Final Report to Solar Reserve on a Geotechnical Investigation for the Proposed CSP Solar Power Project on the Farm Humansrus No 469 (Groenwater), Northern Cape Province

Reference : 12-718R03

Date : 15 August 2012

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Final Report to Solar Reserve on a Geotechnical Investigation for the Proposed CSP Solar Power Project on the Farm Humansrus No 469 (Groenwater), Northern Cape Province

Reference : 12-718R02	Dated : 25 June 2012

EXECUTIVE SUMMARY

A detailed geotechnical investigation has been completed at the farm Humansrus in the Northern Cape for the purpose of developing a 75 MW concentrated solar power (CSP) solar farm. The investigation comprised the excavation, profiling, sampling and testing of test pits together with the drilling, logging, sampling and testing of diamond rotary-cored boreholes and supplemented by geophysical surveys and DPSH penetrometers. These results have been supplemented by existing geotechnical data collected for the proposed PV development to the south and east of the proposed CSP site.

The entire circular-shaped site has been located between the proposed PV areas.

The results have shown that the entire site in general is characterised by either densely-packed boulders of core-stones of medium hard rock lava or intensely-fractured to non-intact, weathered medium hard rock lava. The highly disturbed and variable ground conditions have been attributed to a documented fault that passes through the area, in combination with a possible palaeo-river valley and synclinal folding of the underlying bedrock (see Fig 1).

In terms of the development potential and ease of installation of foundations (specifically for lightly-loaded heliostat panel frames and the more substantial structures proposed in the power block area) the area can be dived into 3 shallow (TLB) geotechnical zones and 3 deep (borehole) geotechnical zones and the following comments, conclusions and recommendations can be made in the table below and represented in Figs 2 and 3:

Geotechnical Zone	Test Points (No)	Area Coverage (%)	Characteristics
S1	11 TP's	37	Shallow refusal TLB (<1.0 mbgl)on dense boulders
S2	15 TP's	50	Intermediate refusal TLB (1.0-1.5mbgl)
S3	4 TP's	13	Deep refusal TLB (>1.5 mbgl)
D1	13 BH's	50	Boulders (0 to 3m max) over saprolitic residuum
D2	7 BH's	27	Thick boulders (6 to 10m) over saprolitic residuum
D3	6 BH's	23	Shallow (<1.5 mbgl) highly fractured bedrock

- *The main foundation load in the Heliostat field area to be resisted is the uplift due to wind at between -10 and -20 kN.*
- *This can be achieved by the dead-weight afforded by conventional spread foundations in the form of beams (possibly pre-cast RC sections).*
- *An alternative foundation option could be the installation of rammed mini-piles (3No triangular pattern in place of each ground beam) possibly only in Zones S3 and D1.*
- *The most appropriate pile type installation (due to the shallow boulders or core-stones or intensely fractured weathered lava) would be percussion-bored mini-piles or Titan piles.*
- *The heavier and more settlement-sensitive receiving tower can be more appropriately socketed into the shallow highly fractured or boulder lava using a mass concrete spread foundation at 6 m below surface. Due to the high variability of the ground conditions, even within the restricted power block area, it is highly recommended that the excavations for major foundations (ie CR tower) be inspected to determine any further remedial ground improvement measures (eg grouting).*

Definitions and Abbreviations

Commercial:

MSJ	Moore Spence Jones (Pty) Ltd

Technical:

CH	Chainage (metres)
mbgl	metres below ground level
mamsl	metres above mean sea level
NGL	Natural Ground Level
FL	Foundation Level
BH	Borehole
SPT	Standard Penetration Test
N	SPT N value (blows per 300 mm)
TLB	Tractor-mounted Loader Backhoe
IP	Inspection pit
TP	Test Pit
DPL	Dynamic Probe Light
DPSH	Dynamic Probe Super Heavy
EABC	Estimated Allowable Bearing Capacity
G1-G10	Standard classification of natural road building materials according to TRH 14
CBR	California Bearing Ratio
MDD	Maximum Dry Density (kg/m3)
OMC	Optimum moisture Content (degrees)
PI	Plasticity Index
LL	Liquid Limit
LS	Linear Shrinkage
RMR	Rock Mass Rating
GSI	Geological Strength Index
RQD	Rock Quality Designation
UCS	Unconfined Compressive Strength (MPa)
C (c')	Cohesion (kPa) – total stress and (effective stress)
Φ (Φ')	Friction Angle (degrees) – total stress and (effective stress)
Kv	Modulus of Subgrade Reaction (MN/mm or kPa/mm)
CFA	Continuous Flight Auger (pile type)
DCI	Driven Cast In situ (pile type)
DHG	Down-the-Hole-Geophysics
ReMi	
MassW	

Final Report to Solar Reserve on a Geotechnical Investigation for the Proposed CSP Solar Power Project on the Farm Humansrus No 469 (Groenwater), Northern Cape Province

Reference : 12-718R02	Dated : 25 June 2012

1 INTRODUCTION AND TERMS OF REFERENCE

Moore Spence Jones (Pty) Ltd (MSJ) was instructed by Mr Fernando Villanueva of Solar Reserve (in an e-mail dated 26 March 2012) to complete the above-mentioned investigation. The scope of works and costs are based on the MSJ quotation dated 30 January 2012 and referenced 12-718.01.

The site is the farm Humansrus No 469, near Groenwater, between Danielskuil and Postmasburg in the Northern Cape Province. The site is also close to Lime Acres and Finch Mine. The entire farm available for investigation is approximately 700 ha.

The proposed 75MW CSP Solar Power Project comprises a number of appurtenant structures necessary to convert solar radiation into electricity suitable for inclusion into the national grid. The main elements consist of heliostat panels fixed onto frames and anchored into the ground, an inverter, turbine and receiving tower occupying an area of approximately 11,000 m2 in the centre of the heliostat field.

This study follows up on a preliminary study of the entire site consisting of a desk top study and shallow test pitting (MSJ report ref 11-764.04). This investigation and report deals with the results of a more detailed secondary investigation comprising in-fill test pitting and deeper borehole drilling in the CSP area.

Table 1a: Summary of Borehole Requirements (Solar Reserve, 2012*)

BH No	X	Y	Depth (m)	Comments and Requirements
Power Block				
B1	3131354.08	35898.43	30	ReMi
B2	3131354.08	35965.21	15	Changed to 30m; ReMi; Standpipe; DHG**.
B3	3131354.08	36032.00	30	ReMi; Electrical Resistivity
B4	3131420.86	35898.43	15	ReMi; Electrical Resistivity
B5 (CR Tower)	3131420.86	35965.21	50	ReMi; DPSH; Standpipe; DHG
B6	3131420.86	36032.00	15	ReMi
B7 (Salt Tank)	3131487.65	35898.43	15	Electrical Resistivity; DPSH
B8	3131487.65	35965.21	15	ReMi; Electrical Resistivity
B9 (Salt Tank)	3131487.65	36032.00	15	DPSH
Heliostat Field				
B10	3132256.53	35348.06	15	
B11	3132256.53	35965.21	10	ReMi
B12	3136256.53	36583.36	15	Standpipe
B13	3131962.45	35679.61	10	
B14	3131958.21	36243.78	10	
B15	3131656.83	35347.06	10	
B16	3131656.83	36583.36	10	
B17	3131420.86	35679.61	30	Changed to 15m
B18	3131422.35	36243.78	10	
B19	3131184.89	35347.06	15	ReMi; Standpipe
B20	3131188.54	35965.21	10	
B21	3131184.89	35683.36	15	
B22 t-line	3132909.19	35965.21	10	Electrical Resistivity
B23 t-line	3131656.83	34753.48	10	Electrical Resistivity; DPSH
B24 t-line	3131656.83	37176.94	10	Electrical Resistivity; DPSH
B25 t-line	3130404.48	35965.21	10	Electrical Resistivity
B26 t-line	3130873.00	36263.00	10	

