



WesEng Consulting Pte Limited
Geotechnical Investigation Report

For

NTPC Limited, India

EFL Ground Mounted Solar Farm with BESS
at Seaqaqa Solar Farm at Seaqaqa, Vanua
Levu, Fiji



■ Draft Report
■ 08th June, 2026

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Executive Summary

This Geotechnical Investigation Report presents the results of a geotechnical / soil study in relation to a proposed Ground Mounted Solar Farm Development with Battery Energy Storage System on land legally known as Native Land Part of Lot 1, 3 & 4 M2726 Seaqaqa, Vanua Levu in Fiji. The development site comprises of land of approximately 225,387m² (22.5387 Ha). The site topography for the Proposed EFL Seaqaqa Solar Farm site is an undeveloped and unimproved raw land with varying contours, undulating landscape and flat plains along the Korovuli Creek, which forms the eastern boundary of the subject lease. NTPC Limited, India, is the Lead Consultant for the Energy Fiji Limited (EFL). NTPC Limited has selected WesEng Consulting Pte Limited as local Consultants to carry out the Topographical Survey and the Geotechnical Investigation.

The purpose of this Geotechnical Report is to:

- Assessing subsurface conditions,
- Identifying suitable foundation types,
- Slope stability,
- Earthworks suitability and analyzing soil corrosivity and resistivity to ensure the plant's long-term stability and performance.
- Determine the suitability of the site for sustainable energy infrastructure (solar farm).

The key points to note in relation to the assessment of the site are as follows:

- The existing natural ground has a minimum Allowable Bearing Capacity of 80 to 424kPa at varying depths for the site based on typical firm to stiff Silty CLAY characteristics (*refer to Section 3.2.5*).
- Adopt proper engineered stormwater drainage.
- Adopt civil engineering standards for excavation, batters, fill platforms, etc.

To conclude, given type of soil encountered in the testing, it is anticipated that the proposed subdivision development is suited for the site.



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1. Introduction

1.1. General

WesEng Consulting Pte Limited was commissioned by NTPC Limited, India at the beginning of January, 2026 to carry out the site studies which includes the geotechnical investigation and topographical survey for the EFL Seaqaqa Solar Farm. The site is located at Seaqaqa, Macuata, Vanua Levu. The site can be accessed off the Nabouwalu Road and is located on the outskirts of Seaqaqa commercial area. The site is approximately 25 kilometers (30-minute drive) from Labasa Town.

The site is iTaukei Land and is Leased under agreement to Energy Fiji Limited (EFL).

1.2. Scope of Work

The scope of the geotechnical assessment included:

- The surface and sub-surface conditions encountered on the site.
- The ground water conditions observed on site, recommend drainage design measures that will be required and stabilisation methods required in areas where underground water will be encountered.
- The thickness of the grass mat and any other highly organic materials.
- Determination of the Allowable Bearing Capacity.
- Discuss and assess for the different types of foundation
- The compressibility of any soft soil and determine the settlement of any fill materials.
- The maximum permissible batter slopes for cut and fills on the site and provides guidelines with respect to suitability of batter slope.
- Provide basic compaction control parameters for the design of fill areas including the precondition of materials prior to filling.
- The preparation of the subgrade foundation prior to placement of fill.
- The suitability of on-site materials for use as structural fills and recommends any remedial methods required to improve their suitability for filling.
- Seismic consideration in accordance with NZS 1170.5 (if applicable).



- The effect of the fill and other development on the boundaries of the site and recommend mitigation (if any) to stabilise the site.
- Discuss and recommend geotechnical testing and monitoring procedures to be followed during the construction period.



Photograph 1: Dynamic Cone Penetrometer (DCP) tests being carried out by WesEng Staff at Test location DCP/HA/SV 2 at EFL Seaqaqa Solar Farm Site.

1.3 Field Investigation

Based on the above Scope of Works, WesEng Consulting Pte Limited were to carry out the following:

- 64 No. Dynamic Cone Penetrometer (DCP) tests to a 3m – 5m target depths.
- 24 No. Hand Auger Borehole tests to a 3m target depth.
- 40 No. Trial Pits to a 3m – 5m target depths
- 218 No. Shear Vane (SV) tests carried out at 1m intervals till target depth of 3m or 5m from the natural ground level, depending on the test location.

A total of three (3) different onsite tests were carried out at each of the sixty-four (64) test locations from 29th January, 2026 to 11th February, 2026. Dynamic Cone Penetrometer Tests (DCP) together with Hand Auger (HA) or Trial Pit (TP) and Shear Vane (SV) tests were carried out at the proposed EFL Seaqaqa Solar Farm Site.



2. Site Overview

2.1. Location

The EFL Seaqaqa Solar Farm site comprises of 64 Ha of Native Land and is located along the Nabouwalu Road and is approximately 30 minutes' drive from Labasa town area. The site is also a 5-minute drive from Seaqaqa Commercial Area as shown on Figure 1

The site is surrounded by green fields, various vegetation and a few houses with the Nabouwalu Road situated on the north west end of the site. On the western end of Site is the Fairdeal Earthmoving Contractors Ltd depot while on the north and eastern end is Korovuli Creek (refer to Figure 1).

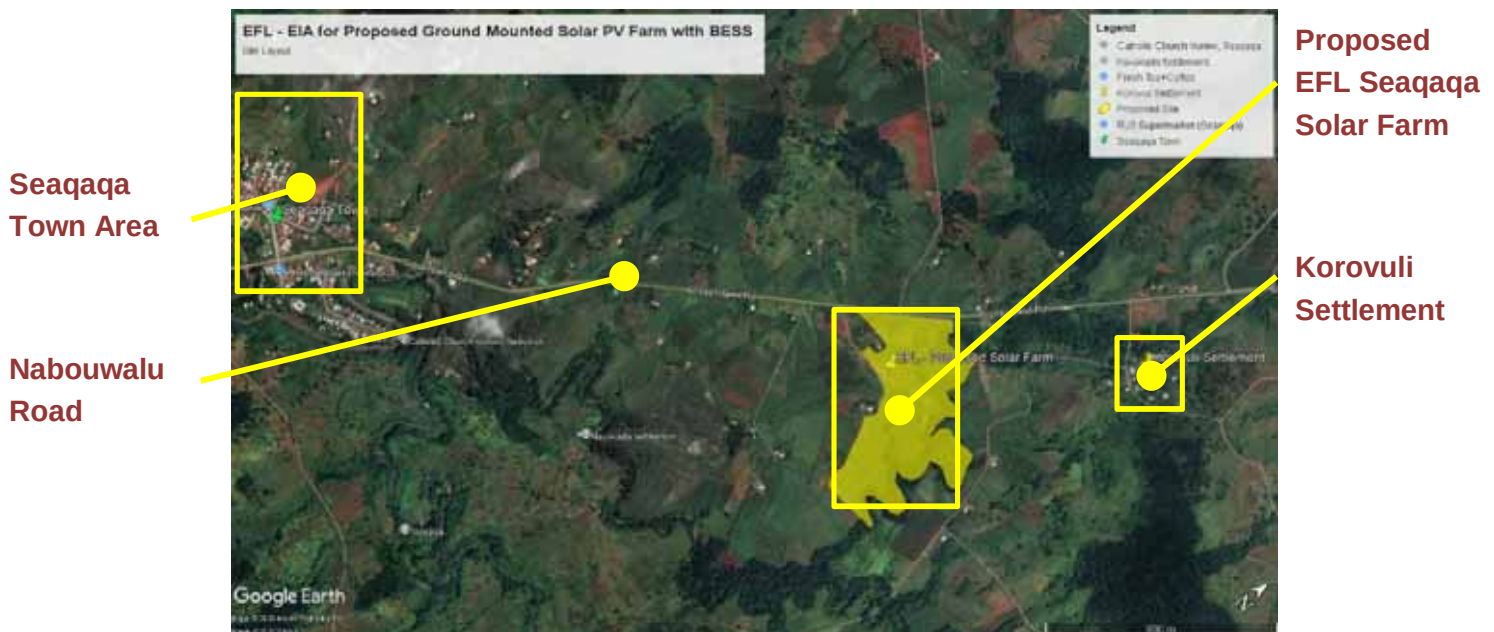


Figure 1: Google Image of the site and the surrounding properties.



2.2 Published Geology

Based on the Fiji Geological Maps – Sheet 5 of the Vanua Levu series – Geology of Seaqaqa Area, the soil in the site area is documented as Clastic sediments with interbedded flows and volcanic breccias as shown in the geology sheet below (*refer to Figure 2*).

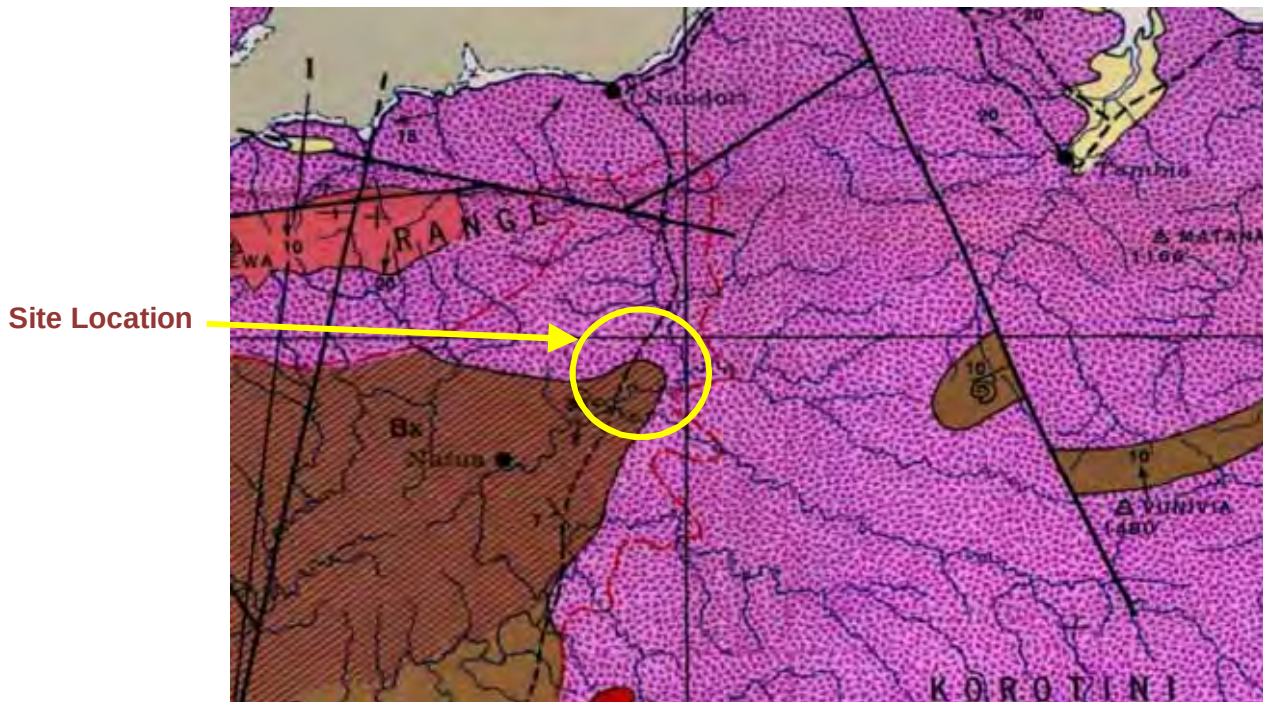


Figure 2: Geology Map of Development Site stating Clastic Sediments with interbedded Flows & Volcanic Breccias.



2.3 Site Location

Reference is made to the 1:50,000 Topographical Map of Lot 46, Natua (Part of), Seaqaqa, Macuata shown below, identifying the location of the Proposed EFL Solar Farm site (*refer to Figure 3*).



Figure 3: Site Locations as shown on the 1:50,000 Topographical Map

2.4 Topography

The site topography for the proposed site is an undeveloped and unimproved green field site with varying contours, undulating landscape, and flat plains along the Korovuli Creek, which forms the northern and eastern boundary of the subject site. The site is hilly to undulating with the Korovuli Creek and the various internal drainage paths, having a major impact on the site. It is apparent that the lower slopes and limited flat areas are utilised for subsistence farming (cassava and bananas) but flood prone due to stormwater originating from the Nabouwalu Road buried cross-culvert and upstream catchment areas (external of this site) (*refer to Drawing No. WEC930-G-001 contained in Appendix A*).

Topographical survey captures a broad range of spot heights including string definitions along the major changes in grades. Features such as creeks, elevated areas, prominent trees, fencing, access roads, embankments and visible constraints. Vegetation present on site consists of



sugarcane, medium to large trees - mango trees, coconut trees and medium to large shrubs. The lowest point is 83.0m (Eastern boundary of the site, adjacent to Korovuli Creek) and the highest point is 102.0m (central of the site) above Mean Sea Level (MSL) as shown on Drawing No. WEC930-G-001 and WEC930-G-002 contained in Appendix A.

2.5 Site Access

The site is accessible off Nabouwalu Road and approximately 25km away from the Labasa Town Central Business District (CBD) area. The project area is directly accessible from the Wainikoro – Dama Road, which abuts its north -western boundary.

2.6 Vegetation

The site is vegetated with grass and medium to large shrubs, grass, medium to large trees - mango trees and coconut trees etc, as shown on Photograph 2 and 3. The topsoil is mostly vegetated with grass and shrubs (*refer to Photograph No. 2 and 3*).



Photograph 2: Indicating the type of vegetation (medium to large grass & shrubs) and access road. Medium to large trees is evident on the site as well.

2.7 Drainage

The stormwater from both the main Cross Island Road and the Feeder Road connecting to it, all discharge to the Proposed Site by way of a cross culvert. The stormwater flows through 2 No.



open earth drains within the site, one connecting to the cross culvert from the Nabouwalu Road as mentioned above, and the other towards the centre where both open earth drains discharge into the Korovuli Creek on the Eastern Boundary of the site. With the Subject Area having vacant and undeveloped land with varying contours and an improper drainage system, the stormwater flows at a slow rate which causes flooding during heavy downpour.



Photograph 3: Photograph indicating the existing open earth drains running through the Northern side of the Subject Area adjacent to the Nabouwalu Road which discharges into the Korovuli Creek.

2.8 Proposed Development

The proposed development plan for EFL Seaqaqa Solar Farm Project will include of a Solar Farm development on approximately 64 acres of vacant and undeveloped land.

2.9 Seismicity (Fiji Building Code, 1990)

Based on a previous assessment of seismic risk within the Fiji Island group¹, the NTPC EFL Seaqaqa Solar Farm project is located within the medium-risk zone for seismic activity, with a corresponding zone factor of 0.6 for zone 6 (Medium risk)

National Building Code, 1990.



