-04.

Table 9.9: Soil Aggresivity results

Sample No.	Description	pH	Stability pH at 20°C	Langelier Index at 20°C	Ryznar Stability Index at 20°C	Aggressiveness Index	Corrosivity Ratio	Final agressiveness index at 20°C
TP11	Calcrete with minor silty gravelly sand	8	8.3	-0.3	8.6	10.1	0.3	319
TP12	Silty sandy gravel	8	8	0	8	10.2	0.2	214
TP14	Highly to completely weathered schist	7.9	8.2	-0.3	8.5	10	9.8	323

According to the corrosivity indices for TP11 the sample is slightly corrosive towards concrete and metals while the Basson Index indicates that the sample is non to mildly aggressive towards concrete. The corrosivity indices for TP12 indicates that the sample is not corrosive towards concrete and metals while the Basson Index indicates that the sample is non to mildly aggressive towards concrete. The corrosivity indices for TP14 indicates that the soil is slightly corrosive towards concrete and highly corrosive towards metals while the Basson Index indicates that the sample is non to mildly aggressive towards concrete.

Table 9.10: Thermal conductivity results

Sample Number	Moisture %	Thermal Conductivity (K) W/m.k	Thermal Resistivity (p) °C.cm/W	Volumetric specific heat (C) MJ/(m³.K)	Thermal Diffusivity (D) mm²/s
TP11	0	0.233	431.9	1.238	0.188
	3	0.592	170.4	0.985	0.604
	7	0.929	111.2	1.485	0.645
TP14	0	0.251	399.8	1.2	0.21
	3	0.63	160.9	1.419	0.448
	7	1.11	92.83	1.47	0.778

Field based thermal resistivity testing was conducted on two (2) samples retrieved from the Sirius site. Thermal conductivity ranged between 0.233 – 0.251 W/m.K at a moisture content of 0 %. Thermal conductivity shows an increase with increasing moisture content.

Table 9.11: Analytical report for water samples taken.

Jeffares & Green

Deterioration of Concrete in Aggressive Waters (Basson)														
Dysansklip and Sirius Solar Farms Farm														
Determinant	Units	Method	Method No	Parameter	DG1		DG2		DG3		Screening Guidelines			
					Lab Result	Index	Lab Result	Index	Lab Result	Index	moderately aggressive	high aggressive	Scale Forming Tendency	Corrosive Tendency
Results														
pH at 25°C	pH units	SABS Method 11	1	V1	7.7	360	7.5	400	7.6	380	6-8	5-6		
pH$_s$ at 25°C	pH units	PCI TM 9.28	calculation	V2	7.63	-140	7.63	260	7.66	120				
Langelier Index LI (pH-pH$_s$)		calc			0.07	0.07	-0.13	-0.13	-0.06	-0.06	-0.2 to -0.3	-0.3 to -0.4		
Calcium hardness	mg $CaCO_3$/l	SABS Method 216	calc	V3	52	985.6	80	924	70	946	>200	<200		
Ammonia	mg N/l	SABS Method 217/218	64	V4	0.4	4	3.56	35.6	3.66	36.6	<50	>50		
Dissolved magnesium	mg Mg/l	SABS Method 1071	9A	V5	8	4.8	11	6.6	9	5.4	<500	>500		
Sulphate	mg SO_4/l	SABS Method 212/1056	67	V6	233	69.9	106	31.8	63.7	19.11	<1000	>1000		
Chloride	mg Cl/l	SABS Method 202	16	V7	27	5.4	16	3.2	19	3.8	<1000	>1000		
Total Dissolved Solids at 180°C	mg/l	SABS Method 213	41	V8	1255		285		440					
Total Alkalinity	mg $CaCO_3$/l		10		341		187		219					
Dissolved calcium	mg Ca/l		8A		21		32		28					
Calculations														
Concrete Aggressiveness Method (Basson)		Leaching Corrosion Sub Index LCSI			401.9		528.0		482.0					
		Spalling Corrosion Sub Index SCSI			26.2		24.7		20.4					
		Aggressiveness Index N (standard local conditions)			**428.1**		**552.7**		**502.4**					
Final Index Description					mildy to fairly aggressive		mildy to fairly aggressive		mildy to fairly aggressive		<750	>750		
Recommendations	Leaching Corrosion Counter Measures				Class 1 concrete, inorganic coating		Class 1 concrete, inorganic coating		Class 1 concrete, inorganic coating					
	Spalling Corrosion Counter Measures				Class 3, 4 or 5 concrete, inorganic coating		Class 4 or 5 concrete, inorganic coating		Class 3, 4 or 5 concrete, inorganic coating					
	Chloride Corrosion Counter Measures for rebar				Minimum 25mm concrete cover over rebar		Minimum 25mm concrete cover over rebar		Minimum 25mm concrete cover over rebar					
Domestic Water Use Method		Langelier			0.1		-0.1		-0.1				>0	<0
		Ryznar			7.6		7.8		7.7				<6.5	>6.5
		Corrosion Ratio			5.4		1.7		1.3					>0.1
		Aggressive Index			11.9		11.7		11.8		10 to 12	<10		

	INDEX	DESCRIPTION
Final Index	0-350	non to mildly aggressive
	350-750	mildy to fairly aggressive
	750-1000	highly aggressive
	1000	very highly aggressive

The results of all three samples indicate mildly to fairly aggressive conditions, based on concrete aggressiveness method (Basson).

Finally based on the chloride corrosion index, the recommended concrete thickness around all rebar is a minimum of 25mm.

The resting water level at the deep geotechnical hole was 18.97m, which is below the recommended piling depth.