*As per RFP, Revision B (3 Jan, 2012).**DHG = Down-Hole-Geophysics

Table 1b: Summary of Test Pit Requirements (Solar Reserve, 2012)

TP No	X	Y	Depth (m)	Comments and Requirements
Power Block				
TP29	3131291.55	35965.21	3 or ref	
TP30	3131550.17	35965.21	3 or ref	
Heliostat Field				
TP1 to TP5	3132571.11	35148.28	3 or ref	Bulk sample; Lab tests for roads
TP6 to TP28	3130701.93	36243.78	3 or ref	

Revisions, adjustments and additions to the initial proposals include the following:

Table 1c: Summary of Amendments and Additions to RFP

Initial Proposal	Amendment/Addition	Reference
B2 to be 15m	Deepened to 30m	E mail F Villaneuva 19/04/12
B17 to be 30m	Shortened to 15m	E mail F Villaneuva 19/04/12
ReMi	MassW	
No piezometers	Standpipe piezometers added to B2, 5, 12 and 19	E mail F Villaneuva 19/04/12
	5No additional BH's in PV area (NW01-NW05)	E mail F Villaneuva 27/04/12
	Review of ground conditions at sub-station	Sms/verbal

2 GENERAL GEOLOGY AND EXISTING INFORMATION

The published geological map of the area (2822 Postmasburg, dated 1977) at a scale of 1:250,000 shows the site to be underlain in part by lava, agglomerate, chert and jasper of the Ongeluk Formation. The most important structural feature is a lineation interpreted from LANSAT imagery. **Furthermore, the 1:500 000 scale hydrogeological map (2722 Kimberley, dated 2003) also shows the presence of a prominent and persistent fault traversing the area (and coinciding with the above-mentioned lineation) in a north-northwesterly direction with a consequent higher potential for groundwater yield (>5l/s).**

The near surface pedogenic deposits will be represented by boulder and well-cemented hardpan ferricrete and calcrete (the latter which was not encountered on the site). This is finally covered by a discontinuous layer of red-brown or flesh-coloured wind-blown (Aeolian) sand which covers most of the central and south-eastern quadrant.

The regional topography of the area is essentially flat (1:300 or 0.3%) in a south-westerly direction from a high of 1550 metres above mean sea level (masl) to 1530 mamsl. A few non-perennial stream gullies have formed an immature undulating terrain to the south and the north-western quadrant comprises.

A Development Facilitation Act (DFA)-type desk study report was provided by MSJ, referenced 11-764.02 dated 30 May 2011 as well as a preliminary geotechnical report was also provided by MSJ, referenced 12-718R01 and dated 7 June 2012.

The project specification and load data supplied by Solar Reserve (Solar Reserve RFP, Rev B, Jan 3, 2012) is summarised below:

Table 2: Summary of Structural Loading and Allowable Settlements

Element	BH No	Load (tons)	Footing (m)	Bearing (kPa)	Max Diff Settlement (mm)
Turbine	B4	2,050	10x30	68	16
Salt tanks	B1, B3	215 kPa	Φ = 50	215	
Pre-fab buildings	B9	14	1.5x1.5	62	31
Transformers	B7	1,140	7.6x12.2	12	24
Water tanks	B6	1,140	Φ = 9.1 to 12.2	18 to 950	37
Pipe rack	B1-B9	45-140	1.5x1.5 to 3x3	156 to 200	37
Pipe rack	B1-B9	455-1,100	3x6 to 9x7.6	160 to 252	37
Heliostat	B10-B21	TBD	TBD	TBD	TBD
CR Tower	B5	20,450	Φ = 40	163	37
Cooling Tower	B8	635	9x18	39	44
Sub-station	BH7	TBD	TBD	TBD	TBD

3 INVESTIGATION METHODS

3.1 *Desk Study*

A number of regional geological studies have been consulted but there appears to be a paucity of detailed site specific geotechnical information.

A number of viable jasperite deposits have been confirmed which are presently being mined on a small scale by the landowner.

The published Hydogeological map (2722 Kimberley, 2003) at a scale of 1:500 000 indicates the presence of a prominent fault line traversing the area in a north-northwesterly direction (see Fig 1).

Weinert's 'N' value of the area is 20 and the climatic region is arid–warm-dry (TRH2, 1987). The annual rainfall is 300 to 400 mm and the mean annual surface temperature is 17.5 to 20.0º with a mean annual evaporation of 2200 to 2400 mm. Clearly, the site lies in a water deficit region. This combination suggests that only mechanical disintegration of the bedrock is to be expected (after Fookes *et al*, 1971) and thus thin residual soils are expected, if any.

The erodilibility index of the area is low at 16 to 20 (Verster and WRC, 1992).

Seismologically, the site is characterised by a seismic intensity of V on the Modified Mercalli Scale (MMS) with a 10% probability of being exceeded at least once in a 50 year period (CGS, 1992). This translates to a predicted maximum horizontal ground acceleration of less than 50 cm/s2 or 0.025g (CGS, 2003). Under these circumstances, the liquefaction hazard of the area is 'unlikely' with an associated peak horizontal acceleration of < 50 cm/s^2 (Welland, 2002).

The total lightening risk (estimated for 2006-2007), based on flashes per km2 and positive polarity lightening measurements, has been determined as severe (Gill, 2008).

According to SANS 10160-1989, the regional basic wind speed (3 second gusts for 50 year return period) is 40 m/s and the maximum mean hourly wind speed is 20 m/s.

3.2 *Inspection Pits*

Thirty test pits (marked TP1 to TP30) were completed mainly over the central portion of the site for the Concentrated Solar Power (CSP) development at the co- ordinated positions determined by Solar Reserve (see Fig 1). The inspection pits were excavated using a Terex 860SX tractor-loader-backhoe (TLB).

The depths of the inspection pits ranged from 0.65 m to 3.2 m, with an average refusal depth of 1.75 m below surface in stiff residuum or dense boulders of weathered bedrock. All inspections pits were recorded as dry.

Additional existing test pit information from the proposed PV area to the south and east has also been included.

A limited number of representative soil samples were recovered and tested at Simlab Soils Laboratory in Kimberley.

A summary of the soil profiles in the Power Block area and Heliostat Field area are presented below:

3.2.1 Power Block Area

Test pits TP29 and 30 were excavated to the north and south of the main Power Block area. The typical profile encountered is summarised below:

Table 3a: Summary of Typical Soil Profile at Power Block Area

Layer	Depth (m-m)	Description	Remarks
Hillwash	0-0.35	Slightly moist, reddish brown, intact, LOOSE, silty SAND with roots.	Potentially collapsible grain structure
Residuum	0.35-0.63	Slightly moist, olive green speckled black, intact, FIRM, partially ferruginised, sandy SILT with scattered cobble gravel (<75mm) of weathered lava.	
Residuum	0.63-1.25+	Slightly moist, olive green mottled red brown speckled black, intact, VERY STIFF, partially ferruginised, sandy SILT.	Difficult excavation

The two test pits experienced partial refusal in very stiff residuum with no seepage encountered.