Figure 3a presents the seismic risk zoning map derived based on this data from the Fiji Building Code, 1990. However, in accordance with the NZS 1170.5, the Zone Factor Map by Stirling & Nguyen, 2014 has a corresponding Zone Factor between 0.3 to 0.35 (*refer to Section 4.7 for Zone Factor Map*).

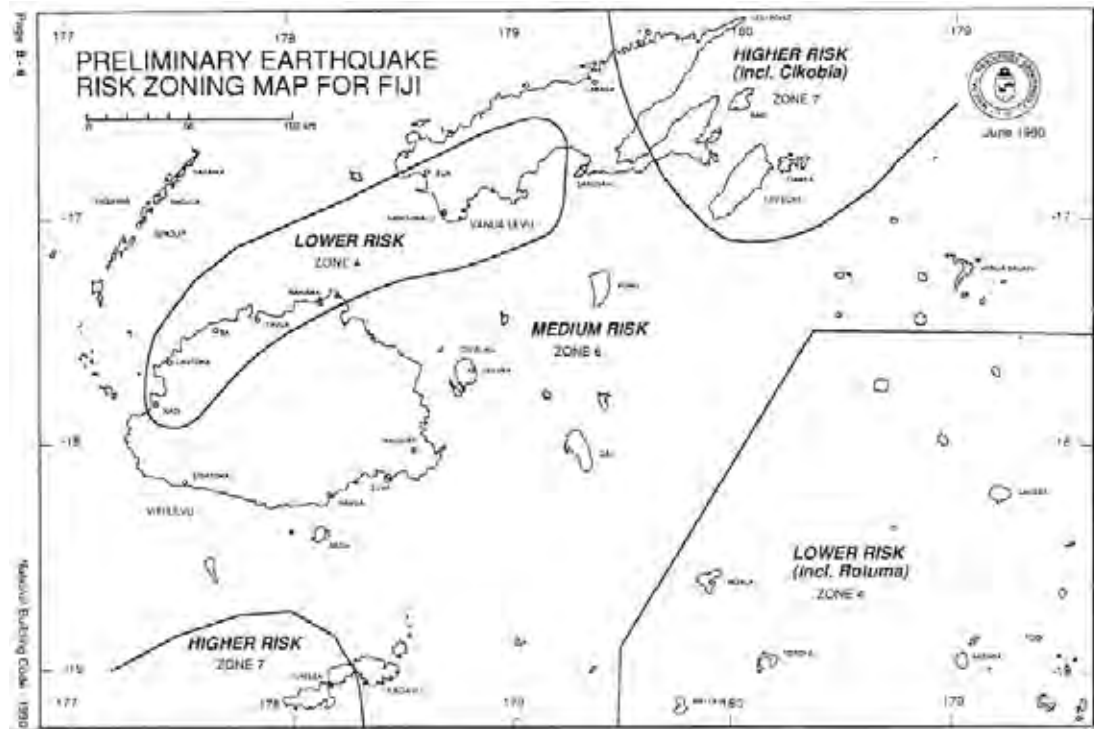


Figure 3a: Seismic risk zoning map³.



3 Site Investigations

3.1 Methodology

The fieldwork component of the geotechnical investigation was carried out from 29th January, 2026 to 11th February, 2026 and included:

- Carried out sixty-four (64) DCP tests ranging from a nominal depth of 0.0 to 5.0m (max) depending on test location or until refusal was encountered, below existing ground level.
- The drilling of twenty-four (24) Hand Auger boreholes (HA) utilising a hand auger (Ø50mm) including logging of the soil profile encountered ranging from a nominal depth of 0.0 to 3.0m or until refusal was encountered, below the existing ground level.
- The excavation of forty (40) Trial Pits (TP) utilizing an excavator including logging of the soil profile encountered ranging from a nominal depth of 0.0 – 5.0m or until refusal was encountered below the existing ground level.
- Carried out two-hundred and eighteen (218) SV tests at a depth of 3.0m for Hand Auger boreholes (HA) and 3.0m - 5.0m for the Trial Pits (TP).



Photograph 4: Photograph indicating the Hand Auger tests being carried out at Test Location DCP/HA/SV 57 at EFL Seaqaqa Solar Farm Site.



The DCP, drilling of boreholes and Shear Vane testing were carried out by two geotechnical testing teams comprising of one Engineering Technicians / one Graduate Engineer per team, who tested and logged the subsurface conditions encountered in accordance with NZGS (New Zealand Geotechnical Society) guidelines.

The positions of all the testing locations are shown on Drawing No. WEC930-G-001 “Geotechnical Plan” which is contained in Appendix A.

We note that testing locations Trial Pit 1, 14, 20, 40 & 41 was terminated at varying depths from 1.2m till 4.9m due to water table or water pocket encounter or hard material (SUVA MARL) encountered.

We note that for HA test 43, the tests advanced till the depth of 2.6m instead of target depth 3.0m due to hard material (SUVA MARL) encountered and was difficult to auger.

We note that all SV tests were carried at 1.0m intervals at most of the testing locations with the exception of testing locations that encountered water table.



Photograph 5: Photograph indicating Hand Auger Borehole being carried out at Test Location DCP/HA/SV 51 at EFL Seaqaqa Solar Farm Site.



Photograph 6: Photograph indicating a low point adjacent to the Nabouwalu Road where ground water was encountered.

3.2 Field Investigation Results

3.2.1 Allowable Bearing Capacity (by Dynamic Cone Penetrometer Tests)

Based on the test results from the DCP Test, tabulated below are the extrapolated Allowable Bearing Capacity values:

DCP Test No.	Reduced Level (m)	Allowable Bearing Capacity (kPa)				
		at 1.0m depth	at 2.0m depth	at 3.0m depth	at 4.0m depth	at terminating depth (5.0m)
DCP 1	93.0	109	113	155 at 2.3m	NR	NR
DCP 2	93.0	174	150	254	NR	NR
DCP 3	92.0	35	97	113	35	35
DCP 4	91.0	80	80	109	80	70
DCP 5	91.0	136	113	136	NR	NR
DCP 6	92.75	80	140	183	NR	NR
DCP 7	95.0	70	140	109	125	144
DCP 8	91.0	56	46	35	NR	NR
DCP 9	91.5	90	113	140	NR	NR
DCP 10	95.5	150	113	70	NR	NR
DCP 11	96.0	140	144	144	NR	NR



DCP 12	91.0	109	144	125	90	230
DCP 13	92.5	97	125	140	90	125
DCP 14	93.5	56	174 at 2.1m	NR	NR	NR
DCP 15	92.0	80	70	90	NR	NR
DCP 16	90.0	144	166	155	NR	NR
DCP 17	94.0	70	35	56	80	97
DCP 18	95.0	80	97	140	NR	NR
DCP 19	91.0	70	70	70	NR	NR
DCP 20	91.5	97	56	56	NR	NR
DCP 21	89.0	140	150	136	NR	NR
DCP 22	96.0	109	97	97	109 at 3.9m	NR
DCP 23	95.5	109	113	125	NR	NR
DCP 24	93.0	136	166	140	NR	NR
DCP 25	87.0	109	109	97	NR	NR
DCP 26	98.0	125	125	136	70	113
DCP 27	98.0	125	125	80	NR	NR
DCP 28	91.5	90	136	207	NR	NR
DCP 29	91.0	113	97	125	NR	NR
DCP 30	96.0	166	109	70	70	70
DCP 31	95.0	90	109	190	NR	NR
DCP 32	92.0	90	80	90	80	174 at 4.6m
DCP 33	89.0	125	155	290 at 2.4m	NR	NR
DCP 34	94.0	166	113	80	NR	NR
DCP 35	93.0	35	56	97	144	97
DCP 36	90.0	97	97	113	NR	NR
DCP 37	89.0	70	90	90	56	97
DCP 38	93.0	140	190	195	NR	NR
DCP 39	94.0	125	140	109	125	35
DCP 40	93.0	97	125	174 at 2.4m	NR	NR
DCP 41	89.0	97	70	97	NR	NR



DCP 42	88.5	136	136	144	NR	NR
DCP 43	91.5	109	285	325 at 2.5m	NR	NR
DCP 44	93.0	125	174	144	NR	NR
DCP 45	94.0	125	174	140	NR	NR
DCP 46	93.5	97	150	144	NR	NR
DCP 47	93.0	136	136	150	113	166
DCP 48	89.0	136	136	144	150	90
DCP 49	90.0	90	97	267	NR	NR
DCP 50	92.0	97	144	166	NR	NR
DCP 51	93.0	125	113	140	NR	NR
DCP 52	90.5	109	140	155	125	140
DCP 53	88.5	56	113	113	NR	NR
DCP 54	91.5	97	109	166	NR	NR
DCP 55	92.0	56	205	113	119	70
DCP 56	91.0	70	80	254	NR	NR
DCP 57	92.0	144	174	166	80	NR
DCP 58	91.0	136	136	207	NR	NR
DCP 59	91.5	109	125	209	NR	NR
DCP 60	91.0	136	166	150	NR	NR
DCP 61	88.5	113	150	242	NR	NR
DCP 62	91.0	113	144	209	155	190
DCP 63	92.5	90	113	280	NR	NR
DCP 64	91.0	140	136	166	NR	NR

Table 1: Allowable Bearing Capacity (Extrapolated from the DCP tests) at the EFL Seaqaqa Solar Farm Site.

Legend: NR - Not Recorded / Tested

The Allowable Bearing Capacity values have been extrapolated from the correlation of Scala Results with Bearing Pressure Graph by Stockwell, M. J. 1977.

3.2.2 Borehole Logs

Twenty-four (24) boreholes were drilled using an Ø50mm hand auger at the locations shown on WEC930-G-001 in Appendix A.



The borehole (soil profile) logs are presented in Appendix B and a summary of the subsurface conditions encountered in the borehole/hand auger investigations are presented in Table 2.

Hand Auger	Reduced Level (m)	Silty CLAY / CLAY	Rootlets	Quartz	Water Table/Pocket	Suva Marl (Soapstone)
HA 2	93.0	0 to 3.0m	0 to 0.8m	0 to 0.3m	NE	NE
HA 9	91.5	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
HA 11	96.0	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
HA 16	90.0	0 to 3.0m	0 to 1.2m	0 to 0.5m	NE	NE
HA 23	95.5	0 to 3.0m	0 to 0.3m	0 to 0.8m	NE	NE
HA 24	93.0	0 to 3.0m	0 to 0.2m, 1.5 to 3.0m	0 to 3.0m	NE	NE
HA 28	91.5	0 to 3.0m	0 to 0.3m	0 to 0.8m	NE	NE
HA 33	89.0	0 to 3.0m	0 to 0.3m	0 to 1.8m	NE	NE
HA 38	93.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
HA 42	88.5	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
HA 43	91.5	0 to 2.6m*	0 to 0.2m	0 to 2.6m	NE	NE
HA 44	93.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
HA 45	94.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
HA 49	90.0	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
HA 50	92.0	0 to 3.0m	0 to 2.5m	0 to 3.0m	NE	NE
HA 51	93.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
HA 54	91.5	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
HA 56	91.0	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
HA 57	92.0	0 to 3.0m	0 to 1.8m	0 to 3.0m	NE	NE
HA 58	91.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
HA 59	91.5	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
HA 60	91.0	0 to 3.0m	0 to 1.5m	0 to 3.0m	NE	NE
HA 61	88.5	0 to 3.0m	0 to 0.2m	0 to 2.4m	NE	NE
HA 63	92.5	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE

Table 2: Summary of Hand Auger Borehole Subsurface Conditions



Legend: NE - Not Encountered

- denotes premature end of borehole due to presence of an obstacle or water table.

* - denotes premature end of borehole but due to presence of hard material.

The soil upper shallow layer profile generally consisted of topsoil layer - Silty CLAY – dark brown and dark brownish orange with some quartz and rootlets with high plasticity. The soil profile was moist at all depths. Granular soil was not encountered at any test locations as the site is undeveloped and unimproved.

The soil in the deeper layer profile generally consisted of Silty CLAY – dark brown intermixed reddish brown and light reddish orange with some quartz with high plasticity. The soil profile was moist throughout as previously mentioned.

No water table was encountered in the Hand Auger tests.



TOPSOIL – Silty CLAY with some rootlets and quartz, dark brownish orange intermixed reddish brown, stiff, moist, highly plastic.

Silty CLAY, light brownish orange, stiff, moist, highly plastic.

Photograph 7: Photograph indicating generally dark brownish orange Silty CLAY (Topsoil) (shallow depth), Silty CLAY with some rootlets and quartz which was extracted from Hand Auger (HA 16).



TOPSOIL – Silty CLAY with some rootlets and quartz, dark brown stiff, moist, highly plastic.

Silty CLAY with traces of quartz, dark brownish orange, stiff, moist, highly plastic.

Photograph 8: Photograph indicating the dark brown topsoil to dark brownish orange soil profile with some rootlets and quartz which was extracted from Hand Auger (HA 60). The Silty CLAY was dark brownish orange and had trace of quartz at deeper depths.

3.2.3 Trial Pit Logs

Forty (40) trial pits were excavated by a 14-tonne Excavator at the locations shown on WEC930-G-001 in Appendix A.

The trial pit (soil profile) logs are presented in Appendix B and a summary of the subsurface conditions encountered in the trial pit investigations are presented in Table 3.