9.6 Foundation and Development Recommendations – Sirius

The pile installation methods considered for Sirius solar farm include displacement steel piles, displacement ductile iron piles and pre-bored steel piles. Exosun is proposing to implement a Exotrack® HZ V2 solar panel system with the following pier configuration:

- Central pile:
  - 160x80x4 Rectangular hollow steel section,
  - IPE 200 (Equivalent to W8 x 10) I-section,

- End piles:
  - C170x60, or
  - IPE 160 (Equivalent to W6 x 9) I-section,

Due to the presence of calcrete, gravel, shallow bedrock, and abundant pebbles, cobbles and boulders, steel displacement piles will likely reach shallow refusal and could deform as a result. The DPSH probes reached refusal at shallow depths across the site Sirius. The SPT N-value readings generally exceed 20 before reaching refusal. Displacement steel hollow piles are generally unsuccessful in soils with a SPT N-value exceeding 20.

The alternatives are pre-drilling steel piles or circular hollow ductile iron displacement piles (TRM or similar approved). The latter has been used successfully at the Khi Solar One plant, approximately 2km north of the sites, where circular hollow ductile iron displacement piles (TRM piles) were driven, open ended, through the alluvial and "socketed" into the underlying bedrock. Similarly, pre-drilled steel piles could be founded in the underlying bedrock resulting in minor vertical settlement.

9.6.1 Loading

The loading imposed on the piles include the self-weight of the panels and piles, wind load imposed on the panels and the load applied by the daily tracking unit. The typical maximum loads applied on the piles were obtained from a document provided by the supplier titled

"Exotrack HZ® foundations loads According to SANS 10160". The loads include the following:

- Vertical Direction:
  - $Fv_{central\_down}$ = 7000 N (downward load on central pile),
  - $Fv_{central\_uplift}$ = -2500 N (uplift load on central pile),
  - Fv_{end_down} = 6000 N (downward load on end pile),
  - Fv_{end_uplift} = -2500 N (uplift load on end pile),
  - $Fv_{motor\ support}$ = 400 N (downward on motor foundation).
- Horizontal Direction:
  - $Fh_{central}$ = 11 000 N (central pile),
  - $Mth_{central}$ = 2400 N.m (moment of central pile),
  - $Fh_{end\ pile}$ = 3000 N (end pile),
  - Fh_{motor} = 900 N (motor foundation).

These loads were applied to theoretical formulae in order to estimate the vertical settlement horizontal displacement rotation and minimum embedment depth of the piles. All calculations were carried out according to the Eurocode 7 Design Approach 1b standards.

9.6.2 Material Classification and Excavation Conditions

Excavation classes and rock classifications for the subsurface materials occurring across the investigated area have been estimated using the definitions provided in the SANS 1200D (1988) and SANS 1200F (1988) standards. The excavatability and rippability of the underlying material were assessed by evaluating borehole logs and the shear wave velocities obtained from the CSW tests. The S-wave velocities for the CSW tests conducted at Dyasonsklip1 is shown in Figure 9.3.

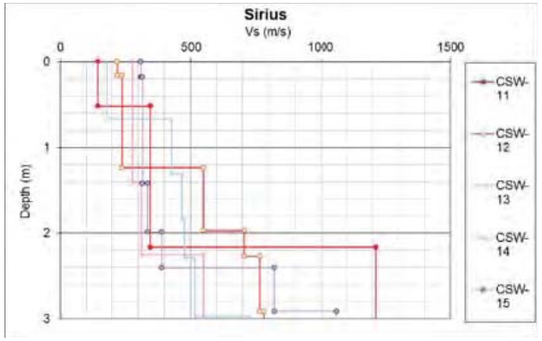


Figure 9.3: CSW test results at Sirius

Table 9.12 shows the excavation classes and rock classifications expected for the subsurface material across the site.

Table 9.12: Excavation Classes for the subsurface material at Sirius

Site	Depth (m)	S-wave Velocity (m/s)	Class of Excavation (SANS 1200D)	Rock Classification (SANS 1200F)
Sirius	0.0m – 1.2m	200m/s< Vs< 400m/s	Soft to Intermediate Excavation with Class A boulder excavation	-
	0.8m – 3.0m	400m/s< Vs< 1200m/s	Intermediate to Hard Excavation with Class A boulder excavation between 1.2 -1.5m	R2 to 2m, R3 to 3m

9.6.3 Material Parameters and Assumptions

The particle size analysis of the alluvial material show that the horizon consists predominantly of a non-plastic sand, with abundant gravel, calcrete, cobbles and boulders in a dense consistency. According to the work done by Gibson (1953) a typical material with these characteristics will exhibit an internal friction angle (ϕ') of 30°.

The following assumptions were made in order to conduct further calculations:

- A unit weight of 18 kN/m^3 was assumed for the alluvium.
- The Poisson's Ratio (*v*) of the subsoils is in the order of 0.3
- The interface friction angle (δ') were assumed to equal to 0.75ϕ',

- The lateral earth pressure coefficient was assumed to be equal to the lateral earth pressure coefficient of earth pressure at rest, $K/K_0=1$.
- The piles were assumed to be open ended.

9.6.4 Bearing Resistance

The minimum pile embedment depth was calculated for the central and corner pile as a function of the base and shaft resistance. The shear strength of the subsurface material, the effective normal stress acting on the piles and the pile properties contribute to the overall bearing resistance of the subsurface material. The bearing resistance formulae as given by Knappet and Craig (2012) are as follow:

Base resistance:

$$Q_{bu} = A_p(N_q \sigma'_q)$$

Shaft resistance:

$$Q_{su} = P_p \int_0^{L_p} \tau_{int}\,(z)dz$$

Where: A_p is the area of the pile cross-section,

N_q is a bearing capacity factor depended on the slenderness and friction angle of the subsoils,

σ'_q is surcharge above the pile base,

P_p is the pile perimeter,

L_p is the length of the pile.