3.2.2 Heliostat Area

The remainder of the test pits (TP1 to TP28) were excavated in the main Heliostat field area and consist of highly variable characteristics that can be generally grouped into two broad soil zones as follows:

Table 3b: Summary of Soil Profile for Soil Zone A: Residual Boulders

Layer	Depth (m-m)	Description	Remarks
Hillwash	0-0.6	Slightly moist, dark red brown, intact, LOOSE, silty SAND with roots.	Potentially collapsible grain structure
Residuum	0.6-1.3+	Numerous sub-rounded and sub-angular cobble gravel (<200mm) and boulders (<550mm) of weathered lava in a matrix of slightly moist, olive green, friable, sandy SILT.	Difficult excavation

This zone comprises 16No test pits (53%) and the refusal of the TLB ranged from 0.8 to 1.4m (average 1.3m) with a solitary deeper test pit TP5 to 3m.

Table 3c: Summary of Soil Profile for Soil Zone B: Clayey Residuum

Layer	Depth (m-m)	Description	Remarks
Hillwash	0-0.6	Slightly moist, reddish brown, intact, LOOSE, silty SAND with roots.	Potentially collapsible grain structure
Residuum	0.6-1.0	Slightly moist, olive green and yellow, intact and shattered and slickensided in places, STIFF to VERY STIFF, partially ferruginised, CLAY to sandy CLAY with scattered gravel of weathered lava.	Potentially medium expansive
Residuum	1.0+	Either highly fractured non-intact, highly weathered LAVA or cobble GRAVEL and BOULDERS of weathered lava.	Difficult excavation

This zone comprises 6No test pits (20%) with refusal depths ranging from 0.8 to 1.45m (average 1.0m).

The occurrence of the relevant test pits in each zone is generally scattered and does not lend itself to group zonation.

3.3 Laboratory Testing

A number of representative samples were collected from the inspection pit and core drilling operations and tested in the laboratories of Simlab in Kimberley, together with Soillab and Rocklab in Pretoria to determine the physical and mechanical properties of the transported and residual soils (grading, Atterberg Limits, compaction characteristics and collapse potential) and determine the uniaxial compressive strength of the bedrock. Copies of the test results are included in Appendix C and relevant existing results from the PV investigation have also been included where appropriate.

3.4 Core Drilling

Diamond rotary-core drilling was presented as an option in the power block area (specifically the tall 200 m tower structure), and to a lesser extent in the heliostat field, especially if rapid rammed mini-pile foundations are adopted to resist uplift forces are considered as a fast-track foundation solution to compare with standard mass or plinth foundations.

Even though the shallow inspection pits suggest that competent weathered boulders or corestones of weathered rock or firm residuum occurs at a shallow depth (providing a suitable foundation horizon for even medium-loaded structures), the main geotechnical constraint would be the minimum foundation key-in depth required to resist overturning or uplift moments. A number of boreholes were drilled to investigate the quality and integrity of the deeper soil profile and rock mass that would eventually comprise the foundation layers for the main structures, especially if the mini-pile or anchor-dowel solution is adopted. The boreholes were evenly distributed according to surface area at an approximate ratio of 1 BH per 27 ha.

The boreholes were all terminated at between 10 and 50 m below surface in highly fractured (possibly clay-filled joints), very soft rock to hard rock lava. The majority of the boreholes are characterised by unusually high core loss, probably due to a non-intact, clay-filled rock mass or boulder development. Minimal core loss and shallow (within 4.5 m depth), competent bedrock was encountered in four boreholes (B5, 12, 15 and 16 in the Heliostat Field area and B1, 5, 6 and 9 in the Power Block area).

Five additional boreholes (NW01-05) were drilled in the PV area to the south and existing borehole data from the PV investigation to the east and south have been included where relevant.

3.5 Dynamic Probe Super Heavy

Five dynamic probe – super heavy (DPSH) penetrometers were completed adjacent to boreholes B5, 7 and 9 in the Power Block area and adjacent to boreholes B23 and 24 in the Heliostat field area. All DPSH results show abrupt, shallow, pre-mature refusal at between 0.9 and 1.2 m below surface and generally reflects the SPT refusal depths encountered in the boreholes, probably on densely-packed boulders or highly fractured, non-intact lava bedrock.

3.6 Geophysical Surveys

A number of geophysical techniques have been utilised to investigate the in situ rock mass characteristics at, and between, the individual borehole locations.

3.6.1 MASW (Alternative to ReMi)

A Geometrics Geode was employed for the work using 24 geophones with natural frequency of 4.5 Hz. MASW surveys were carried out using a 3m geophone separation. The results are included in Appendix D.

3.6.2 Down-the-Hole Geophysics

These surveys were not possible due to collapse of the designated holes below the permanent casing.

3.6.3 Soil Resistivity

An ABEM Lund LS resistivity meter was used to collect the resistivity data at eight boreholes positions, namely B3, 4, 7, 8 in the Power Block area and boreholes B22, 23, 24 and 25 in the Heliostat Field area. Two perpendicular lines each 80 m long were set out at each site and a Wenner configuration was employed for the resistivity measurements. The results are presented in Appendix D.

4 SITE GEOLOGY

4.1 General Geology of the Area

The published geological map of the area (2822 Postmasburg, dated 1977) at a scale of 1:250,000 shows the site to be underlain in part by lava, agglomerate, chert and jasper of the Ongeluk Formation to the south west and also by brown jaspelite and crocodilite with alternating layers of shale and mudstone of the Asbestos Hill Formation along the eastern border.

The most important structural feature shown on this map is a lineation interpreted from LANSAT imagery that corresponds to a well-defined and persistent fault line shown on the Hydrogeological map. The site is also affected to a lesser extent by the north-east trending and south-west plunging Ongeluk-Witwater Synclinorium.

This detailed site investigation has shown that the lava bedrock is overlain by the following:

4.2 Aeolian Sand

This layer was encountered in the majority of the inspection pits to an average depth of 0.6m in the range 0.2 to 1.25m. The layer is generally slightly moist, red brown, very loose to loose, pin-holed and potentially collapsible, silty fine sand.

4.3 Clayey Residuum

This layer was encountered in six (20%) of the test pits evenly distributed around the Heliostat field area and two of the boreholes (B13 and B18) in the Heliostat area and one (B5) specifically in the Power Block area and comprised slightly moist, olive green, intact to shattered and slickensided, clay to sandy clay with scattered gravel. The layer occurs, on average, from 0.6 to 1.0m below surface with an average thickness of 1.6 m but has also been interpreted as occurring as clay-filled joints and fractures to significant depths.

4.4 Boulder Residuum

The majority of the boreholes (17No or 65%) encountered what has been interpreted as cobble gravel and boulders of weathered hard rock lava (possibly with matrix of clay which has not been recovered during the drilling process). The average depth and thickness of the boulder layer is 4.5 m in the range 1.1m to 7.5m (and rarely as deep as 10.9m in borehole B11).

4.5 Weathered Lava Bedrock

This layer was not encountered in any of the inspection pits due to shallow refusal on dense boulders or stiff residuum at an average depth of 1.2 m prevented this.