Trial Pit	Reduced Level (m)	Silty CLAY / Clay	Rootlets	Quartz	Water Table/Pocket	Suva Marl Soapstone
TP 1	93.0	0 to 2.3m*	0 to 2.0m	0 to 2.3m	NE	2.3m
TP 3	92.0	0 to 5.0m	0 to 5.0m	0 to 5.0m	1.2m	NE
TP 4	91.0	0 to 5.0m	0 to 0.2m	0 to 5.0m	3.0m	NE
TP 5	91.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
TP 6	92.75	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
TP 7	95.0	0 to 5.0m	0 to 1.5m	0 to 1.5m	NE	NE
TP 8	91.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE



Trial Pit	Reduced Level (m)	Silty CLAY / Clay	Rootlets	Quartz	Water Table/Pocket	Suva Marl Soapstone
TP 10	95.5	0 to 2.9m*	0 to 0.2m	0 to 2.9m	NE	2.9 to 3.0m
TP 12	91.0	0 to 5.0m	0 to 0.3m	0 to 1.5m	NE	NE
TP 13	92.5	0 to 5.0m	0 to 5.0m	0 to 5.0m	NE	NE
TP 14	93.5	0 to 2.1m*	0 to 1.6m	0 to 1.6m	NE	2.1m
TP 15	92.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	3.0m	NE
TP 17	94.0	0 to 5.0m	0 to 0.3m	0 to 3.0m	NE	NE
TP 18	95.0	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
TP 19	91.0	0 to 3.0m	0 to 0.3m	0 to 1.5m	NE	NE
TP 20	91.5	0 to 4.2m*	0 to 1.0m	0 to 4.2m	NE	4.2m
TP 21	89.0	0 to 3.0m	0 to 0.3m	0 to 1.2m	NE	NE
TP 22	96.0	0 to 5.0m	0 to 0.3m	0 to 5.0m	NE	NE
TP 25	87.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
TP 26	98.0	0 to 5.0m	0 to 0.3m	0 to 3.5m	NE	NE
TP 27	98.0	0 to 3.0m	0 to 0.4m	0 to 3.0m	NE	NE
TP 29	91.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
TP 30	96.0	0 to 5.0m	0 to 0.2m	0 to 4.8m	NE	NE
TP 31	95.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
TP 32	92.0	0 to 5.0m	0 to 0.25m	0 to 5.0m	NE	NE
TP 34	94.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
TP 35	93.0	0 to 5.0m	0 to 0.3m	0 to 4.0m	4.0m	NE
TP 36	90.0	0 to 3.0m	0 to 0.2m	0 to 3.0m	NE	NE
TP 37	89.0	0 to 4.9m	0 to 0.2m	0 to 4.9m	3.0m	4.9 to 5.0m
TP 39	94.0	0 to 5.0m	0 to 0.3m	0.3 to 5.0m	NE	NE
TP 40	93.0	0 to 2.4m	0 to 0.2m	0 to 2.4m	NE	2.4 to 2.7m*
TP 41	89.0	0 to 3.1m	0 to 0.2m	0 to 3.1m	3.0m	3.1 to 3.3m*
TP 46	93.5	0 to 3.0m	0 to 0.3m	0 to 3.0m	NE	NE
TP 47	93.0	0 to 5.0m	0 to 0.2m	0 to 5.0m	NE	NE



Trial Pit	Reduced Level (m)	Silty CLAY / Clay	Rootlets	Quartz	Water Table/Pocket	Suva Marl Soapstone
TP 48	89.0	0 to 5.0m	0 to 0.2m	0 to 5.0m	4.0m	NE
TP 52	90.5	0 to 5.0m	0 to 0.3m	0 to 5.0m	3.0m	NE
TP 53	88.5	0 to 3.0m	0 to 0.2m	0 to 3.0m	3.0m	NE
TP 55	92.0	0 to 5.0m	0 to 0.2m	0 to 5.0m	4.0m	NE
TP 62	91.0	0 to 5.0m	0 to 0.6m	0 to 5.0m	5.0m	NE
HA 64	91.0	0 to 3.0m	0 to 1.3m	0 to 3.0m	NE	NE

Table 3: Summary of Test Pit Subsurface Conditions

Legend: NE - Not Encountered

- denotes premature end of borehole due to presence of an obstacle or water table.

* - denotes premature end of borehole but due to presence of hard material.

The soil upper shallow layer profile generally consisted of topsoil layer - Silty CLAY – dark brown and dark brownish orange with some quartz and rootlets with high plasticity. The soil profile was moist at all depths and saturated at the test locations where water pocket/table was encountered. Granular soil was not encountered at any test locations as the site is undeveloped and unimproved.

The soil in the deeper layer profile generally consisted of topsoil layer - Silty CLAY – dark brown intermixed reddish brown and light reddish orange with some quartz with high plasticity. The soil profile was moist throughout as previously mentioned. SUVA MARL (Soapstone) was encountered in certain test pits at deeper depths (1.6m to 5m from respective ground level).

Ground Water was encountered at low points adjacent to the watercourses during Trial Pit excavation.



Photograph 9: Photograph indicating generally dark brown Silty CLAY, topsoil (shallow depth), Silty CLAY material which was extracted with an excavator (TP 41) which contained some quartz and minor rootlets with water table encountered.



Photograph 10: Photograph indicating the dark brown Silty CLAY (topsoil) with rootlets and some quartz (shallow depth) and dark brownish orange Silty CLAY material which was extracted with excavator (TP 62) at deeper depth.



3.2.4 Shear Strength from Shear Vane (SV) Test

The Shear Vane test results are summarised and are presented in Table 4 below.

Shear Vane Test Locations	Reduced Level (m)	Peak Shear Strength/Remoulded Shear Strength (kPa) at depth of:				
		1m	2m	3m	4m	5m
TP 1	93.0	157/45	182/42	NR	NR	NR
HA 2	93.0	196+	196+	196+	NR	NR
TP 3	92.0	168/42	196+	196+	NR	NR
TP 4	91.0	196+	171/56	90/39	129/20	42/14
TP 5	91.0	196+	196+	196+	NR	NR
TP 6	92.75	196+	196+	196+	NR	NR
TP 7	95.0	168/42	196+	196+	196+	196+
TP 8	91.0	196+	98/50	62/20 @ 2.5m	NR	NR
HA 9	91.5	136/109	167/121	182/103	NR	NR
TP 10	95.5	196+	196+	196+ @ 2.9m	NR	NR
HA 11	96.0	212+	212+	212+	NR	NR
TP 12	91.0	196+	196+	196+	196+	196+
TP 13	92.5	196+	196+	196+	196+	196+
TP 14	93.5	102/32	110/30	NR	NR	NR
TP 15	92.0	118/34	120/55	196+	NR	NR
HA 16	90.0	173/73	143/61	182/94	NR	NR
TP 17	94.0	146/56	140/17	64/17	87/34	123/25
TP 18	95.0	151/64	196+	196+	NR	NR
TP 19	91.0	196+	196+	196+	NR	NR
TP 20	91.5	120/98	108/42	96/24	52/22	NR
TP 21	89.0	196+	196+	196+	NR	NR
TP 22	96.0	164/67	191/61	82/45	91/36	70/18



Shear Vane Test Locations	Reduced Level (m)	Peak Shear Strength/Remoulded Shear Strength (kPa) at depth of:				
		1m	2m	3m	4m	5m
HA 23	95.5	182/91	212+	212+	NR	NR
HA 24	93.0	196+	160/59	196+	NR	NR
TP 25	87.0	146/56	196+	171/87	NR	NR
TP 26	98.0	196+	196+	196+	196+	112/50
TP 27	98.0	196+	157/28	196+	NR	NR
HA 28	91.5	173/73	143/61	182/94	NR	NR
TP 29	91.0	196+	196+	196+	NR	NR
TP 30	96.0	196+	196+	196+	151/59	121/48
TP 31	95.0	200/106	194/100	191/94	NR	NR
TP 32	92.0	197/75	118/61	94/42	106/30	136/18
HA 33	89.0	196+	196+	196+	NR	NR
TP 34	94.0	196+	171/56	196+	NR	NR
TP 35	93.0	42/12	40/28	110/50	92/42	236/20
TP 36	90.0	196+	135/22	129/28	NR	NR
TP 37	89.0	129/70	196+	196+	196+	196+
HA 38	93.0	196+	196+	196+	NR	NR
TP 39	94.0	196+	196+	196+	143/66	76/42
TP 40	93.0	196+	118/14	NR	NR	NR
TP 41	89.0	196+	196+	196+	NR	NR
HA 42	88.5	196+	196+	196+	NR	NR
HA 43	91.5	212+	212+	212+	NR	NR
HA 44	93.0	196+	196+	168/56	NR	NR
HA 45	94.0	196+	196+	196+	NR	NR
TP 46	93.5	196+	196+	196+	NR	NR
TP 47	93.0	196+	196+	196+	196+	196+
TP 48	89.0	196+	196+	196+	146/87	154/34



Shear Vane Test Locations	Reduced Level (m)	Peak Shear Strength/Remoulded Shear Strength (kPa) at depth of:				
		1m	2m	3m	4m	5m
HA 49	90.0	212+	212+	212+	NR	NR
HA 50	92.0	196+	182/118	174/79	NR	NR
HA 51	93.0	196+	196+	135/60	NR	NR
TP 52	90.5	196+	196+	168/81	140/112	118/28
TP 53	88.5	154/79	196+	149/81	NR	NR
HA 54	91.5	212+	212+	212+	NR	NR
TP 55	92.0	168/81	146/79	98/42	56/22	115/44
HA 56	91.0	170/79	212+	212+	NR	NR
HA 57	92.0	196+	196+	196+	NR	NR
HA 58	91.0	196+	196+	196+	NR	NR
HA 59	91.5	212+	212+	212+	NR	NR
HA 60	91.0	196+	196+	196+	NR	NR
HA 61	88.5	179/107	196+	196+	NR	NR
TP 62	91.0	196+	154/79	143/70	84/36	196+
HA 63	92.5	212+	182/91	203/72	NR	NR
HA 64	91.0	196+	174/76	196+	NR	NR

Table 4: Summary of the Shear Vane Test Results.

Legend: NR - Not Recorded / Tested

Please note that maximum shear value 196+ and 212+ are from two different Shear Vane testing devices with different calibration certificates.

3.2.5 Allowable Bearing Capacity from Shear Vane (SV) Test

Based on the test results from the Shear Vane (SV) Test, tabulated below are the calculated Allowable Bearing Capacity values:



Shear Vane Test Locations	Reduced Level (m)	Allowable Bearing Capacity Strength (kPa) at depth of:				
		1m	2m	3m	4m	5m
TP 1	93.0	314	364	NR	NR	NR
HA 2	93.0	392	392	392	NR	NR
TP 3	92.0	336	392	392	NR	NR
TP 4	91.0	392	342	180	258	84
TP 5	91.0	392	392	392	NR	NR
TP 6	92.75	392	392	392	NR	NR
TP 7	95.0	336	392	392	392	392
TP 8	91.0	392	196	124 @ 2.5m	NR	NR
HA 9	91.5	272	334	364	NR	NR
TP 10	95.5	392	392	392 @ 2.9m	NR	NR
HA 11	96.0	424	424	424	NR	NR
TP 12	91.0	392	392	392	392	392
TP 13	92.5	392	392	392	392	392
TP 14	93.5	204	220	NR	NR	NR
TP 15	92.0	236	240	392	NR	NR
HA 16	90.0	346	286	364	NR	NR
TP 17	94.0	292	280	128	174	246
TP 18	95.0	302	392	392	NR	NR
TP 19	91.0	392	392	392	NR	NR
TP 20	91.5	240	216	192	104	NR
TP 21	89.0	392	392	392	NR	NR
TP 22	96.0	328	382	164	182	140
HA 23	95.5	364	424	424	NR	NR
HA 24	93.0	392	320	392	NR	NR
TP 25	87.0	292	392	342	NR	NR



Shear Vane Test Locations	Reduced Level (m)	Allowable Bearing Capacity Strength (kPa) at depth of:				
		1m	2m	3m	4m	5m
TP 26	98.0	392	392	392	392	224
TP 27	98.0	392	314	392	NR	NR
HA 28	91.5	346	286	364	NR	NR
TP 29	91.0	392	392	392	NR	NR
TP 30	96.0	392	392	392	302	242
TP 31	95.0	400	388	382	NR	NR
TP 32	92.0	394	236	188	212	272
HA 33	89.0	392	392	392	NR	NR
TP 34	94.0	392	342	392	NR	NR
TP 35	93.0	84	80	220	184	472
TP 36	90.0	392	270	258	NR	NR
TP 37	89.0	258	392	392	392	392
HA 38	93.0	392	392	392	NR	NR
TP 39	94.0	392	392	392	286	152
TP 40	93.0	392	236	NR	NR	NR
TP 41	89.0	392	392	392	NR	NR
HA 42	88.5	392	392	392	NR	NR
HA 43	91.5	424	424	424	NR	NR
HA 44	93.0	392	392	336	NR	NR
HA 45	94.0	392	392	392	NR	NR
TP 46	93.5	392	392	392	NR	NR
TP 47	93.0	392	392	392	392	392
TP 48	89.0	392	392	392	292	308
HA 49	90.0	424	424	424	NR	NR
HA 50	92.0	392	364	348	NR	NR
HA 51	93.0	392	392	270	NR	NR



Shear Vane Test Locations	Reduced Level (m)	Allowable Bearing Capacity Strength (kPa) at depth of:				
		1m	2m	3m	4m	5m
TP 52	90.5	392	392	336	280	236
TP 53	88.5	308	392	298	NR	NR
HA 54	91.5	424	424	424	NR	NR
TP 55	92.0	336	292	196	112	230
HA 56	91.0	340	424	424	NR	NR
HA 57	92.0	392	392	392	NR	NR
HA 58	91.0	392	392	392	NR	NR
HA 59	91.5	424	424	424	NR	NR
HA 60	91.0	392	392	392	NR	NR
HA 61	88.5	358	392	392	NR	NR
TP 62	91.0	392	308	286	168	392
HA 63	92.5	424	364	406	NR	NR
HA 64	91.0	392	348	392	NR	NR

Table 5: Allowable Bearing Capacity (from SV tests).

Legend: NR - Not Recorded/Tested

3.2.6 Materials Encountered During the Investigation

3.2.6.1 Topsoil

The depth of topsoil across the site generally varies between 100mm to 600mm. Based upon the borehole logs the average depth of topsoil was 610mm over the site.

The depth of topsoil varies from section to section of the site, the recommended depth of stripping topsoil shall be 200 to 610mm, however, the Contractor should be advised to visually inspect areas as they strip the topsoil to ensure that in areas where topsoil is less than 300mm thick, is stripped to only remove the entire topsoil. Identification of topsoil by the Contractors should not become a laborious task.



3.2.6.2 Unsuitable Materials

The highly reactive materials are not suitable for the construction of structural fill or as founding materials for buildings / structures. Whilst these materials may not exhibit undesirable characteristics in their current location, they will cause construction difficulties (loss of shear strength in surrounding soils caused by excessive moisture) and long-term adverse effects to buildings and structures (seasonal surface movement) when exposed to adverse or variable moisture regimes. As these unsuitable materials possess considerably higher optimum moisture contents to other materials at the site, mixing or compaction of these materials will require additional water, which will reduce the shear strength of the surrounding materials. This may be overcome by lime stabilization; however, the lime mix ratio will need to be determined by laboratory testing prior to works commencing. Topsoil identified at the site is also not suitable for structural filling and must be removed prior to placement of any fill or removal of any “borrow” materials. These materials contain organic matter (which will decompose over time) and are likely to be more “silty” (which will cause significant loss of shear strength with small changes in moisture content).

With highly reactive / plastic soils being encountered, we advise that the Contractor should be aware of areas adjacent to the open earth drain / stormwater or adjacent to existing open earth drains / Korovuli Creek or at the low areas of the sites. The depth of topsoil across the site generally varies between 100mm and 500mm. Based upon the borehole logs, the average depth of topsoil was 610mm which could be used as a guide to determining the volume of topsoil over the site. The topsoil materials can be placed over the structural filling to a depth not exceeding 250mm provided the founding depth of the building foundations is greater than 600mm inclusive of the under slabs.