τ_{int} is the shear strength along the interface of the pile and the soil.

9.6.5 Embedment Depth

The minimum embedment depth was calculated by assessing two conditions. For the first condition it is assumed that the bedrock is situated well below the pile base and the second condition when the pile base is situated within competent weathered bedrock.

Table 9.13 indicates the condition that the bedrock is situated well below the pile base, the minimum embedment depth ranges between 2.55m and 2.90m for limiting vertical loads of 7kN in the case of a central pile and 6kN in the case of end piles.

Table 9.13: Minimum embedment depth of pile founded in alluvium

Pile Section	Minimum Embedment Depth (m)	Compression Bearing Resistance ($Q_{bu}+Q_{su}$) (kN)	Tension Bearing Resistance (uplift wind load) (kN)
Central Pile			
Steel Hollow Pile 160 x 80 x 4mm	2.90	7.2	4.8
IPE 200	2.55	7.2	4.5
170 x 9 Circular Hollow (TRM)	2.60	7.0	3.6
End Pile			
IPE 160	2.60	6.0	3.9
C160 x 65	2.70	6.0	4.0
118 x 7.5 Circular Hollow Pile (TRM)	2.90	6.0	4.7

Table 9.14 indicates the second conditions where the pile base is founded within the competent bedrock the pull-out tension load will govern the embedment depth. The minimum embedment depth was found to range between 1.9m and 2.2m at a pull-out load limit of 2.5 kN.

Table 9.14: Minimum embedment depth of piles founded on weathered bedrock

Pile Section	Minimum Embedment Depth (m)	Compression Bearing Resistance ($Q_{bu}+Q_{su}$) (kN)	Tension Bearing Resistance (uplift wind load) (kN)
Central Pile			
Steel Hollow Pile 160 x 80 x 4mm	2.1	-	2.5
IPE 200	1.9	-	2.5
170 x 9 Circular Hollow (TRM)	2.2	-	2.5
End Pile			
IPE 160	2.1	-	2.5
C160	2.1	-	2.5
118 x 7.5 Circular Hollow Pile (TRM)	2.1	-	2.5

It is recommended that pile load tests are conducted at the onset of the construction phase in order to assess and verify the recommended embedment depths.

9.6.6 Settlement

The maximum estimated settlement was calculated in the event that the bedrock is situated well below the pile base level. The settlements were determined by applying the analytical techniques by Randolph and Wroth (1978) method and the T-z Finite Differences method. These methods require the follow input parameters:

- Young's Modulus (E) of the pile,
- Shear Modulus ($G_{0.1\%}$) distribution of the subsoils,
- Poison's Ratio of the surrounding soils (*v*)
- Dimensions of the piles

- Pile Force.

Young's Modulus (E) of a typical steel pile was selected to be 200 x 10^3 GPa, as per the Southern African Steel Construction Handbook (Sixth Edition, 2008). The maximum amount of vertical settlement will occur at the central pile as a result of a vertical load of 7000N.

The operative level of strain for a foundation structure is 0.1%. The shear modulus ($G_{0.1\%}$) values at 0.1% strain were determined by applying a softening function proposed by Rollins et. Al. (1998) to the shear modulus data, Figure 9.4, overleaf, shows the minimum estimated shear modulus ($G_{0.1\%}$) to a depth of 3m.

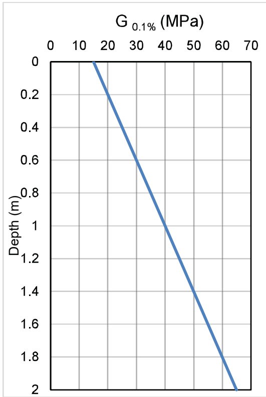


Figure 9.4: Minimum Shear Modulus of Subsoils

Figure 9.5 shows the pile load and settlement response to a depth of 3m. The total vertical settlement was estimated to be 1mm. The settlement of the piles is not sensitive to the marginal dimensional changes of the pile sections.

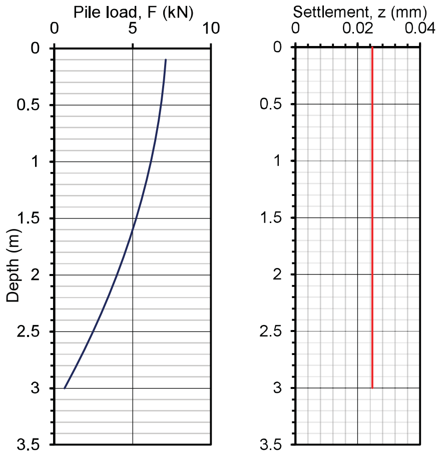


Figure 9.5: Settlement response for a pile to a depth of 3m

9.6.7 Horizontal Displacement and Rotation

The horizontal displacement was estimated by applying elasticity theory solutions. The horizontal displacement and rotation at the pile head, as given by Knappet and Craig (2012), are as follow:

Horizontal Displacement:

$$h = \frac{\left(\frac{E_p}{G_c}\right)^{1/7}}{p_c G_c}\left[\frac{H}{1.85L_c} + \frac{M}{0.83L_c^2}\right]$$

Rotation:

$$\theta = \frac{\left(\frac{E_p}{G_c}\right)^{1/7}}{p_c G_c}\left[\frac{H}{0.83L_c^2} + \frac{M}{0.16p_c^{-0.5}L_c^3}\right]$$

Where: E_p is Young's Modulus of the pile,

G_c is the median shear modulus,

p_c is the homogeneity factor describing shear modulus with depth,

L_c is the pile embedment depth,

H is the horizontal load, and

M is the moment applied around the head.