The first appearance of weathered bedrock is completely weathered, saprolitic lava and was encountered in most of the boreholes at between 0m and 10.9m. The table below shows the variation in rock strength description (based on tactile methods) with depth in the two main areas:

Table 4: Summary of Lava Bedrock Characteristics

Area	**Min depth (m)**	**Max depth (m)**	**Ave depth (m)**	**Description**
Power Block	0 2.1 3.1	3.65 6.0 30.0	2.0 4.2 12.1	Completely weathered saprolite Very soft rock lava Soft to medium hard rock lava
Heliostat Field	0.3 6.0 0.4	10.9 12.4 13.5	4.9 9.6 6.4	Completely weathered saprolite Very soft rock lava Soft to medium hard rock lava

However, four of the boreholes (B10, 17 and 25 in the Heliostat field area and B4 in the Power Block area) have revealed reverse weathering with saprolitic residuum **underlying** highly fractured, very soft to medium hard rock lava. This could be an indication of a possible inclination of the proposed fault that traverses the site (see Fig 1).

4.5.1 Power Block Area

The average rock mass conditions at this level have been described by using the geological strength index (GSI) and rock mass rating (RMR) as follows:

Table 5a: Summary of Parametric Input for GSI at “Shallow” Bedrock level

Parameter and rating	BH1 to 9
Intact rock strength (MPa)*	3-70+ (rating 1-7)
RQD** (%)	0-25 (rating 3)
Disc spacing (mm)	<60 (rating 5)
Disc condition	Clay & SRJ<1mm (rating 10)
Groundwater	Set at max rating of 15
Orientation	Set at max rating of 0
Total RMR rating	34-40
GSI (RMR-5)	***29-35***

*Estimated from rock strength descriptions until lab results concluded
**RQD = rock quality designation

In general the rock mass is Class IV or Poor Rock based on the information at shallow bedrock level of between 3.45 m and 6.4 m below surface.

Table 5b: Summary of Parametric Input for GSI Down to Key-in Foundation Level (CR Tower Only)

Parameter and rating	BH5
Intact rock strength (MPa)	3-25+ (rating 1-2)
RQD (%)	25-50 (rating 8)
Disc spacing (mm)	<60-200 (rating 5-8)
Disc condition	Clay & SRJ<1mm (rating 20+)
Groundwater	Set at max rating of 15
Orientation	Set at max rating of 0
Total RMR rating	49-53
GSI (RMR-5)	***44-48***

In general the rock mass occurring at the proposed key-in foundation depth of 6 m below surface and within the zone of influence of the 40 m diameter circular spread foundation of the CR Tower (ie an effective zone of influence of 1.5xB = 60m + key-in depth = 66m below foundation level) is Class III to IV or Poor Rock to Fair Rock.

The aforementioned GSI values have been used to determine the following rock mass parameters at various levels:

Table 5c: Input Parameters and Rock Mass Parameters at “Shallow” Bedrock Level

BH No	Input Parameters			Output Parameters			
	UCS (MPa)	m_i	GSI	Φ_{rm} (º)	c_{rm} (MPa)	E_{rm} (GPa)	σ_{rm} (MPa)
B1 to 9	3-5	15	29-35	29.3-30	0.1-0.11	0.3-0.7	0.3-0.4

UCS = unconfined compressive strength
Mi = Hoek-Brown constant
Φ = friction angle of the rock mass
Crm = cohesive strength of the rock mass
Erm = Elastic Modulus of the rock mass
σrm = Compressive strength of rock mass

Table 5d: Input Parameters and Rock Mass Parameters Down to Key-in Level (CR Tower Only)

BH No	Input Parameters			Output Parameters			
	UCS (MPa)	m_i	GSI	Φ_{rm} (º)	c_{rm} (MPa)	E_{rm} (GPa)	σ_{rm} (MPa)
B5 Only	3-25	15	43-48	33-35	0.1-1.1	1.1	0.4-4.4

In summary, the minimum allowable bearing capacity (MinABC) at “shallow” weathered bedrock level is approximately 300 kPa and at proposed key-in depth (+/- 6 m) is approximately 400 kPa. This latter bearing capacity provides a FoS = 2 and also concurs with the SPT refusal at the same depth in BH B5.

4.5.2 *Heliostat Field Area*

The average rock mass conditions at this level have been described by using the geological strength index (GSI) and rock mass rating (RMR) as follows:

Table 6a: Summary of Parametric Input for GSI at "Shallow" Bedrock level

Parameter and rating	BH10 to 26
Intact rock strength (MPa)*	3-70+ (rating 1-7)
RQD** (%)	0-25 (rating 3)
Disc spacing (mm)	<60 (rating 5)
Disc condition	Clay & SRJ<1mm (rating 10)
Groundwater	Set at max rating of 15
Orientation	Set at max rating of 0
Total RMR rating	34-40
GSI (RMR-5)	***29-35***

*Estimated from rock strength descriptions until lab results concluded
**RQD = rock quality designation

In general the rock mass is Class IV or Poor Rock based on the information at shallow bedrock level in the range 1m to 10.5 below surface.

The aforementioned GSI values have been used to determine the following rock mass parameters at "shallow" bedrock level:

Table 6b: Input Parameters and Rock Mass Parameters at "Shallow" Bedrock Level

BH No	Input Parameters			Output Parameters			
	UCS (MPa)	m_i	GSI	Φ_{rm} (º)	c_{rm} (MPa)	E_{rm} (GPa)	σ_{rm} (MPa)
B10 to 26	3-5	15	29-35	29.3-30	0.1-0.11	0.3-0.7	0.3-0.4

UCS = unconfined compressive strength
Mi = Hoek-Brown constant
Φ = friction angle of the rock mass
Crm = cohesive strength of the rock mass
Erm = Elastic Modulus of the rock mass
σrm = Compressive strength of rock mass

In summary, the minimum allowable bearing capacity (MinABC) at "shallow" weathered bedrock level is approximately 300 kPa. This estimated bearing capacity provides a FoS > 5 and also concurs with the average SPT refusal at between 1.5 and 4.5 mbgl in boreholes B10 to 26.

5 GROUNDWATER and SEEPAGE

No groundwater seepage was encountered in any of the inspection pits or boreholes drilled on the site. However, the use of drilling water and stabilising fluid has created a temporary artificial water level which is expected to subside over time and in accordance with the secondary permeability afforded by the rock mass. These water levels were measured on 17 May 2012 and ranged from 2.6 mbgl (BH B15) to 23.4 mbgl (BH B5 at the CR Tower site). The average reading in 20 of the boreholes was 8.3 mbgl and is expected to subside over time to at least 23 mbgl.

Published data shows the general water level is considered to be between 20 and 30 m below surface (DWAF & WRC, 1995 and DWAF, 2003) and this is in line with the farmers boreholes located on the site.

Table 7: Summary of Hydrogeological Parameters (DWAF, 1995 and 2003)

Hydrogeological Parameter	Value
Depth to water table	20 to 30 m
Aquifer Type	Intergranular and fractured
Borehole success (accessibility)	<40%
Borehole yield >2l/s (exploitability)	<10%
Actual borehole yield	0.1 to >5 l/s (in fault and geo contact)
Groundwater quality	70 to 300 mS/m or 501 to 1000 mg/l (TDS)
Quality and vulnerability	Minor aquifer of least vulnerability
Max amount BH yield allowed	2500 to 4000 m3/km2/annum
Mean annual recharge	1 to 5 mm
Mean annual precipitation	300 to 400 mm
Mean annual evaporation	2200 to 2400 mm

The published data also shows a significant increase in the general groundwater yield potential of between 0.5 and 1.0l/s throughout most of the site to >5l/s in the region of the fault.

The site is situated within the D73 tertiary sub-drainage region (DWAF, 1999).