The following earthworks bulking factors (Cut to loose stockpile) for the highly plastic materials and topsoil could be used as a guide to assist in construction:

Topsoil: 1.25 – 1.4

Plastic CLAYS: 1.4

The highly reactive / plastic clays could be reused to line any permanently saturated water feature (wetland, lake, etc) constructed at the site. Further recommendations regarding its application in this context will be required to be obtained.



4 Laboratory Test Results

The following laboratory tests were undertaken on samples retrieved during the field investigation:

Tests Carried Out:	Laboratory	Contained in
a) Two suites of Particle Size Distribution Test.	Geotech Testing Fiji in Suva, Fiji	Appendix C
b) Two suites of Atterberg Limits Test.	Geotech Testing Fiji in Suva, Fiji	Appendix C
c) Two suites of Unconfirmed Compressive Strength (UCS).	Trilab in Brisbane, Australia	Appendix D
d) Two suites of Triaxial Test (for cohesionless soils).	Trilab in Brisbane, Australia	Appendix D
e) Two suites of Consolidation Test (Oedometer Test).	Trilab in Brisbane, Australia	Appendix D
f) Two suites of Shrink-Swell Index Tests.	Trilab in Brisbane, Australia	Appendix D
g) Two suites of pH, Sulphate, Chloride content Tests.	University of South Pacific (Applied Science) in Suva, Fiji	Appendix E

4.1 Soil Classification

4.1.1 Particle Size Distribution

The Particle Size Distribution test measures the proportion of soils in each size range. Soil samples can be separated into different fractions or intervals, where each fraction represents a range of particle size.

From the Particle Size Distribution Charts / Graphs contained in Appendix C, the Soils tested are classified as Silty CLAY.

4.1.2 Atterberg Limits Test

The Atterberg Limits tests were undertaken in accordance with the methods outlined in AS1289 3.1.2 & 3.2.1 & 3.3.1. The Atterberg limits are a basic measure of the nature of a fine-grained



soil. Depending on the water content of the soil, it may appear in four states: solid, semi-solid, plastic and liquid. In each state the consistency and behaviour of a soil is different and thus so are its engineering properties. Thus, the boundary between each state can be defined based on a change in the soil's behaviour. The Atterberg limits can be used to distinguish between silt and clay, and it can distinguish between different types of silts and clays. These tests are performed on the % passing the 425 micron sieve.

Results of the laboratory testing are summarised in the table below and presented in full in Appendix E.

Table 6: Summary of the Atterberg Limits Test Results.

Sample Details			Soil Classification Tests			
Testing Location	Sample Depth (m)	Field Description	LL (%)	PL (%)	PI (%)	LS (%)
TP 4	1.8 - 2.0m	Silty CLAY	86	73	13	19.5
TP 34	1.5 – 1.8m	Silty CLAY	84	71	13	17
TP 41	1.0 – 1.3m	Silty CLAY	93	79	14	17

Legend:

NMC = Natural Moisture Content

LL = Liquid Limit

PL = Plastic Limit

PI = Plasticity Index

LS = Linear Shrinkage

4.2 Strength & Compressibility

The Strength and Compressibility Tests were carried out by Trilab in Brisbane, Australia as these tests are not carried out in Fiji.

Two undisturbed block samples of approximately 250mm x 250mm soil for TP 4 and TP34 which was removed from site, were couriered via airplane to Australia for testing.

4.2.1 Unconfirmed Compressive Strength (UCS)

The Unconfined Compressive Strength (UCS) of soil is the maximum axial stress a cohesive soil sample can withstand under zero confining (lateral) pressure before failure. It is a fast, cost-effective way to estimate undrained shear strength, typically used for saturated clays and silts



From the UCS test results which are contained in Appendix D, we note the following Foundation Design Implications:

a) TP 4 (Stiff Soil, UCS / q_u = 149.1 kPa)

- **Foundation Type:** Excellent candidate for driven steel piles (ground screws or H-piles), which are standard for solar farms.
- **Driven Depth:** The stiff soil provides good lateral resistance and skin friction. Piles will likely require standard embedment depths (typically 1.5 to 2.5 meters, depending on wind load).
- **Refusal Risk:** Low risk of pile refusal during driving, though any gravel or cobble content should be checked.

b) TP 34 (Medium/Firm Soil, UCS / q_u = 83.7 kPa)

- **Foundation Type:** Driven piles are still viable, but they will require modifications due to the weaker soil consistency.
- **Driven Depth:** Piles will need to be driven **deeper** to achieve the required skin friction to combat wind uplift.

4.2.2 Triaxial Shear Test (for cohesionless soils)

The Triaxial Shear Test is the industry-standard laboratory method used to determine the shear strength and deformation characteristics of soil. By encasing a cylindrical soil sample in a pressurized cell, engineers simulate the exact stress conditions the soil experiences deep underground to predict how it will behave beneath foundations, slopes, or embankments.

The Triaxial Shear Test results for TP 4 and TP 34 are contained in Appendix D.

4.2.3 Consolidation Test (if settlement is a concern)

A **Soil Consolidation Test** (commonly the **Oedometer Test**) determines the magnitude and rate at which soil compresses and settles under the structural load of solar mounting systems. This ensures pile foundations or ballasts don't sink over time, preventing structural damage to the solar trackers and arrays.

The Oedometer Test results for TP 4 and TP 34 are contained in Appendix D.



4.3 Expansive Soil

4.3.1 Shrink-Swell Index (AS 1289.7.1.1)

The Shrink-Swell Index (I_{ss}) measures a soil's potential to expand (swell) when wet and contract (shrink) when dry. Expressed as the rate of volume change per unit change in soil suction, it is a critical metric for solar farms to prevent structural damage to pile foundations, racking systems, and equipment pads.

The Shrink-Swell Index Test results for TP 4 and TP 34 are contained in Appendix D.

For TP 4, an $I_{ss} = 9$ represents an extraordinarily severe and extreme soil reactivity level that is rarely seen in standard geotechnical testing.

To put this in perspective, an index of 9 is roughly three times higher than what Engineers typically classify as a "highly reactive" clay. For the ground at TP 4, will experience massive, destructive movement during seasonal wetting and drying cycles.

TP 34, Testing **remoulded material** completely changes how $I_{ss} = 3.1$ is interpreted. The sample was remoulded to received density and moisture content. Under the strict rules of the National Association of Testing Authorities (NATA), the standard AS 1289.7.1.1 is exclusively intended for **undisturbed soil samples**. Therefore, TP34 results cannot be considered further.

4.4 Soil Corrosivity Testing

4.4.1 pH, resistivity, sulphate/chloride content

Soil corrosivity testing at a solar farm determines the degradation rate of buried steel piles and grounding components. Testing involves measuring electrical resistivity (the primary indicator), pH, redox potential, moisture content, and soluble ions (chlorides and sulphates) to design safe, long-lasting foundations.

The pH, Chloride and Sulphate test results are contained in Appendix E and a summary of the laboratory test results is contained in Table 7, below:

Test Locations	pH	Chloride (mg/L)	Sulphate (mg/Kg)
TP 4	4.5 at 25.7°C	1,896	19.8
TP 34	4.5 at 25.9°C	1,773	27.0
TP 41	4.2 at 25.9°C	2,385	18.3

Table 7: Soil Corrosivity Tests Results.



Laboratory testing undertaken on representative soil samples indicates strongly acidic soil conditions, with measured pH values ranging from 4.2 to 4.5. Chloride concentrations are elevated, ranging from approximately 1,770 mg/L to 2,385 mg/L, while sulphate concentrations are low and are not considered aggressive toward buried concrete structures.

Groundwater was encountered in Test Pits TP4 and TP41 at a depth of approximately 3.0 m below ground level. The presence of groundwater is considered a significant factor in the assessment of corrosion potential, as saturated soil conditions increase electrical conductivity and facilitate electrochemical corrosion processes. The combination of acidic soil conditions, elevated chloride concentrations, and groundwater presence creates an environment conducive to accelerated corrosion of buried steel and galvanized steel components.

Based on the available laboratory test results and groundwater observations, the soils across the site are assessed as having a high to very high corrosion potential toward buried metallic infrastructure. The highest corrosion risk is anticipated in areas where steel elements extend into or remain in contact with saturated soils below the groundwater table.

It is recommended that corrosion protection measures be incorporated into the design of steel piles, earthing systems, and other buried metallic infrastructure. Such measures may include increased corrosion allowances, enhanced galvanizing systems, protective coatings, or other project-specific corrosion mitigation strategies consistent with the required design life of the solar farm.

4.5 Seismic Considerations (NZS 1170.5)

Evaluating a solar farm under NZS 1170.5 requires critical attention to subsoil classification, liquefaction potential, and soil-structure interaction. For piled or ground-mounted PV structures, the primary seismic risk is rarely structural collapse, but rather foundation lateral yielding, soil settlement, and rack deformation.

As documented in Section 4.2.1 from the UCS Tests:

- TP 4 (Stiff Cohesive Soil), UCS / q_u = 149.1 kPa)
- TP 34 (Medium/Firm Cohesive Soil), UCS / q_u = 83.7 kPa)

This stronger profile further secures the site's **Class C (Shallow Soil Site)** classification under NZS 1170.5:2004.

Class C (Shallow Soil): NZS 1170.5 Clause 3.1.3 classifies sites based on the total depth of the soil layer to low-amplitude shear wave velocity bounds. Because your softer clay layer (s_u of



41.9 to 74.5 kPa) is relatively shallow (3m to 5m) and likely overlies denser material or bedrock, the site will most likely qualify as **Site Class C (Shallow Soil Sites)**.

In accordance with the NZS 1170.5, The Zone Factor Map by Stirling & Nguyen, 2014, the site has a corresponding Zone Factor between 0.3 to 0.35 as shown in Figure 4 below.

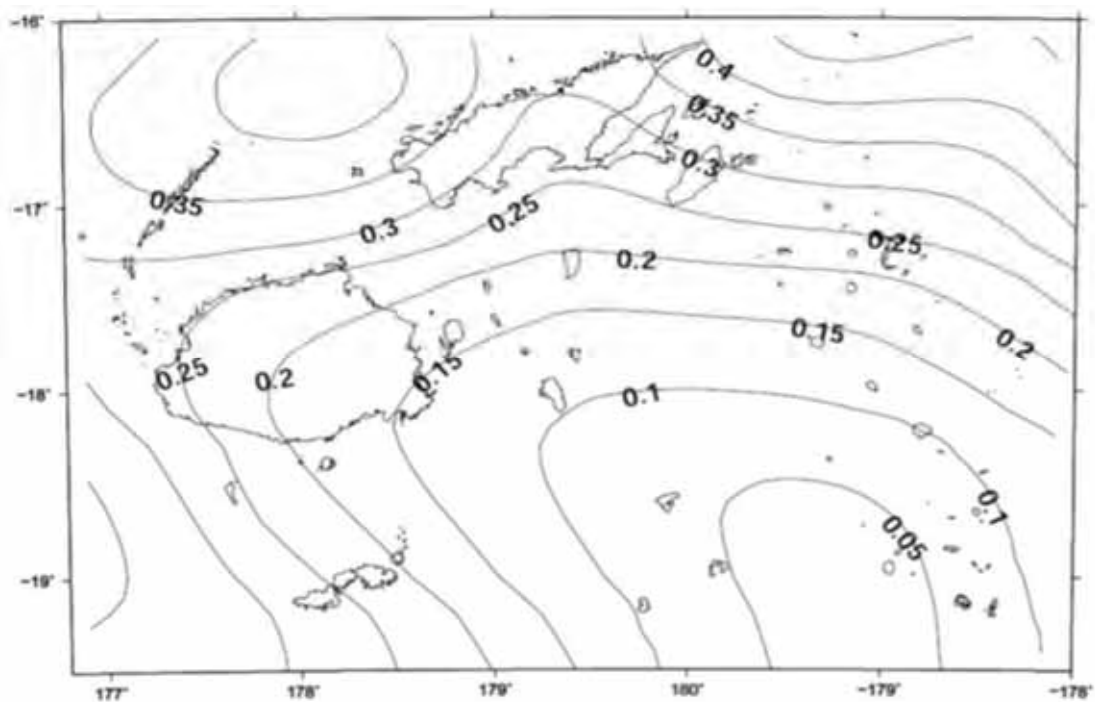


Figure 4: Seismic risk zoning map².



5 Recommendations

5.1 Grass Mat Thickness

The ground cover on the day the investigation varied across the site. Photograph 9 below illustrates the extent of each of the major ground covers at the site. Majorly, tall grass, shrubs / rank grass (weeds) growth and large trees was confined to the periphery of the site and by Korovuli Creek and certain other areas on site as shown on Drawing No. WEC930-G-002. The undisturbed thickness of this grass mat varied between 0mm to 200mm. Organic material in the upper soil horizon was limited to the top 600mm. However, rootlets / roots were evidence at deeper depths even up to 5m, due to large trees being located close to Korovuli Creek.

The depth of topsoil across the site was generally varies between 200mm and 500mm. Based upon the borehole logs, the average depth of topsoil/rootlets was 610mm over the site.



Photograph 11: Photograph indicating the type of trees, grass and shrubs / grass present within the EFL Seaqaqa Solar Farm Site.

5.2 Soil Compressibility and Settlement of Any Fill Materials

It is recommended that soft materials be avoided in any construction on this land / site. Soft soil can be defined as soils not passing a proof roll test. This can be achieved by adopting founding depths of foundations below soft materials or by removing soft materials and replacing the soft materials with engineered fill (*refer to Section 5.4-Method for foundation preparation prior to placement of fill*)



It should be noted that the recommendations of this indicate a safe bearing capacity of the firm to very stiff soils and therefore soils are unlikely to compress if this loading is not exceeded.

From the Plasticity Index Test carried out at Hand Auger Location No. 1A, the results received for the soils represented by the borehole determine the ratings for the Compressibility and Shrink-Swell potential of the soil based on Plasticity Index and Liquid Limit as per data from Table 3.4 and Table 3.5², respectively:

✓ Based on Plasticity Index:

Testing Location	Sample Depth (m)	Field Description	Plasticity Index (%)	Ratings for Compressibility and Shrink-Swell Potential based on Plasticity Index (PI)
TP 4	1.8 - 2.0m	Silty CLAY	13	Low (PI < 25)
TP 34	1.5 – 1.8m	Silty CLAY	13	Low (PI < 25)
TP 41	1.0 – 1.3m	Silty CLAY	14	Low (PI < 25)

✓ Based on Liquid Limit:

Testing Location	Sample Depth (m)	Field Description	Liquid Limit (%)	Ratings for Compressibility and Shrink-Swell Potential based on Liquid Limit (LL)
TP 4	1.8 - 2.0m	Silty CLAY	86	Very High (LL > 75)
TP 34	1.5 – 1.8m	Silty CLAY	84	Very High (LL > 75)
TP 41	1.0 – 1.3m	Silty CLAY	93	Very High (LL > 75)

The above Ratings for Compressibility and Shrink-Swell Potential based on Plasticity Index and Liquid Limit shall be taken into account in the design of the PV Panel foundations.