The horizontal displacement (h) at the pile head was calculated to be a maximum of 2mm with a rotation (θ) of 2°.

9.6.8 Motor support

The motor support foundation is required to carry a vertical dead load of approximately 400N and a horizontal wind load of approximately 900N. The load combination applies an eccentric stress to the subsurface soil. It was assumed that the foundation is situated on the alluvium with a horizon thickness of approximately 1.5m. A square foundation was assumed with a width (B) and length (L) of 1m.

The operative level of strain for a foundation structure is 0.1%. The settlement of the foundation is dependent on Young's Modulus (E) of the subsoils at 0.1% strain. From the CSW results the lowest measured Young's Modulus ($E_{0.1\%}$) at 0.1% strain was selected for the settlement calculations. The Young's Modulus ($E_{0.1\%}$) was taken 50 MPa. The maximum amount of settlement is estimated to be 0.121mm. The amount of settlement across the width of the foundation is shown in Figure 9.6. The settlement will occur immediately during construction.

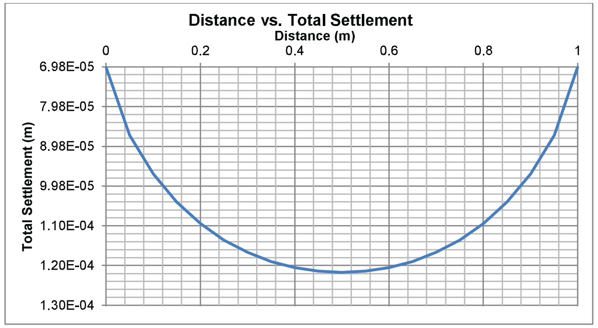


Figure 9.6: Total Settlement of the motor support foundation

9.6.9 Pile Installation

Displacement (Driven/rammed) steel hollow piles will not be feasible for over 90% of the development areas, owing to the presence of calcrete, cobbles and boulders. Furthermore, displacement piles will likely refuse before reaching a satisfactory embedment depth of 2m, "socketed" into weathered bedrock, in order to withstand uplift loads imposed by the wind. Small portions (about 10%) Sirius could be suitable for the use of displacement piles, but this has to be confirmed by pile load test at the onset of the construction phase.

The pier configuration recommended by Exosun is feasible provided that the holes are pre-drilled to a minimum embedment depth of 2m, "socketed" into weathered bedrock. A Rectangular 160x80x4mm section or a IPE 200 (W8 x 10) section is feasible for the use as a central pile and the C170 section is feasible for the use as an end pile provided that the above conditions are adhered to.

It is recommended that a pre-drilled displacement pile installation method is implemented, where the holes are pre-drilled to a minimum embedment depth of 2m with the pile base founded in weathered bedrock. After a hole has been drilled, a well graded fill material is placed and compacted from the base to the existing ground level. The pile section is then driven, open ended, to the minimum embedment depth. The diameter (D_0) of the boreholes should fall into the range of $1.00>L/D_0>1.25$, where L is the largest cross-sectional dimension of the pile. The well graded soil mixture used to refill the hole should be placed and compacted to 93% modified AASHTO at an optimum moisture content of -1 to +1 in, not exceeding, 300 mm thick layers. The grading of the fill material must be verified on site by a qualified geotechnical professional. It is essential that quality testing is carried out on site to ensure that the specified compaction is achieved. Typically a commercially-sourced G5 material is utilised, with a measure of fine-grained, alluvial onsite material mixed in to provide as a binder. It is recommended that, should the appointed contractor select this installation method that a series of varying soil mixes with differing percentages of G5/alluvial fines be submitted for laboratory testing to ascertain the most favourable density combination. A wet mix will always lead to greater compaction than dry mixing, and should be considered for the on-site installation.

Alternatively, circular ductile iron piles, open ended, should be driven to "socket" into the weathered bedrock. This method has been proven to be successful at the Khi Solar One plant, situated about 2km north of the sites. A typical 170 x 9mm section will be suitable to use as a central pile and a 118 x 7mm section for an end pile.

It is recommended that pile load tests are conducted at the onset of the construction phase in order to assess and verify the recommended embedment depths and to finalize the selections of the most effective pile types and pile installation procedure.

The interpretation of geotechnical conditions across the site is based upon the point information provided by the various investigation positions. Conditions intermediate to these have been inferred by interpolation, extrapolation and the use of professional judgement, to gain an overall evaluation of the geotechnical conditions likely to be encountered during construction. In the event, during the pre-drilling of the piles, of significant deviations from the inferred conditions being encountered, then it is recommended that the services of a

geotechnical professional are sought, as the recommended embedment depth may need to be exceeded.

10 CONCLUSION

The geology of the area is characterized by a thin, generally less than 1m, reworked calcrete and silty gravelly sand to nodular hard pan calcrete underlain by Gneiss, Schist and Granite of the Areachap Group of the Namaqua-Natal Metamorphic province. Rock outcrops of Gneiss, Schist and Granite may occur at surface. Excavatability of the material is expected to require *intermediate to hard* excavation techniques across the study area.

Based on the available information, the sites are suitable for the proposed development of the Solar Farm. It is foreseen that rammed piles, as recommended by the preferred supplier, will be a suitable option for the proposed foundations. The available geological information indicates that the use of screwed piles will not be suitable for the construction of the proposed foundations due to the presence of hardpan calcrete which will resist installation. Extensive pre-drilling over more than 90% of the sites will however be required in the installation process of the piling, owing to the presence of the hardpan calcrete.

The extent of the investigations undertaken is deemed adequate, within the time and budget constraints, to present an overview of the geotechnical conditions across the investigation site.