With respect to stormwater run-off and infiltration, the generally flat and bowl-shaped topography of the site will not encourage the run-off and stormwater ingress onto the site and should be designed for and managed accordingly.

6 LABORATORY TEST RESULTS AND ANALYSIS

A number of samples were collected and tested at the SANAS accredited laboratory Simlab (in Kimberley), together with Soillab and Rocklab in Pretoria in order to determine the mechanical characteristics of the various soil layers and the unconfined compressive strength (UCS) of the bedrock. A summary of the results for the soil and rock component is included in the tables below and have been supplemented by the existing results from the PV investigation:

6.1 Soil Component

A summary of the foundation indicator, compaction and classification results is given in Table 8a below. The following conclusions and comments can be made:

- The majority of the materials encountered on site are coarse granular sand and gravel but with a component of fines (between 13 and 39%) such that the plasticity index (PI) becomes significant and influential in their classification.

- Due to this, the normally good Public Roads Administration (PRA or AASHTO) classification of A.2.4 (good to excellent subgrade quality) is downgraded to a G8 (according to TRH14) or worse due to the high PI in many instances.

- The best construction material is found at Detail 3 of the PV area (to the east of the Heliostat Field area) and Main Camp of the PV area (to the south of the Heliostat Field area) and qualifies as upper subbase. Within the main Heliostat Field Area similar good quality material (G6 or better) can be found at TP5, 21 and 25.

- All of the materials have been estimated (by the grading modulus or GM and PI relationship) as having potentially high California Bearing Ratio (CBR) values at 93% Modified AASHTO dry density (MADD) of between 7 and 80% (average 23%).

- Only two of the samples tested on site have a medium potential for heave (TP26, 0.2-0.8m and TP24, 0.4-1.1.

- Only two samples tested showed a LL in excess of 50% which may be an indication of a potential for consolidation settlement.

Table 8a: Summary of Foundation Indicators, Compaction and Classifications (Combined CSP and PV Area, 2010)

Location	DEPTH (m-m)	Particle Size (%)		Atterberg Limit *			GM	Modified AASHTO		CBR Values (%)					UCS	PRA (AASHTO)	TRH 14
		Coarse Sand	Silt & clay	LL	PI	LS		MDD (kg/m³)	OMC (%)	@ Compaction MDD (%)							
										90	93⁺	95	98	100			
Heliostat Field Area																	
TP3	0.0-0.1	7	31	26	10	5.5	2.33				46				GW	A.2.4(0)	G7
	0.65-0.9	20	45	52	22	11	2.13	1878	14.9	6	7	7	8	8	GM	A.2.7(4)	G9+
TP5	0.0-0.8	7	14	23	8	4.0	1.06				23				SC	A.2.4(0)	G6
	1.0-2.0	28	14	21	8	4.0	2.53	2273	7.5	16	27	38	64	90	GW	A.2.4(0)	G6
TP16	0.0-1.0	5	33	26	13	6.0	2.54				45				GW	A.2.6(4)	G7
	1.0-1.6	4	42	42	18	8.5	1.07				13				SC	A.2.7(4)	G8
TP18	0.9-1.9	14	52	89	24	12	0.82				8				MH	A.7.5(20)	G9
TP21	0.6-0.8	5	21	20	5	2.5	1.82				44				SM	A.2.4(0)	G5
TP24	0.0-0.6	3	38	18	10	5.0	0.91				18				SC	A.2.4(0)	G7
	0.4-1.1	5	43	37	23	12	0.93				9				CL	A.6(16)	G9
TP25	0.3-1.2	10	24	24	9	4.5	1.31				25				SM	A.2.4(0)	G6
TP26	0.0-0.2	60	40	25	9	4.0	0.61				15				CL	A.4	G9+
	0.2-0.8	7	43	45	23	12	1.42				12				CL	A.2.7(4)	G8
DETAIL 2 (Sub-station)																	
TP 25	0-0.6	8	32	43	21	10	2.19				23				GM	A.2.7	G8+
TP 25	0.6-0.7	18	35	58	21	11	1.42				13				SC	A.2.7	G8+
TP 25	0.7-1.7	55	22	52	17	8.5	2.30				30				SC	A.2.7	G8+
DETAIL 3 (East of Heliostat Field)																	
TP 33	0.6-22	36	30	59	29	15	2.12				14				SC	A.2.7	G6
TP 34	0-1.3	8	13	NP	NP	0	1.32	1917	8.3	3	7	12	26	44	SM	A.2.4	G9
TP 38	0.9-1.5	26	16	NP	NP	0	2.13				72				SM	A.1.b	G4
TP 41	2.1-2.8	26	18	NP	NP	0	2.33				80				GM	A.1.b	G4
TP 50	0.5-1.6	31	39	38	14	7.0	2.34				37				GC	A.6	G6
MAIN CAMP (South of Heliostat Field)																	
X1	0-0.5	7	25	NP	SP	1.5	1.12				37				SM	A.2.4	G7
X3	1.3-1.45	46	29	67	29	15	1.92				12				SM	A.2.7	G8+
X4	1.5	36	29	58	23	12	1.94				17				SM	A.2.7	G8+
X6	1.2-2.3	37	30	41	16	8.0	2.38				35				GM	A.2.7	G6

LL = liquid limit
PI = plasticity index
LS = linear shrinkage
GM = grading modulus
MDD = maximum dry density
OMC = optimum moisture content
UCS = Unified Classification System
PRA = Public Roads Administration

Table 8b: Summary of Shear Strength Parameters (PV Area, 2010)

TP No	Depth (m-m)	Dry density (kg/m3)	Cohesion (kPa)	Friction (deg)	MC (%)
Detail 3 (East of Heliostat Field)					
33	1.0	1522	3.1	22	20.3
38	0.4	1390	3.9	8.6	2.9
43	0.55	1456	19.0	16.8	2.6

MC = moisture content

In the event that either rammed or percussion-bored piles are used to anchor the panel frames, the shear strength parameters estimated above can be used to design the diameter and embedment depth required for the mini piles. The appropriate earth pressure coefficient in uplift is about 0.9.

Table 8c: Summary of Consolidation Parameters (PV Area, 2010)

TP No	Depth (m-m)	Dry density (kg/m3)	Mv (m2/MN)	E Modulus (MPa)	MC (%)	Settlement under 20 kPa (mm)
Detail 3 (East of Heliostat Field)						
33	1.0	1426	0.9	1.1	20.4	5-10
38	0.4	1479	0.5	1.84	4.5	2-5
43	0.55	1485	0.6	1.61	3.1	2-5

Mv = coefficient of volume change
E Modulus = elastic modulus

Seven samples have been tested with a LL>50% which is an indication of a potential to consolidation settlement under load. It can be seen that for a typical foundation beam 500 mm wide exerting a bearing pressure of not more than 20 kPa (dead plus live wind load), consolidation settlements of less than 10 mm can be expected in the long term.

A single collapse potential test was conducted on the upper Aeolian sand at TP 26 and provided a collapse potential of 1.7% at a load of 65 kPa and a dry density of 1514 kg/m3. The potential collapse settlement that should be expected will be in the range 5 to 10 mm.