² (Murphy, 2007)



There was no evidence of FILL from the area around borehole, however, in the event that any other unidentified FILL is discovered during construction, the following action should be taken:

- Ascertain the extent of FILL (area, approximate depth).
- Verify that the FILL is free from contaminants which could pose a hazard to human and animal health.
- Remove the FILL materials and either export from the site or relocate or replace.

The Geotechnical Consultant (WesEng Consulting Pte Limited) should be called to provide specific recommendations for the treatment of any FILL discovered at the site.

The issues relating to the extent of soft material (unsuitable material) may become evident during construction is required to be removed and replaced with suitable FILL material prior to the construction of any structures. However, the locations at which tests have been carried out, the reusability and treatment of the soft material is elaborated in Section 3 and 4 of this Report. Note that the treatment of the soft material is for subsoils only.

5.3 Ground Water, Drainage Design Measures and Stabilisation Methods

Observations during the undertaking of engineering boreholes does not reveal evidence of high groundwater but a high groundwater would be expected close to the low-lying areas as tests were carried out close to open earth drain / stormwater channel (at the centre of the site) or adjacent to Korovuli Creek. Based on experience with similar soil profiles to that encountered during this investigation, it is envisaged at a seasonal or perched water table may occur after heavy rainfall or during wetter times of the year, in soils adjacent to permanently running watercourses, adjacent to the stormwater channel or at the low elevation area (adjacent to the Korovuli Creek). Silty materials (including some of the Silty CLAY observed at this site) particularly during construction are susceptible to significant loss of strength with changes in moisture content, however we have often observed that good construction conditions and management and subsequent good pavement performance can be attained where attention to providing good drainage has been provided. Improvements to subsurface drainage could be gained through the provision of appropriately installed subsurface drains (subsoil drains), which can intercept water and divert water to appropriate outlets. Outlets should be above the 10% AEP flood level. Additionally, the construction of drainage channels (along roads at in the rear of allotments or at the nominated locations) immediately after bulk earthworks will assist in “de-watering” however, care must be taken to ensure that the outlet of these drains is above the 10% AEP flood level.

In areas where subsurface drainage cannot be achieved, consideration of insitu stabilisation or subgrade replacement may be required. Insitu stabilisation may be undertaken subject to



verifying the type and quantity of any additive (lime, cement, bitumen, etc) by laboratory testing. Lime (and cement) stabilization is commonly effective in non-ferrous silty CLAYS. Subgrade replacement with sound crushed rock could be adopted in isolated areas where the existing soft material is completely removed to a firm base (insitu CBR of 6% or approximately 500 to 700mm over softer materials subject to inspection by a suitably qualified and experienced geotechnical consultant). The use of a geotextile would be beneficial to prevent the contamination of the engineered backfill from the underlying unsuitable/softer subgrade material. Geogrids may also be utilised with geotextile to provide a firmer pavement layer to stabilize the entire pavement layer.

Should further or more detailed information regarding groundwater at this site wish to be obtained a specific groundwater investigation including surveying, the installation of piezometers and periodic monitoring should be carried out.

5.4 Method for foundation preparation prior to placement of fill.

Prior to construction of any building foundations or placement of any fill material, all earthwork areas shall be stripped down to natural ground or engineered fill thoroughly clearing the site of all rubbish, debris and vegetation. The results of the site investigation indicate that the existing topsoil layer is generally 100mm to 500mm in thickness.

The cleared subgrade should be inspected by a Civil Engineer experienced in geomechanics and familiar with the contents of this report prior to commencement of works. Where uncontrolled fill materials are encountered this material shall be removed and replaced with general engineered fill.

General filling materials should comprise only clean inorganic soils and rock. Such materials should be placed and compacted in layers no more than 150mm in loose thickness with a vibratory roller of appropriate weight and energy to achieve the minimum dry density ratio specified in Table 5.1 of AS3798:1996 (*refer to Appendix G*) with a suitable water content.

The general filling materials should also have a low air void ratio (i.e. <3%) so that the completed fill is of a low permeability thereby reducing the ability of water to enter and cause swelling in the soil.

Where backfill is required to form a final subgrade to the floor slab it is recommended that clean, free-draining compacted hardfill be utilised. Again, compaction should be carried out utilising an appropriate vibratory compactor to achieve the minimum dry density ratio specified in Table 5.1 of AS3798:1996 with suitable water content. The use of a geotextile would be beneficial to prevent the contamination of the engineered backfill from the underlying unsuitable subgrade material.



Compaction tests should be carried out on the fill layers after the compactive effort has been applied. The fill layers should be compacted to greater than 100% of Maximum Dry Density (Standard Compaction) within 150mm of subgrade level in cuts and within 200mm of subgrade level in fills and structural filling.

It is recommended that in areas where unsuitable/soft/heaving material which is not excavated to prevent further aggravation of the situation, application of a geotextile fabric would be advantageous to prevent unsuitable material protruding into the fill layers and contaminating the fill layers and thus weakening the layers.

Regular observations, inspection and testing should be carried out by a Civil Engineer experienced in geomechanics and familiar with the contents of this report to ensure that the fill placement has been carried out to appropriate standards and for certification purposes. A suitable earthworks specification for construction should be prepared in accordance with applicable standards (eg. AS3798:1996, NZS4402:1986, NZS4431:1989 and NZS4404:2004) to assist the Contractor and ensure the standard of works.

5.5 Batter Slope

The maximum batter slopes that may be constructed at this site will be primarily dictated by the ultimate ability to stability. As a guide, the following criteria are provided:

- | | |
|---|-------|
| ▪ Drivable landscaped cut or fill slope: | 1:5 |
| ▪ Slope abutting water accessible by public: | 1:8** |
| ▪ Slope abutting water not accessible by public | 1:3** |
| ▪ Maximum cut batter – Silty CLAY*: | 1:1 |
| ▪ Maximum fill batter – Silty CLAY*: | 1:2 |

* Applies to silty CLAY material ONLY. Adequate drainage at the top of batter which diverts surface water and ground water away from the batter slope and does not promote pooling of water must be provided. Slope must be protected from erosion by fixing a geofabric to the batter with 300mm pins in the short term until selected suitable vegetation is established on the embankment (i.e.: vetiver grass). The embankment may require shaping to accommodate vegetation. Fill batters must be constructed by overfilling the relevant areas and trimming back to the desired fill batter slope. Care should also be taken to avoid the construction of services and drains at the toe of the batter. Berms in high cuttings / fill batters should be considered for maintenance purposes.

**Batter slopes adjacent to watercourses should be provided with protection against scouring and wave action.



At the time of the investigation, it is highly unlikely that large (> 2m) batters would be constructed. It is on this basis that the above general guidelines have been provided. If it is anticipated that large batters are anticipated to be constructed, then a detailed slope stability analysis should be commissioned by the Ministry to provide the necessary design and construction parameters to achieve an optimal design.

5.6 Suitability of On-Site Materials for Use as Structural Fill and Recommend Any Remedial Methods Required To Improve Their Suitability for Filling.

The following site derived materials have been deemed suitable for use as structural fill throughout the site:

- Sand or Gravel (predominantly located in the existing road sections of the site).
- Some of the silty CLAY material can be considered dependent on the moisture content and the CBR value.
- Any other material deemed suitable by a suitably qualified and experienced geotechnical consultant during construction.

These materials must be placed in accordance with the filling and subgrade preparation recommendations of this report.

Note: Any reddish brown or blackish brown Silty CLAY if encountered predominantly located directly adjacent to the existing watercourse or at the low area (adjacent to the Creek) which has water ponding or if observed to be below a perched water table, shall not be utilised as fill material. If excavation in this area is required to provide borrow material for structural filling, it will be essential to dewater this area in the first instance. Further, drying back of the materials by either one or a combination of the following methods may be required:

- Ripping or ploughing or scarifying the clays.
- Allowing the exposed clays to dry in the atmosphere.
- Mixing the drier clays with the wetter clays and allowing curing (24 hours).
- Thoroughly mixing lime with the clays and allowing curing (24 hours).

This recommendation may apply to any suitable site derived Silty CLAY materials utilised for structural filling where the material is wetter than optimum moisture content.



General filling materials should comprise of only clean inorganic soils and rock. Such materials should be placed and compacted in layers no more than 150mm in loose thickness with a vibratory roller of appropriate weight and energy (typically a sheep's foot type roller).

A suitable earthworks specification for the project should be prepared in accordance with applicable standards (eg. AS 3798:1996, NZS4402:1986, and NZS4404:2004) to assist the Contractor and ensure the standard of works. The specification should specify a low air void ratio (i.e. <3%) so that the completed fill material is of a low permeability and reduces the ability of water to enter and cause swelling in the soil. Fill compaction should be carried out utilizing an appropriate vibratory compactor to achieve the minimum dry density ratio specified in Table 5.1 of AS 3798:1996 with a suitable water content.

No recommendations have been made for any materials imported to this site.

5.7 Basic Compaction Control Parameters for the Design of Fill Areas, Including the Precondition of Materials Prior To Filling.

All fill shall be moisture conditioned and compacted in accordance with the following requirements:

5.7.1 Clearing and Grubbing

Clearing and grubbing is the removal of the following from the site:

- Vegetation such as trees, tree stumps, tree roots, logs, brush, noxious weeds and decayed vegetable matter.
- Refuse such as pole stumps, rubbish dumps and debris piles resting on or protruding from the ground surface.
- Obstructions such as unutilised structures - concrete paving, concrete edgings, drainage pits, foundations, fences, etc, but not underground obstructions such as drainage pipes, service conduits and fuel tanks.

In areas where excavation will be carried out, all vegetation, refuse and obstructions shall be grubbed to a depth of not less than 0.3m below the stripped surface or not less than 0.6m below the finished surface of the subgrade, whichever is the lesser treatment.

5.7.2 Compact and Moisture Condition - Stripped Surface

The stripped surface upon which any fill material shall be placed shall be compacted and moisture conditioned to the following specification:



- **Density**- a relative density of not less than 95.0% and 100% (roadways and for upper 1m layer from finished subgrade level) of the density determined in AS1289.5.1.1 (standard compactive effort) shall be obtained.
- **Moisture**- a relative moisture ratio not less than 90% than the optimum moisture content (OMC) determined in AS 1289.5.1.1 (standard compactive effort) shall be obtained. Note that it may be difficult to compact materials with a moisture content greater than 105% of optimum moisture content.
- **Consistency of subgrade conditions** - to ensure that consistent subgrade conditions have been achieved in the field, the following testing shall be undertaken:

5.7.2.1 Proof Roll Test:

The proof roll test remains the most practical acceptance test for subgrade stability and shall be applied to all pavement subgrades. The acceptance criteria shall be when an area withstands test rolling without rutting and with no visible deformation or springing. Areas failing proof-roll tests require re-treatment until accepted by the Superintendent (Civil Works Supervisor/Engineer). Irrespective of design CBR value and expansion characteristics of the subgrade material, if local soft areas/spots of subgrade are encountered at the time of construction, then these areas shall be treated or replaced in order to achieve a stable subgrade platform for acceptance by proof rolling.

5.7.2.2 Density and Moisture Conditions

Unless otherwise specified, acceptance of material and work will be based on testing of the material or work in lots. A lot will consist of a single layer, batch or area of like work which has been constructed or produced under essentially uniform conditions and is essentially homogeneous with respect to material and appearance. Unless otherwise specified, the extent of each lot shall not exceed one day's production or 2500m², whatever the lesser.

Six (6) density tests shall be performed at the locations determined in AS 1289.1.4.2 or other approved methods. Discrete portions of a lot which are non-homogeneous with respect to material and appearance shall be excluded from the lot and shall be either treated as separate lots, or reworked. Where the area excluded from a lot as non-homogeneous exceeds 10% of the total lot area or at other specified percentage of the total lot area, the whole of the lot shall be rejected.

The characteristic value of density ratio of the lot shall be calculated as $X - 0.92S$ for six tests per lot where X and S are respectively the mean and the standard deviation of the individual density ratio test values for the lot.



The mean of density ratio is defined by:

$$\bar{x} = (x_1 + x_2 + \dots + x_i + \dots + x_n) / n$$

The standard deviation of density ratio test values is defined by:

$$s = \sqrt{\{[(\bar{x} - x_1)^2 + (\bar{x} - x_2)^2 + \dots + (\bar{x} - x_n)^2] / n - 1\}}$$

where x_i , $i=1, 2, 3, \dots, n$, is the individual density ratio test value and n is the number of tests per lot.

The relative density and moisture content / moisture ratio can be determined using the following test methods on the condition that the same test method is used consistently throughout the full testing regime:

- AS 1289.5.1.1 (lab) and AS 1289.5.8.1 (field) – Use of a calibrated nuclear gauge in the field and determination of the Maximum Dry Density (MDD) and Optimum Moisture Content (OMC) in the laboratory.

5.7.3 Placement of Fill

5.7.3.1 Embankments and Allotments

Filling to embankments and allotments shall be approved material spread in successive layers not exceeding 150mm (loose measurement) and compacted throughout to a dry density of not less than 95.0% and 100% (roadways and for upper 1m layer from finished subgrade level) of AS 1289.5.1.1 and at a moisture ratio between 90% and 105% of the optimum moisture content determined in AS 1289.5.1.1. The top 150mm of filling to allotments shall consist of topsoil obtained from stripping; graded and shaped to the finished levels shown on drawings and as specified.

5.7.3.2 Road Pavements and Footpaths

Filling under road pavements and footpaths shall be approved material approved by the supervising geotechnical engineering consultant spread in successive layers not exceeding 150mm (loose measurement) and placed by a method acceptable to the Superintendent and compacted throughout to a relative dry density of not less than 100.0% of that determined in AS 1289.5.1.1 and a moisture ratio between 90% and 105% of the optimum moisture content determined in AS 1289.5.1.1.

5.7.3.3 Benching

Where a fill is to be constructed on sidelong ground or against an existing fill, surfaces on or against which the fill is to be constructed which have a slope steeper than 4 horizontally



to 1 vertically shall, unless otherwise specified be cut progressively in the form of benches over the full area to be covered by new layer of filling (150mm maximum bench height). The width of each bench shall be such as to permit safe and effective operation of plant but shall be not less than 1m. Material excavated during benching may be used in the fill if it is of satisfactory quality.

5.7.3.4 Frequency of Testing

Testing of Moisture and density ratio shall be carried out in accordance with AS 3798 – Guidelines on Earthworks for commercial and residential developments - Table 8.1.