It must be borne in mind that the overall interpretation of geotechnical conditions is based upon point information derived from the respective test positions and that conditions intermediate to these have been inferred by interpolation, extrapolation and professional judgement. The interpretation of the geotechnical conditions is based on information available at the time of drafting this report.

--oOo--

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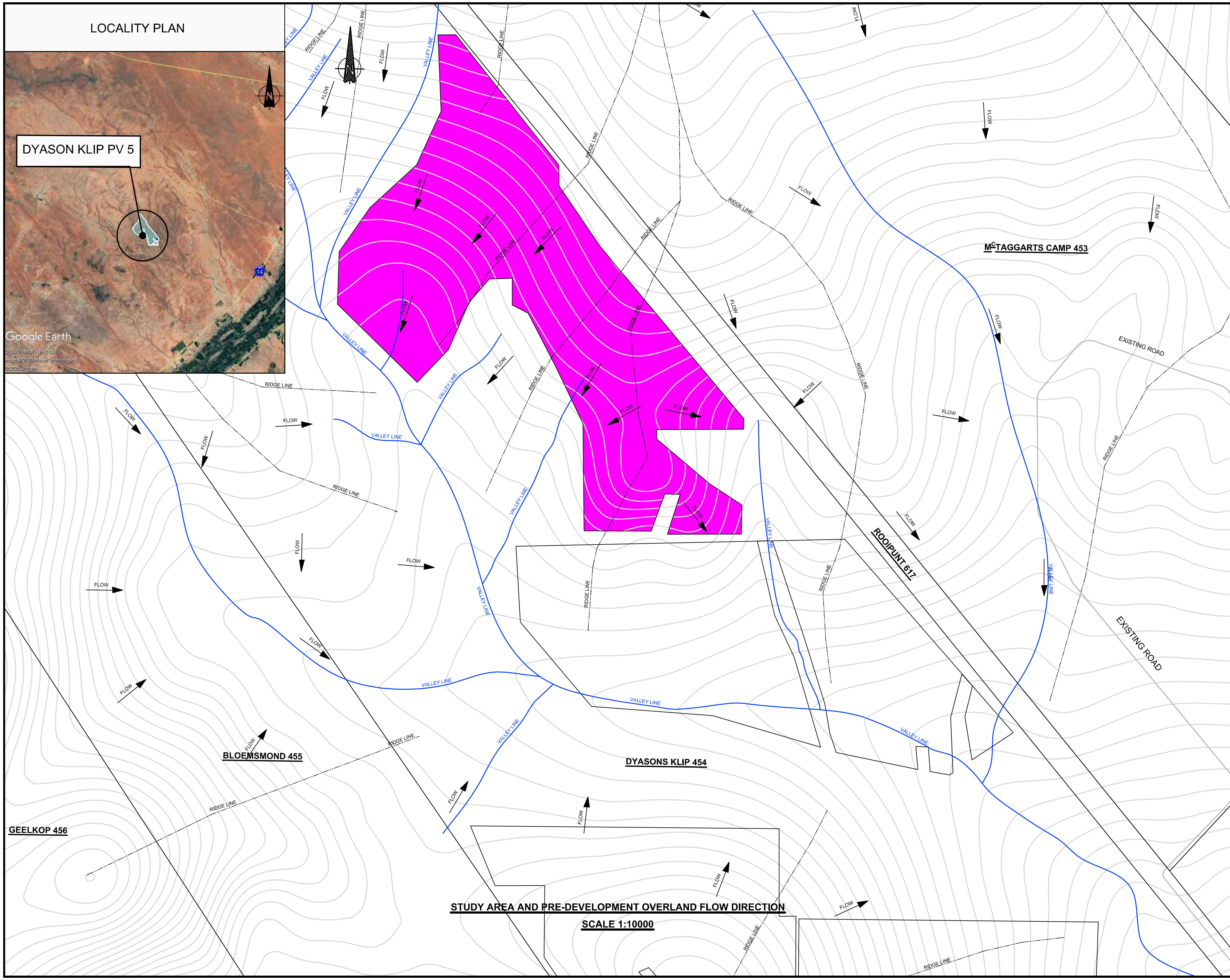
APPENDIX B: EXTERNAL & INTERNAL ROAD LAYOUT / PRE & POST DEVELOPMENT FLOW LAYOUTS



GENERAL NOTES

1. THIS DRAWING PROVIDES INFORMATION RELATED TO THE FORM OF THE WORK WHEN COMPLETED TOGETHER WITH GUIDANCE, WHERE FUNDAMENTAL TO THE DESIGN, FOR THE SEQUENCE OF CONSTRUCTION. THIS DRAWING DOES NOT PURPORT TO INCLUDE ANY NECESSARY PRECAUTIONS, ITEMS OR COMPONENTS REQUIRED FOR CONSTRUCTION SAFETY. ALL SUCH PRECAUTIONS MUST BE TAKEN AND ITEMS OR COMPONENTS SUPPLIED BY THE CONTRACTOR.
2. ANY DISCREPANCIES ARE TO BE CONFIRMED WITH THE ENGINEER ON SITE PRIOR TO WORK PROCEEDING.
3. ONLY WRITTEN DIMENSIONS SHALL BE USED.
4. ALL DIMENSIONS ARE IN ....