Table 8d: Summary of Chemical Analysis (PV Area, MSJ 2010)

TP No	Depth (m-m)	pH	EC (mS/m)	Comments
DETAIL 2 (Sub-station)				
25	0-0.6	6.56	57.5	• Slightly acidic • Corrosive
25	0.6-0.7	6.63	62.8	• Slightly acidic • Corrosive
25	0.7-1.7	7.02	141	• Slightly acidic • Highly corrosive
DETAIL 3 (East of Heliostat Field)				
33	0.6-2.2	7.59	127	• Moderately alkaline • Highly corrosive
34	0-1.3	6.45	115	• Moderately alkaline • Highly corrosive
38	0.9-1.5	7.08	20.9	• Slightly acidic • Mildly corrosive
41	2.1-2.8	6.70	17.5	• Slightly acidic • Mildly corrosive
50	0.5-1.6	6.75	9.1	• Slightly acidic • Non corrosive

pH = potential for hydrogen
EC = electrical conductivity

The above results show that the shallow soils are generally slightly acidic and can be mildly to highly corrosive in a moist environment. However, the generally arid nature of the climate in the Northern Cape suggests that this may not be a problem and therefore substantial corrosion protection measures to buried steel (such as foundation reinforcement) will not be required.

6.2 Rock Component

Table 8e: Summary of UCS Rock Strength Test Results

BH No	Depth (m-m)	Geology	UCS (MPa)	Strength
Power Block Area				
B1	1.3.5-13.7	Lava	15.7	Medium hard
B17	3.54-3.74 9.91-10.07 14.81-15.00	Lava Lava Lava	273.2 272.5 310.5	Extremely hard Extremely hard Extremely hard
Heliostat Area				
10	12.45-12.72	Lava	544.7	Extremely hard
11	14.45-14.6	Lava	229.5	Extremely hard
12	3.6-3.8	Lava	2.7	Very soft (saprolite)
15	2.15-2.36 5.7-5.86 7.78-8.02	Lava Lava Lava	176.3 194.5 73.3	Very hard Very hard Very hard
17	3.54-3.74 9.91-10.07 14.81-15.00	Lava Lava Lava	273.2 272.5 310.5	Extremely hard Extremely hard Extremely hard
19	8.06-8.22 12.21-12.41	Lava Lava	70.4 155.1	Very hard Very hard
21	13.7-13.82	Lava	97.6	Very hard

UCS = unconfined compressive strength

It should be remembered that the values provided above are for the intact rock. The influence of the joints on the rock mass should be taken into account when estimating the allowable bearing capacity and the rock mass strength has been estimated in Tables 5c, 5d, 6c and 6d sections 4.5.1 and 4.5.2.

7 DEVELOPMENT RECOMMENDATIONS

7.1 Proposed Development and Recommended Procedures

It is understood that the proposed development will comprise the various components comprising individual heliostat pedestals, a salt converter and turbine, a main 200m high receiving tower together with access roads, services and an office component. Based on the shallow (TLB) and deep (borehole) results, the following broad geotechnical zones have been proposed:

Table 9: Summary of Geotechnical Zones and Characteristics (see Fig 2 and 3)

Geotechnical Zone	Test Points (No)	Area (%)	Characteristics
S1	11 TP’s	37	Shallow refusal TLB (<1.0 mbgl)on dense boulders
S2	15 TP’s	50	Intermediate refusal TLB (1.0-1.5mbgl)
S3	4 TP’s	13	Deep refusal TLB (>1.5 mbgl)
D1	13 BH’s	50	Boulders (0 to 3m max) over saprolitic residuum
D2	7 BH’s	27	Thick boulders (6 to 10m) over saprolitic residuum
D3	6 BH’s	23	Shallow (<1.5 mbgl) highly fractured bedrock

7.2 Trenchability and Rippability Assessment

The excavatability characteristics have been estimated from the performance of the TLB used for this investigation, supplemented by the previous investigation for the PV area. The average depth of the inspection pits is 1.1 m (at refusal) in the range 0.6 m to 1.9 m (and rarely to 3m at TP5). However, the site can be divided into the separate sites as follows:

Table 10: Summary of Refusal Depths of Inspection Pits (Combined CSP and PV Area, MSJ 2010)

Area	Min ref depth (m)	Max ref depth (m)	No IP's	SD	Average ref depth (m)	Comment (refusal < 1m)
Sub-station						
Detail 2	1.1	1.7	3	0.3	1.3	Nil
Main Heliostat Field						
TP1-28	0.75	1.9(3.0)	28	0.5	1.23	TP1, 3, 14, 15, 19, 21, 26 & 27
Power Block Area						
TP29&30	0.6	1.35	2	0.5	1.0	TP29
East of Heliostat Field						
Detail 3	0.5	3.0	13	0.9	1.9	IP 46 & 49
South of Heliostat Field						
Main Camp	1.2	2.3	7	0.5	1.7	Nil

SD = standard deviation

In general, only nine inspection pits (30%) experienced refusal at less than 1m depth.

The proximity of the above zones to one another reflects the generally undulating and sporadic nature of the occurrence of hard excavation and competent founding levels.

Trenching may be carried out with similar light earthmoving machinery as used in the investigation (i.e. TLB) to the final depths shown in the profiles (see Figure 1 and Appendix A).

In general, the majority of bulk earthworks and restricted excavations for service trenches and foundations should be classed as "soft excavation" according to South African Bureau of Standards SABS 1200D down to an average depth of 1.1 m, although there are isolated occurrences of shallow and even outcropping boulder excavation conditions that may require heavy ripping or even localised blasting (particularly evident at the substation area).

The minimum foundation key-in requirements for the major structures suggests an average foundation depth of 1.1 m below surface for the Heliostat panels and some of the power block structures, but a minimum of 6 m below surface for the CSP receiving tower. In this event, hard rock excavation and/or boulder excavation will be required below an average depth of 1.1 m below surface and will require heavy ripping, pneumatic assistance and even localised blasting in some areas.

The overall area reflecting a deeper excavatability to a maximum of 3m (with a minimum of obstructions to penetration), based on TLB refusal depths and drilling information is very limited. This may become important in the assessment of driveability or ramability of driven pile installations as a foundation option.

7.3 Drainage

The most important factor in the promotion of a stable site is the control and removal of both surface and ground water from the site. The natural ingress of groundwater and the additional localised inundation due to the development itself should be managed and controlled to prevent erosion and collapse settlement.

It is important that the design of the stormwater management system allow for the drainage of accumulated surface water. Such water should be directed towards the natural drainage lines such as streams and ditches on the margins of the site. Disposal of stormwater should in any case conform to the local authority's requirements and due to the large size of the site consideration will have to be given to attenuating the surface drainage water in engineered dams.

All drainage installations should be completed prior to construction.

7.3.1 Surface Drainage

Surface drainage of building platforms should be designed to direct water away from fill edges, to prevent overtopping of the fill crest and erosion of the fill embankment slopes. Surface water on these platforms should be directed to, and collected in, open lined drains or piped to the drainage line. It is important that grassing or protection of fill embankments be carried out as soon as possible after construction and in this site to also minimise ponding of the water to reduce slope instability and piping as the unconsolidated materials are permeable and moderately erodible.

Run-off from mirrors should be channelled and discharged into the appropriately designed stormwater attenuation dams.

Most importantly will be the collection, diversion and retention and/or disposal of concentrated stormwater from the steep hills and the main access road to the site.

7.3.2 Sub-Surface Drainage & Stormwater Retention

It is strongly recommended that the subsoil drains (if required) be designed according to the specific filter criteria of the *in situ* soils to prevent piping of the material and subsequent rapid erosion of material. In this respect (and based entirely on visual and tactile descriptions in the soil profiles), the upper soils are considered to be 'Zone 2' type soils (ie d_{85} > 0.075 mm and d_{50} <0.6 mm). Based on a d_{15} of between 0.03 to 0.04 mm, the permeability of the soil is between 10^{-6} and 10^{-7} m/s and thus the geotextile permeability requirement is between 10^{-5} and 10^{-6} m/s. In general, for subsoil purposes, an A2 or equivalent geotextile would be appropriate.