To ensure that consistent subgrade conditions have been achieved in the field, the following testing shall be undertaken:

a) Proof Roll test

The proof roll test remains the most practical acceptance test for subgrade stability and shall be applied to all areas of fill. The acceptance criteria shall be when an area withstands test rolling without rutting and with no visible deformation or springing. Areas failing proof-roll tests require re-treatment until accepted by the Superintendent (and/or the Council Civil Works Supervisor). Irrespective of design CBR value and expansion characteristics of the subgrade material, if soft local areas of subgrade are encountered at the time of construction, then these shall be treated or replaced in order to achieve a stable subgrade platform for acceptance by proof rolling.

b) Density and Moisture Conditions:

Unless otherwise specified, acceptance of material and work will be based on testing of the material or work in lots in accordance with Table 8.1 of AS3798.

The relative density and moisture content / moisture ratio can be determined using the following test methods on the condition that the same test method is used consistently throughout the full testing regime:

- AS 1289.5.1.1 (lab) and AS 1289.5.8.1 (field) – Use of a calibrated nuclear gauge in the field and determination of the MDD and OMC in the laboratory; or
- AS 1289.5.7.1 (lab) and AS 1289.5.8.1 (field) – Use of a calibrated nuclear gauge in the field and determination of the MDD and OMC in the laboratory using the rapid HILF method (for materials -4% to +6% of OMC).



c) Supervision:

Level 1 supervision (Refer AS 3798) of all filling works is required. Regular observations, inspection and testing should be carried out by a Civil Engineer experienced in geomechanics and familiar with the contents of this report to ensure that the fill placement has been carried out to appropriate standards and for certification purposes. The frequency of the field density testing should be equal or greater to the requirements of Table 5.1 of AS 3798:1996.

d) Granular Fill:

If Granular materials are utilised for filling works, then the density and moisture ratio will be required to be determined in accordance with AS 1289.5.2.1 (Modified compactive effort).

5.7.4 Settlement of Fill

The magnitude and rate of settlement resulting from placement of the fill materials should be monitored at various locations across the site through the installation of settlement markers until practical completion has occurred. Capturing this data is important as it provides a guideline for the future design of shallow foundations and structures on the site with the ability to accommodate differential settlements. The construction of any dwellings prior to the completion of practical settlement is not recommended.

The maximum amount of settlement is anticipated to occur in the lower lying areas of the site that is underlain by Silty CLAY soils. Consideration should be given into utilizing these probable fill areas after settlement has ended.

At completion of site earthworks and practical completion of settlement, an earthworks completion report should be prepared summarizing the works undertaken on site, presenting the results of the fill testing, and any new subsurface information gathered since this investigation. During this work an evaluation of the suitability of the works to support structures can be undertaken including available bearing capacities for shallow foundations.

5.8 Allowable Bearing Capacity

From the above it can be concluded that the existing natural ground has varying Allowable Bearing Capacity from 80kPa to 424kPa at varying depths for the respective site areas based on typical firm Silty CLAY characteristics (*refer to Section 3.2.5*). However, the following issues shall be taken into account:



- i. The top 0.5m to 1.0m of soft subgrade material (if encountered) at the low-lying areas adjacent to the stormwater channel or at the isolated low elevation area (adjacent to any existing watercourse) which has stormwater ponding or at any other location shall be replaced with suitable engineered backfill (Suva Marl – Soapstone).
- ii. No footings should bear on the entire topsoil layer or any unengineered fill layer or on the top subgrade or on tree roots inclusive of any organic matter which will decompose in time (at least 0.61m below the natural ground level). If possible, the footings should not be located beneath the water table level as the degree of difficulty in construction would increase.
- iii. Any concrete slabs shall be designed to withstand any differential settlement which can be caused by the settlement of the fill material or decomposition of organic matter (vegetation, etc).
- iv. Should the Contractor feel that after the footings are excavated, the subgrade looks weak or does not have an allowable bearing capacity equal or greater than 80kPa, further Shear Vane tests should be carried out to ensure compliance with this Report.
- v. It is recommended that DCP or SV tests should be carried out within the excavated footing base or in any filled areas during the construction phase to confirm any changes to the soil Allowable Bearing Capacity which may have taken place over time.
- vi. The Structural Engineer shall design the building footings to ensure that no force from the proposed buildings is transferred to any adjacent property subgrade which may cause heaving of the adjacent buried and exposed structures (such as buildings, footpaths, roadways).

5.9 Settlement Analysis

From the laboratory test results provided in Appendix C to D and the field test data in Appendix B, the short term and long-term settlement under the PV panels load can be ascertained by various calculations. However, under the sub-consultancy Contract we have excluded these calculations as we are not aware of the loads from the PV panels and neither are we involved in the design of the panel foundations. However, under Section 5.2, of this Report, we have commented on the site soils - Ratings for Compressibility and Shrink-Swell Potential based on:

- ✓ Plasticity Index (PI)
- ✓ Liquid Limit (LL)



5.10 California Bearing Ratio (CBR)

A design CBR of 4 to 5% (minimum) should be adopted at the site subject to the provision of a 150mm depth capping layer over the existing soft subgrade materials. Some of the highly plastic dark brown Silty CLAY detected in some boreholes or immediately adjacent to the watercourse in the low-lying areas may not be suitable subgrade materials and should be replaced with materials of low (preferable) and moderate plastic materials derived from the balance of the site. A capping layer should be provided over all naturally occurring subgrade materials to reduce the likelihood of underlying reactive soils from causing reflective cracking through the proposed pavement. The capping layer should be no less than 150mm in thickness and meet the following criteria shown in *Table 8* below.

Physical Properties			Limits of Grading (% Passing) After Compaction Sieve Size AS (mm)			Plasticity Index x % passing 0.425 mm after compaction (max.)	Plasticity Index (max) After Compaction
CBR (min) (%)	Swell (max) (%)	Permeability (max) m/s	75.0	4.75	0.075	1000	25
7	2.5	5×10^{-9}					
			100	60 - 80	10 - 40		

Table 8: Capping Layer Material Criteria.

A design CBR of 4 to 5% (minimum) can be adopted on top of the capping layer. The overall pavement thickness (including the capping layer) should not be less than 350mm in order to minimize the environmental effects (cracking of pavement due to the underlying reactive soils).

An alternative to the placement of a capping layer is the insitu-stabilisation of the existing subgrade material. Laboratory testing of the proposed subgrade materials (stabilised samples) prior to any stabilisation works is to be undertaken to determine layer thickness (minimum 250mm) and stabilization agents and quantities in order to meet the above criteria. A minimum design CBR of 4 to 5% should be adopted on top of the stabilized layer or utilising the relevant CBR which represents the road pavement in consideration.

If subgrade stabilisation is utilised, then sub surface drainage should be provided behind the back of kerb or pavement edge. In order to minimise the effects of differential subgrade drying and subsequent longitudinal cracking, the depth of the subsurface drain should not be any greater than 150mm from the top of any stabilized layer.



5.10.1 Subgrade Preparation:

All subgrade areas (even those which are based on existing materials) shall be moisture conditioned and compacted using purpose build equipment. The acceptance criteria for moisture and density shall be as follows:

5.10.1.1 Density – Existing Subgrade:

A relative density of not less than 100.0% of the density determined in AS 1289.5.1.1 (standard compactive effort) shall be obtained.

5.10.1.2 Density – Capping Layer (if required):

A relative density of not less than 100.0% of the density determined in AS 1289.5.1.1 (standard compactive effort) shall be obtained.

5.10.1.3 Density – Stabilised Layer (if applicable):

A relative density of not less than 100.0% of the density determined in AS 1289.5.1.1 (standard compactive effort) shall be obtained. Laboratory testing of the stabilized layers should be undertaken within 2 hours of final compaction if a cement additive is utilized. Loading of the stabilized layer after compaction should be undertaken in accordance with any recommendations provided in the design of same

5.10.1.4 Moisture:

A relative moisture ratio not less than 90% than the optimum moisture content (OMC) determined in AS 1289.5.1.1 (standard compactive effort) shall be obtained. Note: It may be difficult to compact materials with a moisture content greater than 105% of optimum moisture content.

5.10.2 Consistency of Subgrade Conditions:

To ensure that consistent subgrade conditions have been achieved in the field, the following testing shall be undertaken:

5.10.2.1 Proof Roll Test:

The proof roll test remains the most practical acceptance test for subgrade stability and shall be applied to all pavement subgrades. The acceptance criteria shall be when an area withstands test rolling without rutting and with no visible deformation or springing. Areas failing proof-roll tests require re-treatment until accepted by the Superintendent (and/or the Council Civil Works Supervisor). Irrespective of design CBR value and expansion characteristics of the subgrade material, if soft local areas of subgrade are encountered at the time of construction, then these shall be treated or replaced in order to achieve a stable subgrade platform for acceptance by proof rolling.



5.10.2.2 Density and Moisture Conditions:

Unless otherwise specified, acceptance of material and work will be based on testing of the material or work in lots. A lot will consist of a single layer, batch or area of like work which has been constructed or produced under essentially uniform conditions and is essentially homogeneous with respect to material and appearance. Unless otherwise specified, the extent of each lot shall not exceed one day's production or 2500m², whatever the lesser.

Six (6) density tests shall be performed at the locations determined in AS 1289.1.4.2 or other approved method. Discrete portions of a lot which are non-homogeneous with respect to material and appearance shall be excluded from the lot and shall be either treated as separate lots, or reworked. Where the area excluded from a lot as non-homogeneous exceed 10% of the total lot area or at other specified percentage of the total lot area, the whole of the lot shall be rejected.

The characteristic value of density ratio of the lot shall be calculated as $X-0.92S$ for six tests per lot where X and S are respectively the mean and the standard deviation of the individual density ratio test values for the lot.

The mean of density ratio is defined by:

$$\bar{x} = (x_1 + x_2 + \dots + x_i + \dots + x_n) / n$$

The standard deviation of density ratio test values is defined by:

$$s = \sqrt{\{[(\bar{x} - x_1)^2 + (\bar{x} - x_2)^2 + \dots + (\bar{x} - x_n)^2] / n - 1\}}$$

where x_i , $i=1, 2, 3, \dots, n$, is the individual density ratio test value and n is the number of tests per lot.

The relative density and moisture content / moisture ratio can be determined using the following test:

- AS 1289.5.1.1 (lab) and AS1289.5.8.1 (field) – Use of a calibrated nuclear gauge in the field and determination of the MDD and OMC in the laboratory.

5.10.3 Subgrade CBR:

The insitu CBR of the proposed subgrade shall be measured using a calibrated dynamic cone penetrometer at the same locations as the density and moisture tests. A minimum subgrade CBR of 4 to 5% immediately below the subgrade surface shall be required to be obtained at all locations.



5.10.4 Suitable Subgrade Materials:

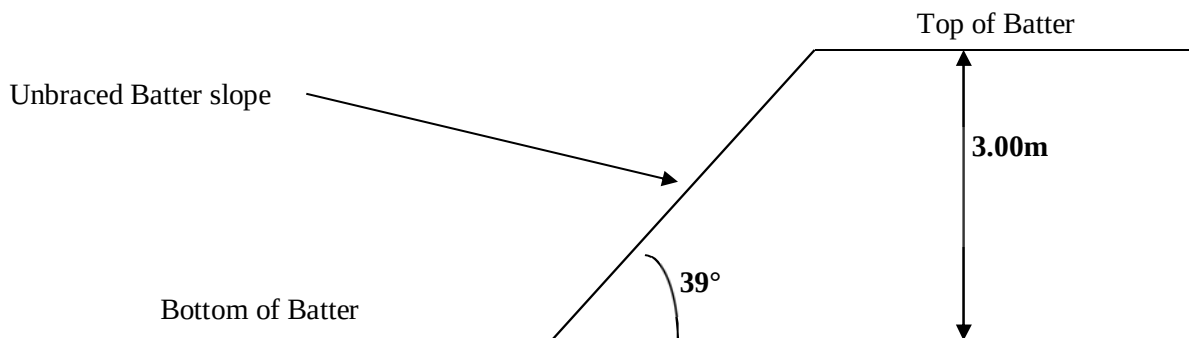
The light brown / greyish brown / light brownish intermixed coloured Silty CLAY materials are suitable subgrade materials (subject to any subgrade replacement due to soft subgrade material). Any high plasticity dark brown/blackish/reddish brown Clay is not suitable (if uncompactable or has a CBR < 4%) and should be capped/replaced, not exceeding 0.7m thick suitable capping/replacement materials (such as Suva Marl / Soapstone).

5.11 Unbraced Excavations In Cohesive Soils.

If unbraced excavations are required, Plasticity Index tests are to be carried out at the respective areas. For the site, the following tests were carried out:

- i. TP 4, PI = 13,
- ii. TP34, PI = 13;
- iii. TP41, PI = 14

From the Atterberg Limits test (PI = 10 – 20), any unbraced excavations in cohesive soils shall have a slope angle not exceeding 39 degrees with a corresponding batter vertical height not greater than 3.00m as per recommendations of Table 20.2³



5.12 Measures to Promote Site Stability (if applicable).

In conjunction with our site stability recommendations presented in this report, we provide the following additional measure to promote site stability:

- a) Prevent any excavations which will result in the removal of toe support of any hillsides/slopes.

³ (Rajapakse, 2008)



- b) Re-vegetation of hillsides/slopes.
- c) Minimise removal of trees on existing hillsides/slopes which usually result in the destabilisation of the hillsides/slopes.
- d) Excavate cut-off drains at the top of batter slopes to channel away stormwater from the batter slope. Saturation of the slope leads to increased pore water pressure and reduction of soil strength which eventuate in landslides.
- e) Excavate cut-off drains (rocklined v-invert drains) to the top of batter slopes. Saturation of the slope caused by high intensity rainfall (stormwater runoff) directly flowing over the top of the hillside and along the slope causing rillings and soil erosion.
- f) Provide proper rocklined v-invert drains to channel stormwater from the site and into existing natural watercourse(s).
- g) Provide proper rocklined v-invert drains of the existing natural watercourse(s) on site to prevent collapse of drain walls and to prevent the silting and blockage of the stormwater path which leads to minimising the probability of flooding to the low-lying areas abutting the natural watercourse.
- h) Utilise gabion basket or reinforced concrete retaining wall systems at sections which is to support existing squatter houses if adjacent to a lower-level access road or service pipeline installation to prevent any adverse settlement issues relating to the existing residential dwellings.

5.13 Geotechnical Testing and Monitoring Procedures to Be Followed During the Construction Period.

Geotechnical monitoring and testing procedures are fundamental to the sustained performance of pavements and foundations at this site. It is on this basis that detailed recommendations have been provided previously in the relevant recommendations of this report. In summary, the following testing and monitoring procedures have been recommended:

- A suitable earthworks specification for the project should be prepared in accordance with applicable standards (eg. AS 3798:1996, NZS4402:1986, and NZS4404:2004) to assist the Contractor and ensure the standard of works. The specification should specify a low air void ratio (i.e. <3%) so that the completed fill material is of a low permeability and reduces the ability of water to enter and cause swelling in the soil. Fill compaction should be carried out utilizing an appropriate vibratory compactor to achieve the minimum dry density ratio specified in Table 5.1 of AS 3798:1996 (*refer to Appendix D*) with a suitable water content.