LEGEND:

- PROPOSED SITE
- EXISTING ASPHALT/GRAVEL ROAD
- VALLEY LINE
- RIDGE LINE
- STORMWATER CHANNEL
- DETENTION POND
- ALTERNATIVE PROPOSED ROAD
- PROPOSED EXTERNAL ROAD
- PROPOSED INTERNAL ROAD

REFERENCE DRAWINGS

Revision Details	By / Chk	Date	Rev
ISSUED FOR INFORMATION	TP / MLM	28.05.20	A

51 WESSEL ROAD
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SiVEST

Project Title

DYASON KLIP PV5

Drawing Title

STUDY AREA & PRE-DEVELOPMENT OVERLAND FLOW

Scale: AS_SHOWN — ON ORIGINAL PLAN 0 10 20 30 40 50

DRAWN BY: T PROPHET DATE: 05/2020

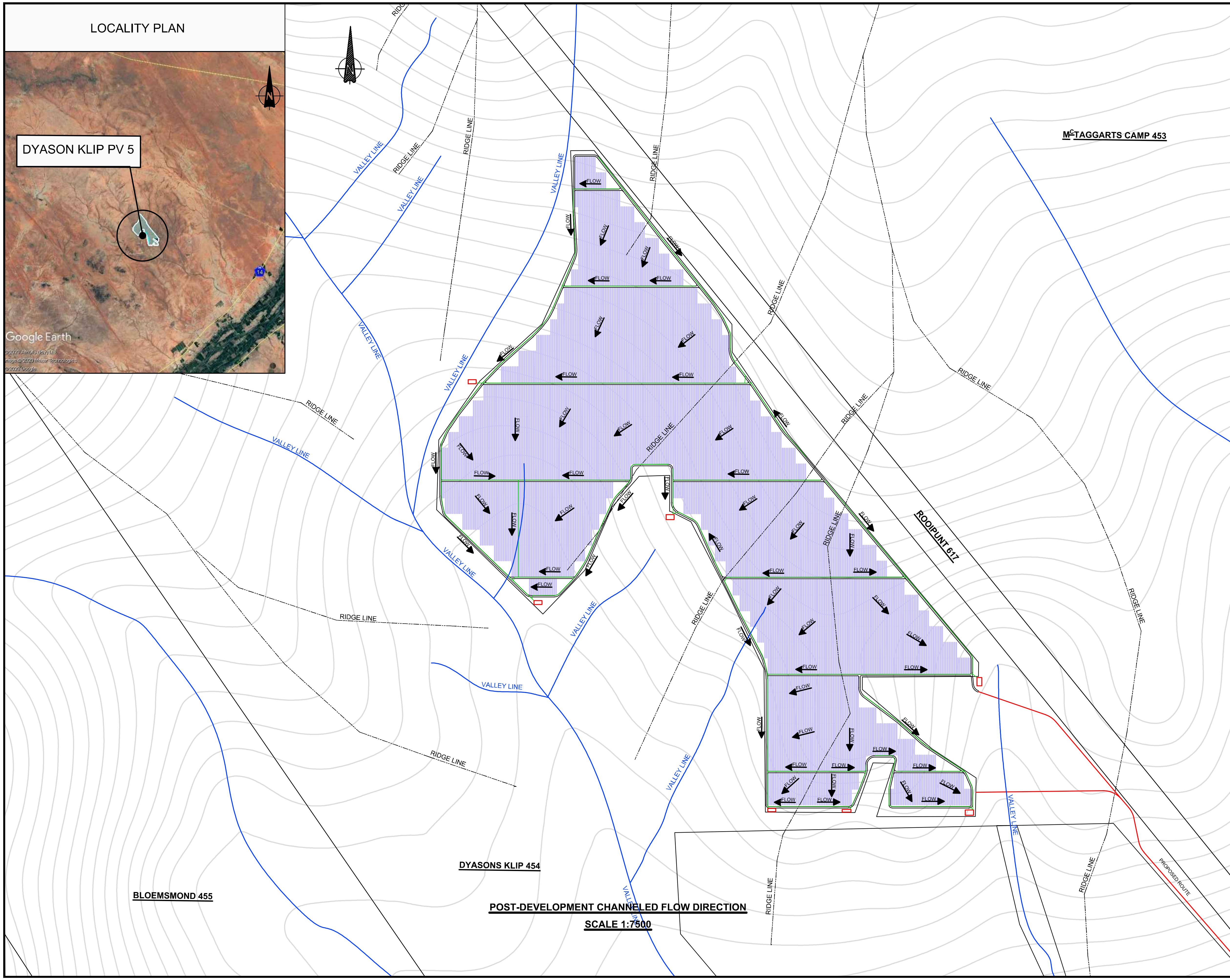
DESIGNED BY: R HIRST Pr No: -

SIGNATURE: DATE:

CHECKED ON BEHALF OF SIVEST BY: R HIRST Pr No: -

SIGNATURE: DATE: 05/2020

SiVEST Project Number	SiVEST Drawing Number	Revision
16145	5300	A



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- STORMWATER CHANNEL
- DETENTION POND
- ALTERNATIVE PROPOSED ROAD
- PROPOSED EXTERNAL ROAD
- PROPOSED INTERNAL ROAD