In addition to the above, the inherent permeability of the sub-surface has been estimated at each site from the laboratory test results and classifications to aid in the preliminary design of stormwater retention dams as follows:

Table 11: Summary of Variation in Permeability of Subsurface

Location	**Soil Classification (USCS)**	**Permeability (cm/s)**
Main CSP Power Block Area	GW to SC/SM	(2.7+/- 1.3) x 10^{-2} to (3+/-2) x 10^{-7}
Heliostat Field Area	GW to SC/SM	(2.7+/- 1.3) x 10^{-2} to (3+/-2) x 10^{-7}
Detail 2 (Sub-station)	SC to GM	3x10^{-7}
Detail 3 (East of Heliostat Field)	SC to GM	3x10^{-7}
Main Camp (South of Heliostat Field)	SM to GM	>3x10^{-7} to 7x10^{-6}

7.4 Earthworks and Site Clearance

7.4.1 Site Clearance

All vegetation should be cleared from the areas over which structures are to be built. This will include a significant number of densely-packed, medium sized scrub bushes.

7.4.2 Earthworks

From the generally gentle sloping nature of the site it is anticipated that cuts and fills will be of the order of less than 1 m but ground level variations over a short distance can attain a maximum of 2.5 m in isolated places.

It is recommended that all earthworks be carried out in accordance with the South African National Standards SANS 1200 (latest version).

In general, it is recommended that cut slopes and fill embankments have a maximum slope of 1 vertical to 2 horizontal to ensure stability. The need for the subsoil drainage both beneath and in fills if there are any will have to be assessed during the earthworks, taking into account the height and locality of individual fills.

The fills should be placed in layers not exceeding 200 mm loose thickness, and compacted to a minimum of 93% Modified AASHTO maximum dry density at 2% wet of optimum moisture content. **Boulders larger than $^2/_3$ of the layer thickness and clayey or organic material must not be included in the fill material. This would suggest that the boulder-sized fill material could be placed in layers not more than 450 mm thick, but should be restricted to the base of any significant compacted engineered fill. Alternatively a mobile crusher station should be considered.**

Both during and after construction, the site should be well graded to permit water to drain away readily and to prevent ponding of water anywhere on the ground surface. All terraces and earthworks in general should be sloped to a gradient of not less than **1 vertical in 50 horizonta**l to prevent ingress of water into the subsoils since these soils are significantly permeable. Surface drainage should be directed away from the crests of fill embankments to prevent over-topping and erosion of fill slopes.

7.5 Foundations

7.5.1 General

The evaluation of the founding conditions has been carried out for the proposed structures and is based on limited inspection pit profiling and borehole drilling combined with surface mapping only since the foundation indicator tests and UCS tests are still outstanding. **These evaluations should be confirmed by further inspections of foundation excavations during construction.**

The overall area reflecting a deeper excavatability to a maximum of 3m (with a minimum of obstructions to penetration), based on TLB refusal depths and drilling information, is minimal. This may become important in the assessment of driveability or ram-ability of driven pile installations as a foundation option.

7.5.2 Foundation Recommendations

The following foundation recommendations will be influenced by the type and loading of the structure and the location of the particular structure with respect to the varying ground conditions. The following design parameters are applicable to shallow spread foundations and deeper piles and have been estimated by empirical correlations with soil classifications:

Table 12: Summary of Provisional Foundation Design Parameters

Location	Soil Classification	Kv (MN/m3)	Cu (kPa)	Φ (deg)
Main CSP Power Block Area	GW to SC/SM	50-150	0 to 75	31 to >38
Heliostat Field Area	GW to SC/SM	50-150	0 to 75	31 to >38
Detail 2 (Sub-station)	SC to GM	25 to 135	0 to 75	31 to >34
Detail 3 (East of Heliostat)	SC to GM	25 to 135	0 to 50*	9 to >34*
Main Camp (South of Heliostat)	SM to GM	25 to 135	0 to 50	34

Kv = Modulus of subgrade reaction
Cu = undrained cohesive strength
*Contains actual laboratory test results

<u>Light to medium loads (Heliostat panels):</u>

The potentially medium collapsible conditions (1.7%) within the near surface hillwash is present over the majority of the site. The average thickness of the sand is 0.6 m. The actual effective potential collapse settlements have been estimated in the region of 5 to 10 mm. Foundations should be taken below this layer or alternatively the layer should be over-excavated and re-compacted in layers. A maximum allowable bearing capacity of 150 kPa would then be appropriate.

Light to medium loads (< 200 kPa) should utilise conventional strip or pad foundations below the potentially collapsible layer and very loose boulder layer and onto either dense boulder or firm residuum at an average nominal depth of 1.1 m below surface. However, due to the fast tracked nature of the foundation construction combined with the significant number of foundations required (estimated at 7No per one table of 5x20 modules of total length 40 m requiring tens of thousands of footings), pre-cast beams should be considered as a primary conventional spread foundation solution.

However, if rapid mini-pile installation is considered then the ramming method is not generally recommended due to the shallow hard boulder layer (average 1.1 m depth) over the majority of the site and the lack of expertise in the country (in general) and in the area (in particular). A more preferable piled foundation would be percussion-bored or drill and grout (generally not more than 300 mm diameter with a minimum embedment depth of 10 to 15 times diameter, ie 3 to 4.5 m depth). An embedment depth of 50% of this could be considered if two piles replaced each beam foundation (to be confirmed by design check).

It is essential, however, that the surface bed beneath any ancillary masonry-type structures be adequately prepared and pre-collapsed by either a) ripping and re-compaction in situ to at least 500 mm below surface or b) over-excavation to the base of the unconsolidated layer and re-compaction of the same material in layers not more than 150 mm.

The ground conditions at the proposed sub-station position are represented by TP31 and BH4 from the previous PV investigation (MSJ, 2010). Essentially competent weathered lava bedrock exists as nominal depth and no foundation problems are envisaged apart from the corrosive to highly corrosive subsoils.

Medium to heavy loads (power block structures including tower):
The majority of the structures will be classified as medium loads of between 167 kPa and 240 kPa. It is imperative that the column loads of these structures be transmitted to pad foundations taken down to the weathered lava bedrock level generally at an average depth of 3 to 4.5 m. However, the resistance to overturning requirement will require greater key-in depths of up to 6 m below surface for the CR Tower. This will require either over-excavation in densely-packed boulders or highly fractured, non-intact bedrock or anchoring of the shallow foundation by suitably designed and installed grouted dowels or piles.

The safe bearing capacity of very soft to medium hard rock lava is estimated at 300 kPa. However, at the greater proposed key-in depth of up to 6 m, this can be increased to 400 kPa.

7.6 Surface Beds

The upper hillwash soils classify as A.2.4 (PRA Classification) and SM (Unified Soil Classification) and are considered good to fair as a subgrade material, providing moderate subgrade strength with an estimated reconstituted CBR in excess of 10% at 93% Mod AASHTO density. The design value for the modulus of subgrade reaction (Kv) for floor slab design would normally be between 55 to 80 MN/m3 provided the subgrade beneath surface beds or floor slabs is ripped to a depth of 500 mm and re-compacted to 93% Mod AASHTO dry density.