- Compaction and moisture control of all areas upon which fill or pavements will be constructed.
- Compaction, moisture, depth and material control of all fill construction using proof rolling, insitu CBR testing and density testing.
- Regular observations, inspection and testing should be carried out by a Civil Engineer experienced in geomechanics and familiar with the contents of this report to ensure that the fill placement has been carried out to appropriate standards and for certification purposes. The frequency of the field density testing should be equal or greater to the requirements of Table 5.1 of AS 3798:1996.
- The magnitude and rate of settlement resulting from placement of the fill materials (if required) should be monitored at various locations across the site through the installation of settlement markers until practical completion has occurred. Capturing this data is important as it provides a guideline for the future design of shallow foundations and structures on the site with the ability to accommodate differential settlements. The construction of any dwellings prior to the completion of practical settlement is not recommended.
- The maximum amount of settlement is anticipated to occur in the lower lying rear area of the site adjacent to the stormwater channel / Korovuli Creek that is underlain by significant deposits of dark brown or saturated / wet soils. Consideration should be given into utilizing these areas after all backfilled settlement has ended.

5.14 Subgrade and Pavement Notes

5.14.1 Subgrade Improvement

Should poor subgrade conditions be encountered during construction, WesEng Consulting Pte Limited should be contacted to provide revised recommendations which may include subgrade improvement options.

Should subgrade improvement be required to allow construction to proceed, this could take the form of either granular (Select FILL) replacement, in situ stabilisation or sand (drainage blanket) replacement. Further information relating to the most appropriate type and depth of subgrade improvement can be provided by WesEng Consulting Pte Limited.

5.14.2 Subsurface Drainage

Providing adequate drainage is essential for the proper function of pavements. A preferred subsurface drainage system would be the installation of a 100mm diameter subsoil drain system, installed along both sides of the proposed pavement to a maximum depth of 150mm below



subgrade level or no greater than 150mm into a stabilised layer. The installation of a subsurface drainage system shall be dependent on the project budget.

To improve the trafficability of the site and to minimise any construction delays, adequate drainage of the whole site during construction should be provided. No pooling of water at the surface, either natural or stripped, and within the boxed pavements should be allowed. The pavements should be constructed with a minimum cross fall of 3%.



6 Limitations & Validations

6.1 Limitations

This report has been prepared in good faith based on the information provided by our client **NTPC Limited, India** for the **Proposed EFL Seaqaqa Solar Farm in Vanua Levu, Fiji**. Therefore, no responsibility or liability to any third party is accepted for any damages, howsoever arising, from contents of this report or its use by any third party. Where such liability cannot be excluded, it is reduced to the full extent lawful. The use of this report **is not** appropriate where there have been any changes in the nature of the project or the conditions present during any field investigation or site inspection. No responsibility or liability is accepted where any part of this report is utilised in isolation, out of context or without consideration of the total document. This report assumes that the pavement / soil profile described in the engineering logs is representative of the entire site along the proposed alignment.

If at a later time it is found that the information previously provided to WesEng Consulting Ltd was incorrect, incomplete and/or if at the time of construction, the pavement/soil conditions differ drastically from those initially reported, WesEng Consulting Pte Ltd **should be contacted immediately** and this report may be required to be reviewed and amended, if appropriate.

The recommended pavement designs for flexible pavements within this report are based on the performance requirement that significant damage can be avoided, provided that foundation site conditions are properly maintained. Maintenance of pavements should include maintaining stable moisture conditions and will require attention to the following items:

- a) Drainage of the site
- b) Limitations on over-wetting and drying
- c) Restrictions on trees and shrubs
- d) Repair of any surface deficiencies (including crack sealing) and strengthen any distressed areas.

The probability for failure for reasonable site conditions is low, but higher if extreme conditions are encountered. It is not considered possible nor economical to design for extreme conditions that could occur in the foundation if a site is not properly maintained. In some cases designers, contractors or asset owners may consider to undertake additional precautionary measures within their pavement design where extremes of wetting and drying for example, cannot be economically protected against by undertaking maintenance measures alone. The level to which these measures are implemented should be based on site conditions, reactivity of the site and the type of construction. This report applies to normal sites.



6.2 Validations

Collection of information and its appraisal, should continue during any construction works, to confirm or otherwise the assumed ground and pavement model. In most cases this is undertaken by an experienced contractor, construction surveyor, superintendent, clerk of works or engineer. In some cases, it is preferred that the geotechnical consultant assesses and monitors the ground conditions and / or pavement conditions (which may also include compaction control testing) during construction, particularly in light of potentially poor or variable ground conditions or unexpected findings.



7 References

- 1) AS 1289:2000 “Methods of Testing Soils for Engineering Purposes”.
- 2) AS 3798:1996 “Guidelines on Earthworks for Commercial and Residential Developments”.
- 3) Department of Mineral Resources, 1:50,000 Series Viti Levu Map Sheet 5 “Geology of the Seaqaqa Area”
- 4) Department of Lands, 1:50,000 Topographical Map of Lautoka (Sheet L27)
- 5) New Zealand Geotechnical Society, December 2005 “Field Description of Soil and Rock – Guideline for the Field Classification of Soil and Rock for Engineering Purposes”.
- 6) NZS 4402:1986 “Methods of Testing Soils for Civil Engineering Purposes”.
- 7) NZS 4404:2004 “Land Development and Subdivision Engineering”.
- 8) Murphy, P. H. (2007). General Levels of Interpreting for Plasticity Index, Liquid Limit and Uses. In P. H. Murphy, *Interpreting Soil Test Results. What Do All The Numbers Mean?* (p. 36). CSIRO.
- 9) (2008). Unbraced Excavations in Cohesive Soils (heights Less Than 15ft). In R. Rajapakse, *Geotechnical Engineering Calculations and Rules of Thumb* (p. 273). Oxford: Butterworth-Heinemann.
- 10) National Building Code, 1990.
- 11) GNS Science Consultancy Report 2014/261



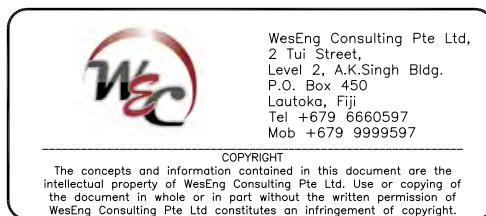
8 Appendix A – Drawings

Appended, are the following drawings:

- ✓ WEC930-G-001
- ✓ WEC930-G-002



A	ISSUED FOR INFORMATION	24/03/26
REV No:	COMMENT:	DATE:
REVISION		



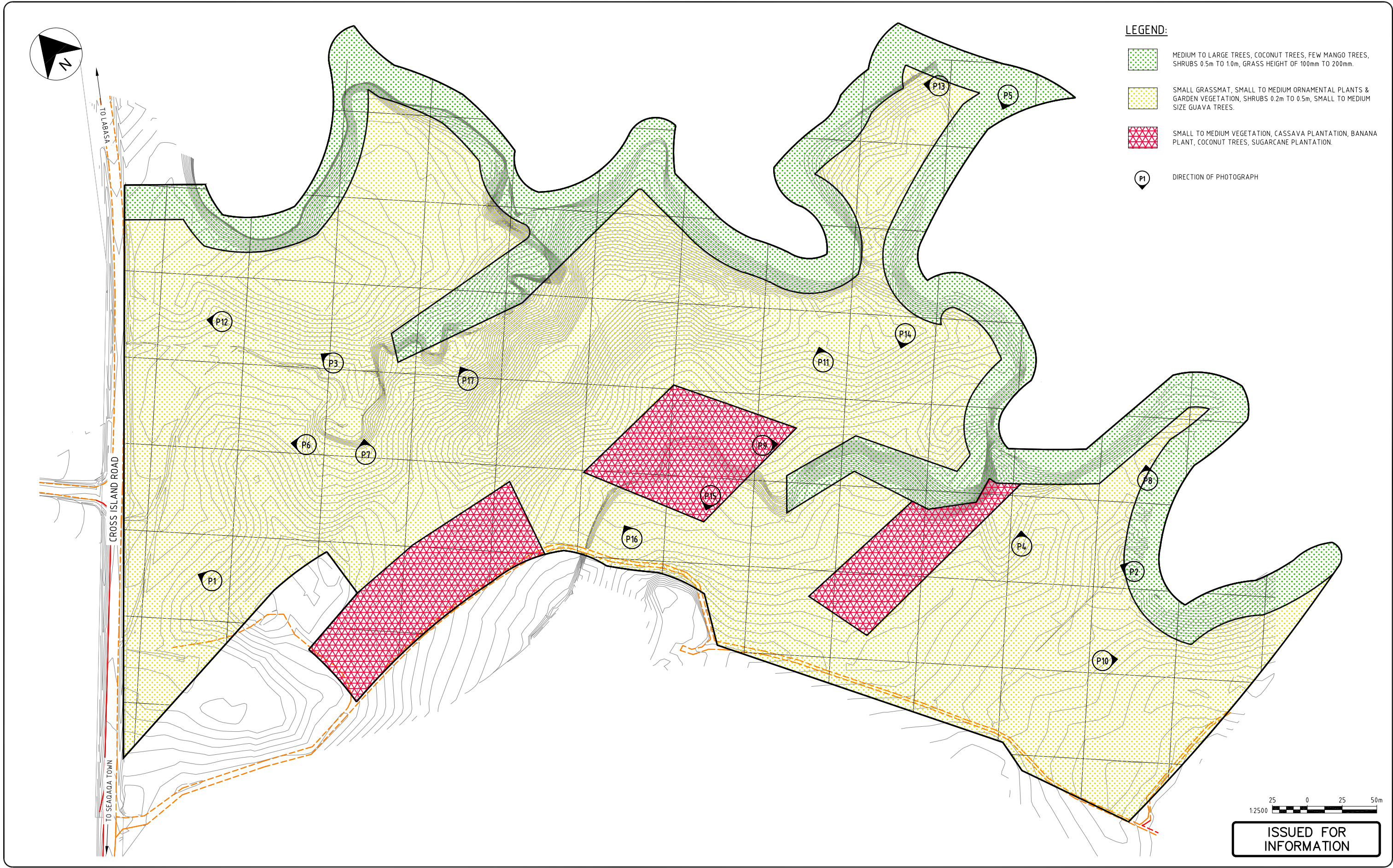
PROJECT:

NTPC LIMITED, INDIA
PROPOSED ENERGY FIJI LIMITED
SEAQAQA SOLAR FARM, SEAQAQA,
MACUATA, VANUA LEVU, FIJI

TITLE

GEOTECHNICAL LOCATION TESTING PLAN

	NAME:	DATE:	SCALE:
DESIGNED:	AKS	24/03/26	AS SHOWN
DRAWN:	MTK	24/03/26	@ A3
CHECKED:			
JOB No:	WEC 930		REV No:
DRAWING No:	WEC930-G-001		A



A	ISSUED FOR INFORMATION	24/03/26
REV No:	COMMENT:	DATE:
REVISION		

CLIENT:

एनटीपीसी
NTPC


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PROJECT:

NTPC LIMITED, INDIA
PROPOSED ENERGY FIJI LIMITED
SEAQAQA SOLAR FARM, SEAQAQA,
MACUATA, VANUA LEVU, FIJI

TITLE

GRASS MAT PLAN

	NAME:	DATE:	SCALE:
DESIGNED:	AKS	24/03/26	AS SHOWN
DRAWN:	MTK	24/03/26	@ A3
CHECKED:			
JOB No:	WEC 930		REV No:
DRAWING No:	WEC930-G-002		A



9 Appendix B – Field Hand Auger & Dynamic Cone Penetrometer Results



10 Appendix C – Laboratory Test Results – PSD & PI

The following test results are contained in this Appendix:

- ✓ Geotech Testing Pte Ltd – Particle Size Distribution (PSD) & Plasticity Index (PI) Test Results (3 No. A4 Pages).

Material Test Report



GEOTECH
TESTING PTE LTD

"Specialize in Aggregate & Soil Testing"

Geotech Testing Fiji

Vatuwaqa Laboratory

61 Millet Street Vatuwaqa Suva

Phone: 9494 506

Email: geotechtestingfiji@hotmail.com

Report Number: P26995-1
Issue Number: 1
Date Issued: 03/03/2026
Client: WESENG CONSULTING LTD

Project Number: P26995
Project Name: NTPC - EFL SEAQAQA SOLAR FARM PROJECT
Project Location: SEAQAQA, VANUA LEVU
Work Request: 6369
Sample Number: 26-6369A
Date Sampled: 10/02/2026
Dates Tested: 20/02/2026 - 25/02/2026
Sample Location: SEAQAQA SOLAR FARM PROJECT Ch: TP 41m, Depth: 1.0-1.3M
Material: RED CLAY
Material Source: SEAQAQA SOLAR FARM SITE



Accredited for compliance with ISO/IEC 17025

Approved Signatory: Jonathan Prasad
Director

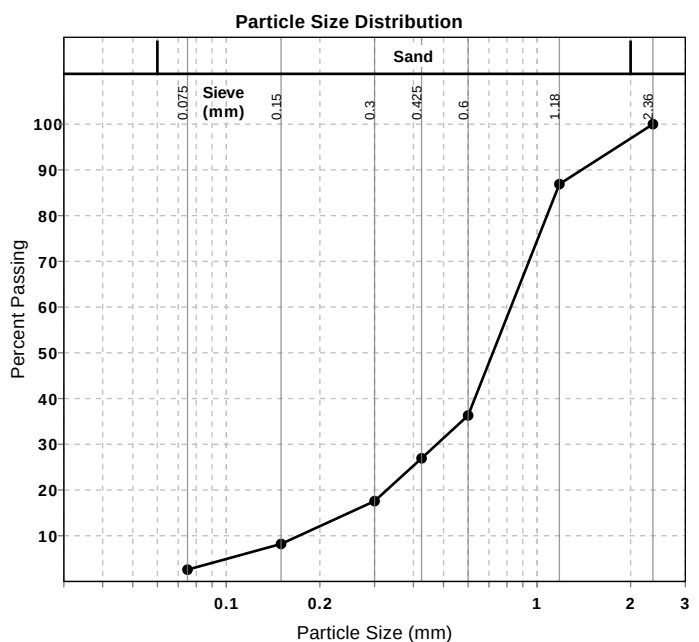
Accreditation Number: 1344

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

Particle Size Distribution (NZS 4407.3.8.1)					
Sieve	Passed %	Passing Limits	Retained %	Retained Limits	
2.36 mm	100		0		
1.18 mm	87		13		
0.6 mm	36		51		
0.425 mm	27		9		
0.3 mm	18		9		
0.15 mm	8		9		
0.075 mm	3		6		