REFERENCE DRAWINGS

Revision Details	By / Chk	Date	Rev
ISSUED FOR INFORMATION	TP / MLM	28.05.20	A

51 WESSEL ROAD
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SOUTH AFRICA

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Project Title

DYASON KLIP PV5

Drawing Title

POST-DEVELOPMENT CHANNELED FLOW

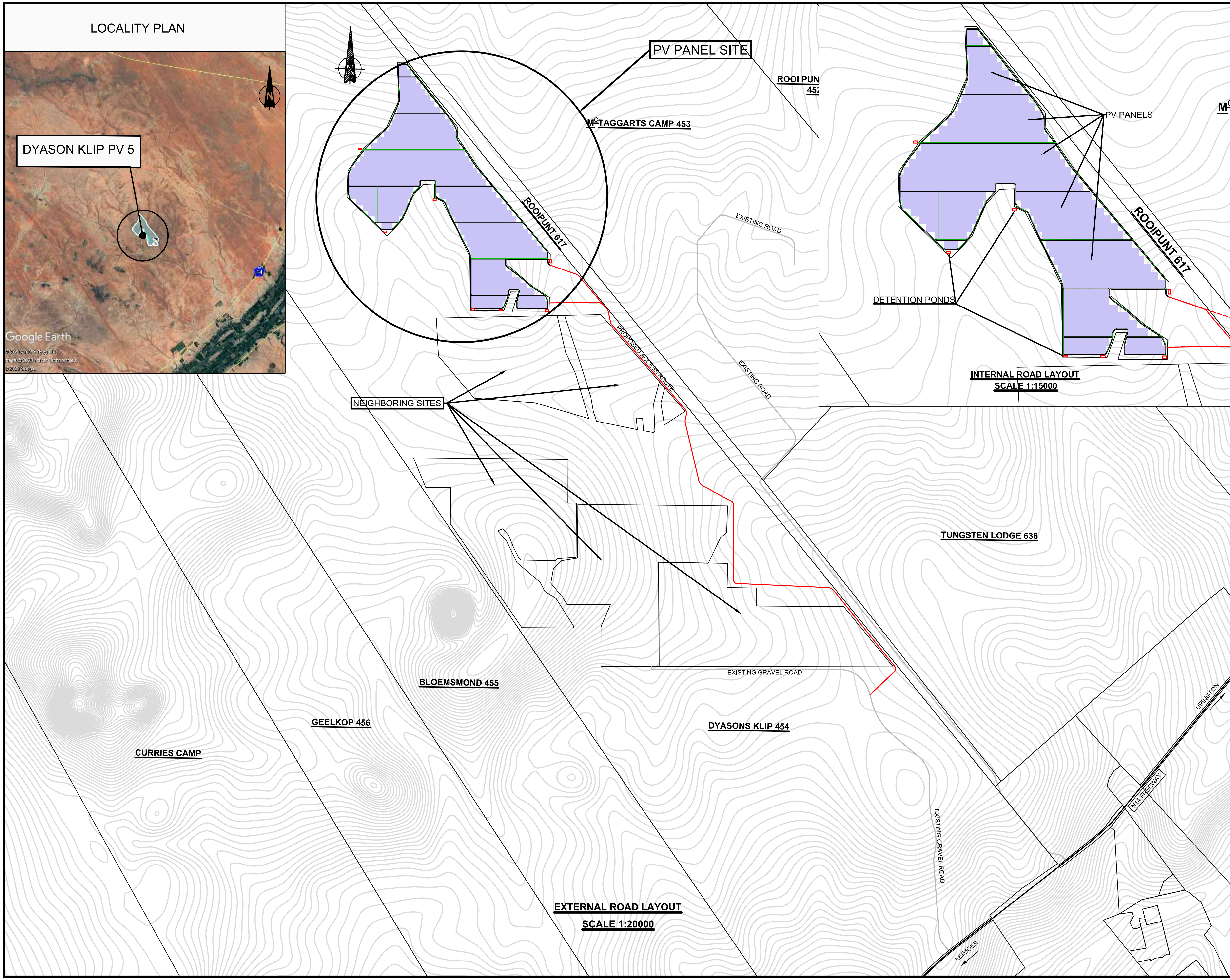
Scale: AS_SHOWN
ON ORIGINAL PLAN 0 10 20 30 40 50

DRAWN
BY : T PROPHET DATE: 05/2020

DESIGNED
BY : R HIRST Pr No: *
SIGNATURE : DATE :

CHECKED ON BEHALF OF SiVEST
BY : R HIRST Pr No: *
SIGNATURE : DATE : 05/2020

SiVEST Project Number	SiVEST Drawing Number	Revision
16145	5301	A



THIS DRAWING HAS BEEN PREPARED UNDER THE CONTROLS ESTABLISHED BY THE SiVEST QUALITY MANAGEMENT SYSTEM AND MEETS THE REQUIREMENTS OF THE SETA QUALITY GRADING SYSTEM THAT IS ISO COMPLIANT

GENERAL NOTES

1. THIS DRAWING PROVIDES INFORMATION RELATED TO THE FORM OF THE WORK WHEN COMPLETED TOGETHER WITH GUIDANCE, WHERE FUNDAMENTAL TO THE DESIGN, FOR THE SEQUENCE OF CONSTRUCTION. THIS DRAWING DOES NOT PURPORT TO INCLUDE ANY NECESSARY PRECAUTIONS, ITEMS OR COMPONENTS REQUIRED FOR CONSTRUCTION SAFETY. ALL SUCH PRECAUTIONS MUST BE TAKEN AND ITEMS OR COMPONENTS SUPPLIED BY THE CONTRACTOR.
2. ANY DISCREPANCIES ARE TO BE CONFIRMED WITH THE ENGINEER ON SITE PRIOR TO WORK PROCEEDING.
3. ONLY WRITTEN DIMENSIONS SHALL BE USED.
4. ALL DIMENSIONS ARE IN ....