7.6 Material Classification and Usage

The materials encountered on site have been classified in terms of their suitability for use in earthworks and road construction on the basis of field observations and laboratory test results. The classification is summarised in Table 13 below:

Table 13: Classification and Recommended Use of Materials

Material Description	Classification and Assessment of Materials	Recommended Use and Other Applicable Comments
Aeolian Sand	Dry, pale red brown, <u>very loose</u>, potentially collapsible, silty fine SAND. PI = 8-13 PRA/USCS = A.4-A.2.4 and SC-GW Material quality G6 to G7.	Fair in situ subgrade material. CBR = 15 to 45% @ 93% Mod AASHTO density. Potentially collapsible (1.7% estimated) and requires pre-collapse by ripping and re-compaction or founding below this level.
Boulder Residuum	Dry, pale grey, loose to very loose, gravel and cobbles (clast-supported, rough, hard, spherical, sub-rounded) of granite and quartz. PI = 5 to 24% PRA = A.2.4(0)-A.7.6 and SC/SM Material quality G5 to G9	Excellent subgrade material but will require extensive sorting and/or crushing on site. CBR = 8 to 44% @ 93% Mod AASHTO density. The boulder residuum can be suitable for use as a gravel wearing course material for roads if mixed with a binder.
Weathered lava	Pale grey speckled black, slightly weathered, medium to coarse-grained, laminated, friable, <u>soft to medium hard rock</u>, LAVA to saprolitic residuum. UCS = 2.7 to 300+ MPa	Good to excellent in situ subgrade material (but possibly too deep to be encountered). Could be used in general fill applications and layerworks up to subbase. Outcrop and sub-outcrop requires over-excavation beneath road prism.

All of the samples collected and tested show marginal and conditional suitability for use as wearing course gravel when compared to similar material in surrounding areas:

Table 14: Summary of Potential Wearing Course Sources (MSJ, 2009)

IP	Depth (m-m)	Shrinkage Product (Sp)	Grading Coefficient (Gc)	Category (CSRA, 1990)	Comments
Aeolian					
17	0.3-2.1	265	3	A	Erodible
Boulder Residuum					
37	0.7-2.4	56	32	B	Ravels & corrugates
Weathered rock					
19	0-0.4	0	14	B	Ravels & corrugates

In general, the chosen source of wearing course material should conform to the following:

Table 15: Recommended Specifications (mostly after CSRA, 1990)

Maximum size (mm)	37.5
Oversize Index (Io)	5% maximum
Shrinkage Product (Sp)	100-365
Grading Coefficient (Gc)	16-34
Soaked CBR (%)	>15 at 95% Mod AASHTO density
Trenton Impact Value (%)	20-65

It would appear that the mixing of the Aeolian sand with either the talus or crushed weathered rock gravel would probably provide sufficient binding and increase the shrinkage product value of the coarser-grained material to produce an acceptable gravel-wearing course for road construction around the site.

7.8 Recommended Subgrade Treatment

The wind-blown Aeolian materials are highly variable in thickness and generally appear as well graded fine silty sands. The estimated CBR of this subgrade at 93% MADD is in excess of 10%. The upper 500 mm should be ripped and re-compacted to 93% MADD, prior to the construction of layerworks.

This methodology should also be applied to those areas where road alignments or parking areas encounter sporadic rock outcrop. The rock should be over-excavated to allow a sufficient depth of material and layerworks to provide a consistent lateral continuity of material to minimise hard and soft spots.

7.9 Pipe Bedding

None of the in situ materials occurring on site meet the strict bedding specifications of SANS 1200LB but the grading of some of the talus and broken rock may be suitable (evenly graded between 0.6mm and 19mm) if a pipe bedding stiffness modulus (E') value of 7.5 MPa is adopted and if the finer material (possibly up to 10%) is sieved out.

8 CONCLUSIONS AND RECOMMENDATIONS

- This report contains the results of a geotechnical investigation carried out for the proposed concentrated solar power (CSP) Solar Power project at Humansrus, Northern Cape Province.

- **The investigation comprised the excavation of 30No test pits, 26No boreholes, 5No DPSH penetrometers and geophysical surveys including resistivity and MASW but the proposed DHG geophysical techniques could not be employed due to collapse of the designated boreholes below the installed casing. The results of these investigations have been supplemented by existing geotechnical data completed in 2010 for the surrounding PV area (MSJ, 2010).**

- **It is considered that conditions prevailing at the site during the site investigation are such that the site is considered suitable for the proposed development, provided the recommendations in this report are adhered to. The materials encountered on site are highly variable in structure, thickness, lateral extent, geotechnical characteristics and**

quality and as such, each structure and element of the development should be treated as a separate entity.

- **The majority of the site that could be accessed is underlain by wind-blown Aeolian sand, boulder residuum, saprolitic residuum and weathered, highly fractured lava. The recommended foundation options are below the potentially collapsible Aeolian sand (at nominal depth) for light to medium loads (< 200 kPa) and down to the dense boulder layer or weathered bedrock for medium to heavy loads and settlement sensitive structures, with an estimated allowable bearing capacity of between 300 and 400 kPa at an average depth of between 1.1 m (average TLB refusal depth) and 4.5 m (average first occurrence of weathered rock).**

- **The site can be divided into three shallow geotechnical zones (based on TLB data) and three deep geotechnical zones (based on borehole data) as follows and also shown in Figs 2 and 3:**

Geotechnical Zone	Test Points (No)	Area (%)	Characteristics
S1	11 TP's	37	Shallow refusal TLB (<1.0 mbgl)on dense boulders
S2	15 TP's	50	Intermediate refusal TLB (1.0-1.5mbgl)
S3	4 TP's	13	Deep refusal TLB (>1.5 mbgl)
D1	13 BH's	50	Boulders (0 to 3m max) over saprolitic residuum
D2	7 BH's	27	Thick boulders (6 to 10m) over saprolitic residuum
D3	6 BH's	23	Shallow (<1.5 mbgl) highly fractured bedrock

- **The main CR Tower structure will exert a bearing pressure of 163 kPa on a 40 m diameter circular base foundation placed at 6 m below surface. The rock mass at this level has been analysed and an allowable bearing capacity of 400 kPa has been determined and thus the FoS for bearing capacity failure is over 2.**

- **Alternatively, rapid mini pile foundation systems could be considered such as percussion-bored or drill and grout. The popular ramming method used in Europe would probably not allow sufficient key-in depth due to the presence of boulders and/or shallow weathered bedrock, and could only be considered in Zones S3 and D1.**

- The material encountered on site is highly variable at G5 to G8+ quality and selected layers will be suitable for general fill and selected layers up to upper subbase and possibly even specialised applications such as pipe bedding and wearing coarse gravel, but will require mixing and sorting.

- Finally, the ground conditions described in this report refer specifically to those encountered in the inspection pits on site. **It is therefore quite possible that conditions at variance with those discussed above can be encountered elsewhere.**

- **It is therefore important that Moore Spence Jones (Pty) Ltd carry out further site investigations during the construction phase**. Any change from the anticipated ground conditions could then be taken into account to avoid unnecessary design and expense. In this regard it is important that the construction phase of the project be treated as an augmentation of the geotechnical investigation.

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APPENDIX A

Inspection Pit Profiles

APPENDIX B

Borehole Logs

APPENDIX C
Laboratory Test Results

APPENDIX D
Geophysical Surveys

APPENDIX E
DPSH Penetrometer Results

SITE PLANS