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)				Min	Max
Sample History	Oven Dried				
Preparation Method	Dry Sieve				
Liquid Limit (%)	93				
Plastic Limit (%)	79				
Plasticity Index (%)	14				

Linear Shrinkage (AS1289 3.4.1)				Min	Max
Moisture Condition Determined By	AS 1289.3.1.1 / AS 1289.3.1.2 / AS 1289.3.9.1 / AS 1289.3.9.2				
Linear Shrinkage (%)	17.0				
Cracking Crumbling Curling	Cracking				



Material Test Report



GEOTECH
TESTING PTE LTD

"Specialize in Aggregate & Soil Testing"

Geotech Testing Fiji

Vatuwaqa Laboratory

61 Millet Street Vatuwaqa Suva

Phone: 9494 506

Email: geotechtestingfiji@hotmail.com

Report Number: P26995-1
Issue Number: 1
Date Issued: 03/03/2026
Client: WESENG CONSULTING LTD

Project Number: P26995
Project Name: NTPC - EFL SEAQAQA SOLAR FARM PROJECT
Project Location: SEAQAQA, VANUA LEVU
Work Request: 6369
Sample Number: 26-6369B
Date Sampled: 10/02/2026
Dates Tested: 20/02/2026 - 25/02/2026
Sample Location: SEAQAQA SOLAR FARM PROJECT Ch: TP 34m, Depth: 1.5-1.8M
Material: RED CLAY
Material Source: SEAQAQA SOLAR FARM SITE



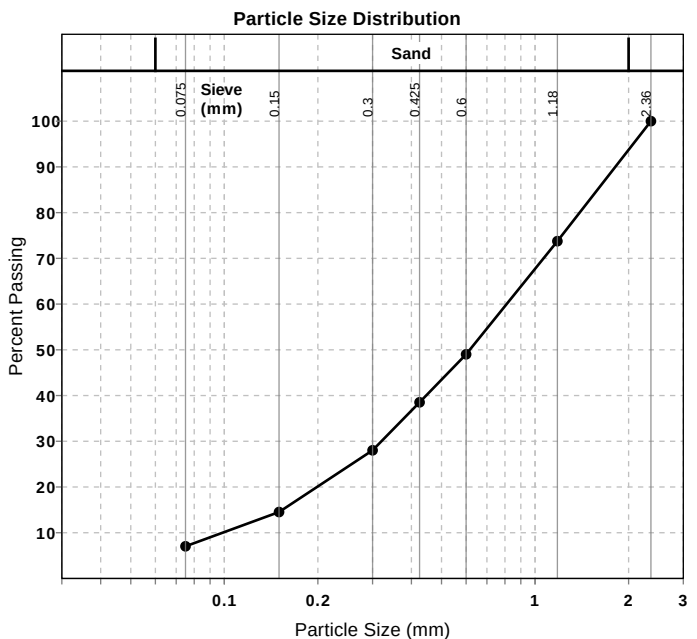
Accredited for compliance with ISO/IEC 17025

Approved Signatory: Jonathan Prasad
Director

Accreditation Number: 1344

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

Particle Size Distribution (NZS 4407.3.8.1)					
Sieve	Passed %	Passing Limits	Retained %	Retained Limits	
2.36 mm	100		0		
1.18 mm	74		26		
0.6 mm	49		25		
0.425 mm	39		10		
0.3 mm	28		10		
0.15 mm	15		13		
0.075 mm	7		7		
Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)					
			Min	Max	
Sample History			Oven Dried		
Preparation Method			Dry Sieve		
Liquid Limit (%)			84		
Plastic Limit (%)			71		
Plasticity Index (%)			13		
Linear Shrinkage (AS1289 3.4.1)					
			Min	Max	
Moisture Condition Determined By			AS 1289.3.1.1 / AS 1289.3.1.2 / AS 1289.3.9.1 / AS 1289.3.9.2		
Linear Shrinkage (%)			17.0		
Cracking Crumbling Curling			Cracking		



Material Test Report



GEOTECH
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Report Number: P26995-1
Issue Number: 1
Date Issued: 03/03/2026
Client: WESENG CONSULTING LTD

Project Number: P26995
Project Name: NTPC - EFL SEAQAQA SOLAR FARM PROJECT
Project Location: SEAQAQA, VANUA LEVU
Work Request: 6369
Sample Number: 26-6369C
Date Sampled: 10/02/2026
Dates Tested: 20/02/2026 - 25/02/2026
Sample Location: SEAQAQA SOLAR FARM PROJECT Ch: TP 4m, Depth: 1.8-2.0M
Material: RED CLAY
Material Source: SEAQAQA SOLAR FARM SITE



Accredited for compliance with ISO/IEC 17025

Approved Signatory: Jonathan Prasad
Director

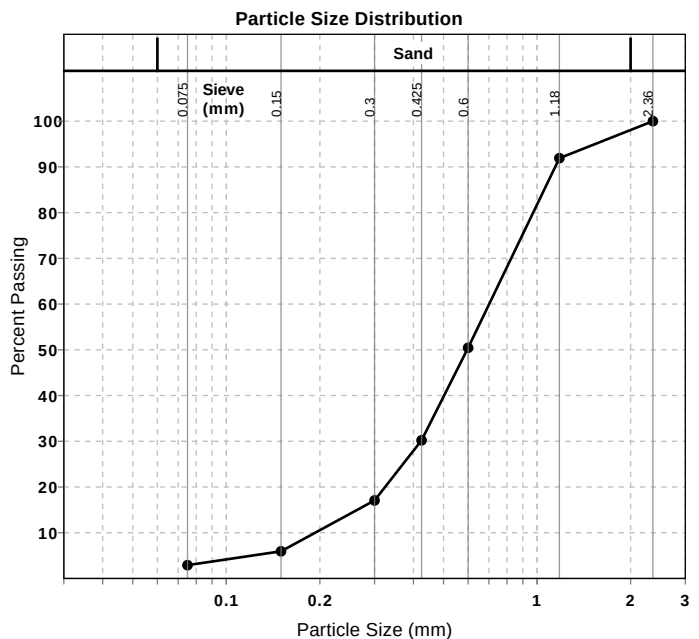
Accreditation Number: 1344

All tests reported herein have been performed in accordance with the laboratory's scope of accreditation

Particle Size Distribution (NZS 4407.3.8.1)					
Sieve	Passed %	Passing Limits	Retained %	Retained Limits	
2.36 mm	100		0		
1.18 mm	92		8		
0.6 mm	50		41		
0.425 mm	30		20		
0.3 mm	17		13		
0.15 mm	6		11		
0.075 mm	3		3		

Atterberg Limit (AS1289 3.1.2 & 3.2.1 & 3.3.1)				Min	Max
Sample History	Oven Dried				
Preparation Method	Dry Sieve				
Liquid Limit (%)	86				
Plastic Limit (%)	73				
Plasticity Index (%)	13				

Linear Shrinkage (AS1289 3.4.1)				Min	Max
Moisture Condition Determined By	AS 1289.3.1.1 / AS 1289.3.1.2 / AS 1289.3.9.1 / AS 1289.3.9.2				
Linear Shrinkage (%)	19.5				
Cracking Crumbling Curling	Cracking				





11 Appendix D – Laboratory Test Results – UCS, Triaxial, Oedometer & Shrink Swell Index

The following test results are contained in this Appendix:

- ✓ Trilab – Unconfirmed Compressive Strength (UCS); Triaxial; Oedometer & Shrink Swell Index Test Results (28 No. A4 Pages).



12 Appendix E – Laboratory Test Results – pH, Chloride & Sulphate

The following test results are contained in this Appendix:

- ✓ University of South Pacific, The Institute of Applied Science - pH, Chloride & Sulphate Test Results (2 No. A4 Pages).

ANALYTICAL RESULT(S)

Customer : Weseng Consulting Pte Ltd

Attention : Mr Andrew K Singh
Phone : (679) 6660 597 / 9999 597
Email : weseng@connect.com.fj

Address : PO Box 450
Lautoka Box



Dr Vincent Lal
Manager Laboratory Services



Dr Isoa Korovulavula
Acting Director

16 / 3 / 2026

Date

ANALYTICAL RESULT(S)
(results apply to samples as received)

Customer : Weseng Consulting Pte Ltd
Attention : Mr Andrew K Singh
Report No : RS2026/125
Sample Delivered by Customer

Sample Type : Soil
Date Received : 19/02/26

Customer I.D. Lab No.	TP4 2026/743	TP34 2026/744	TP41 2026/745	Date Analysed	Method Ref. No.
pH	(Units) 4.5 at 25.7°C	4.5 at 25.9°C	4.2 at 25.9°C	26/02/26	AP 4500 – H ⁺ B
Chloride	(mg/L) 1,896	1,773	2,385	26/02/26	SPACNET
Sulphate	(mg/Kg) 19.8	27.0	18.3	24/02/26	SPACNET

Note 1 : AP = APHA 24th Edition, 2023; SPACNET = South Pacific Agricultural Chemistry Laboratory Network, Recommended Methods for Soil, Plant, and Water Analysis, 2008


Philip Gabriel
Key Technical Person

This test report shall not be reproduced except in full



13 Appendix F – Photographs



Photograph 12: Dynamic Cone Penetration tests being carried out by the Engineering Technician at Test Location DCP 5.



Photograph 13: Hand Auger tests being carried out by the Engineering Technician at Test Location HA 51.



Photograph 14: Shear Vane tests being carried out by the Engineering Technician at Test Location TP 47.



Photograph 15: Dynamic Cone Penetration test being carried out by the Engineering Technician and Graduate Engineer at Test Location TP 37.



Photograph 16: Hand Auger tests being carried out by the Graduate Engineer at Test Location HA 33.



Photograph 17: Photograph indicating the Open Earth Drain running along the top half of the proposed site, discharging at Korovuli Creek at the North-East Boundary.



14 Appendix G – Table 5.1 from AS 3798 (1996)

Appended, are the following Tables:

- ✓ Table 5.1

Minimum relative compaction values for different applications of various projects are given in Table 5.1. These values are for a specification of the 'no value to be less than' type, and are not applicable where statistical control criteria (see Note, Clause 5.1) are applied. In the latter circumstances, the minimum acceptance criteria will be different and dependent upon the scheme adopted. This Standard does not give guidance on such values.

TABLE 5.1
MINIMUM RELATIVE COMPACTION

Item	Application	Minimum relative compaction, %	
		Minimum density ratio (Cohesive soils) (see Note 1)	Minimum density index (Cohesionless soils) (see Note 2)
1	Residential—lot fill, house sites	95 std (see Note 3)	65
2	Commercial—fills to support minor loadings, including floor loadings of up to 20 kPa and isolated pad or strip footings to 100 kPa	98 std (see Note 4)	70
3	Pavements (see Note 5): (a) Fill to support pavements Pavement— (b) Subgrade (to a depth of 0.3 m)	95 std 98 std	65 80
	(c) Sub-base courses	95 mod	not usually applicable
	(c) Base course —Heavily loaded —Other	98 mod 95 mod (see Note 5)	not usually applicable not usually applicable

NOTES:

- Test for General filling works Test for pavement construction work:
- For standard compactive effort (std) see AS 1289.5.1.1 and for modified compactive effort (mod) see AS 1289.5.2.1.
 - Density index as a means for control of achieved relative compaction may be difficult to use and interpret. Local correlations with other methods may exist and can be used where these are well established.
 - Development on this fill will be restricted to single- and some double-storey houses, with floor slab average imposed bearing pressure not exceeding 20 kPa, and strip or pad footings not exceeding imposed bearing pressures of 100 kPa. Residential developments other than these are considered as commercial. A minimum dry density ratio of 98% or higher may need to be considered if collapse on saturation or excessive settlement is likely to occur.
 - Commercial developments are likely to impose loads on fills which will have a more severe effect than those of houses, even where contact pressures are limited to those stated. The engineer should assess the load-carrying capacity and expected deformations associated with proposed filling and be satisfied the fill can perform its required function. Where highly loaded fills are proposed, the minimum relative compaction may need to be increased.
 - Where pavements will be required to carry a significant volume of heavy vehicles, the minimum compaction criteria for the upper levels of the fill and pavement materials, may need to be reviewed. For all pavements, it is essential that the specification for compaction of subgrade materials reflects the condition under which tests carried out for pavement thickness design are conducted.
 - Local practice in some areas for well-known materials may rely on acceptance criteria based on other parameters, e.g. penetrometer, AS 1289.F3.3.
 - The ground surface exposed after stripping should also be compacted in accordance with Table 5.1, to a depth of not less than 150 mm. If this is not carried out or not possible (for example, due to presence of soft ground), the consequences should be taken into account by the engineer.

- 8 The above criteria should be applied in conjunction with visual appraisal of the standard of compaction by the superintendent or as otherwise provided in the contract. The specification should define the circumstances where visual appraisal (including test rolling, if appropriate), may override test results for relative compaction.
- 9 It is recognized that in some parts of Australia, such as Central Australia and the northern parts of Western Australia, the use of modified compactive effort is preferred, because in the natural moisture content it more closely approximates the modified optimum rather than the standard optimum. This Standard does not give guidance on minimum relative compaction requirements in these areas.

Notwithstanding the need for moisture control in some circumstances, it is not necessary to routinely specify and enforce moisture control in non-critical areas, or in materials where subsequent engineering performance is unlikely to be adversely affected by placement moisture. Inclusion of such a requirement is likely to increase the cost of placement. If specified, the moisture content of the fill material after compaction will commonly be required to be in the range 85% to 115% of the optimum moisture content determined in the standard compaction test (AS 1289.5.1.1). However, the engineer should assess the applicability of this moisture range for the particular application being considered.

It should be appreciated that some materials degrade rapidly during compaction. Care will be required to avoid over-compaction of such materials, e.g. some weak or weathered rocks. The engineer may need to consider revision of the relative compaction guidelines of Table 5.1, to take account of the associated changing compaction characteristics. This will be particularly important where further breakdown occurs during laboratory compaction.

NOTE: If further breakdown occurs during laboratory compaction, the resulting moisture density relationship may not satisfactorily relate to that which existed at the time of field compaction. As a consequence, comparison of the field dry density to the laboratory maximum dry density (MDD) may not adequately reflect the design intent (if MDD reduces with breakdown), or may unfairly penalize the constructor (if MDD increases with breakdown). Where such variations are likely, testing should be carried out at the time of design and realistic requirements which reflect the design intentions should then be specified. It is not appropriate in these circumstances to sample material for laboratory testing before field compaction (see Clause 7.4).