LEGEND:

- PROPOSED SITE
- EXISTING ASPHALT/GRAVEL ROAD
- VALLEY LINE
- RIDGE LINE
- STORMWATER CHANNEL
- DETENTION POND
- ALTERNATIVE PROPOSED ROAD
- PROPOSED EXTERNAL ROAD
- PROPOSED INTERNAL ROAD

REFERENCE DRAWINGS

Revision Details	By / Chk	Date	Rev
ISSUED FOR INFORMATION	TP / MLM	28.05.20	A

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Project Title

DYASON KLIP PV5

Drawing Title

EXTERNAL & INTERNAL ROAD LAYOUT

Scale: AS_SHOWN — ON ORIGINAL PLAN 0 10 20 30 40 50

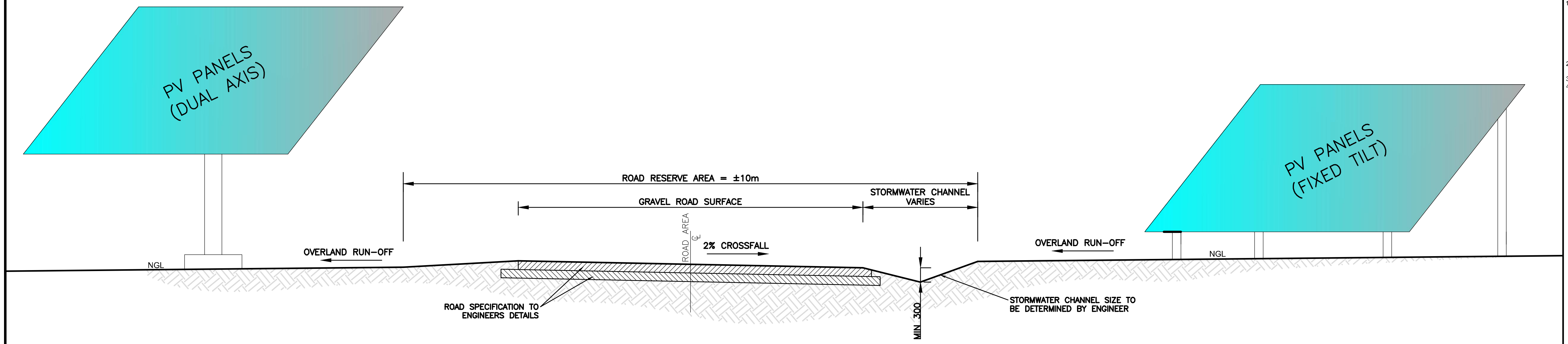
DRAWN
BY : T PROPHET — DATE: 05/2020

DESIGNED
BY : R HIRST — Pr No: *
SIGNATURE : — DATE :

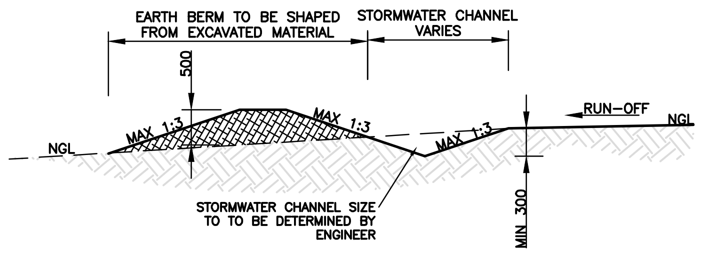
CHECKED ON BEHALF OF SiVEST
BY : R HIRST — Pr No: *
SIGNATURE : — DATE : 05/2020

SiVEST Project Number	SiVEST Drawing Number	Revision
16145	5500	A

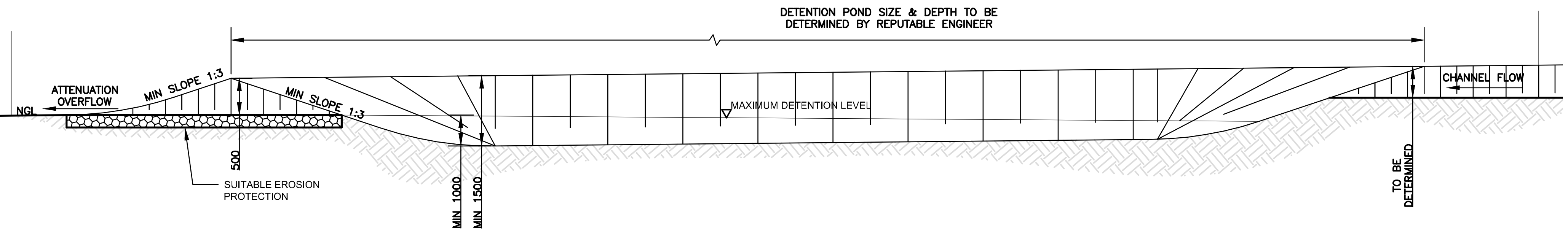
APPENDIX C: TYPICAL DETAIL



TYPICAL DETAIL OF GRAVEL ROAD
SCALE 1:1000



TYPICAL BERM DETAIL
SCALE 1:1000



TYPICAL SECTION THROUGH DETENTION POND
SCALE 1:1000

THIS DRAWING HAS BEEN PREPARED UNDER THE CONTROLS ESTABLISHED BY THE SiVEST QUALITY MANAGEMENT SYSTEM AND MEETS THE REQUIREMENTS OF THE SETA QUALITY GRADING SYSTEM THAT IS ISO COMPLIANT

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4. ALL DIMENSIONS ARE IN ....

REFERENCE DRAWINGS

Revision Details	By / Chk	Date	Rev
ISSUED FOR INFORMATION	TP / RH	28.05.20	A

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SiVEST

Project Title

DYASON KLIP PV 5

Drawing Title

TYPICAL DETAILS

Scale: AS_SHOWN	ON ORIGINAL PLAN 0 10 20 30 40 50
DRAWN BY : T PROPHET	DATE: MAY_2020
DESIGNED BY : R HIRST	Pr No: •
SIGNATURE :	DATE :
CHECKED ON BEHALF OF SiVEST BY : R HIRST	Pr No: •
SIGNATURE :	DATE :

SiVEST Project Number	SiVEST Drawing Number	Revision
16145	5302	A

SiVEST Civil Engineering Division

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