APPENDIX 20

Preliminary Geotechnical Assessment Report

Ruakākā Energy Park Solar Farm

Prepared for Meridian Energy Ltd
Prepared by Beca Limited

07 July 2023

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Contents

Revision History

Revision Nº	Prepared By	Description	Date
1	Daan van der Have	Resource Consent	31/05/2023
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Document Acceptance

Action	Name	Signed	Date
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on behalf of	Beca Limited		

Executive Summary

Meridian Energy Limited (Meridian) are proposing to develop a new solar farm near Ruakākā, Northland across three sites; Site 1, Site 2 and Site 3. Beca Limited (Beca) was engaged by Meridian to provide a resource consent design for the proposed development. This report comprises of a preliminary geotechnical assessment supporting the resource consent design submission. The report summarises a site visit and review of available information, including site investigation data undertaken by other geotechnical consultants.

The three sites were divided into five areas of similar ground conditions. Weak and organic rich materials were encountered at the ground surface within low lying areas, underlain by medium dense to dense sands. Elevated areas typically comprise of medium dense to dense sands overlying loose to medium dense sands. A cemented sand layer was encountered near the ground surface widely across Site 2.

Surface water was observed within most of the farm drains. Groundwater levels are expected to typically be within 1m of the ground surface within low lying areas and may rise to the ground surface during prolonged wet weather. Groundwater levels are expected to typically be within 4m of the ground surface within elevated areas.

Notable identified geotechnical hazards for the development include soft and compressible soils, liquefaction in a moderate to large earthquake, hard cemented sands in Site 2 (difficult to penetrate) and buried obstructions.

The identified geotechnical hazards are not anticipated to prohibit the development of the site as a solar farm, though some may influence the economic feasibility of development in some areas.

Pile foundations are expected to be suitable to support photovoltaic panels. Thick deposits of weak soils, where encountered, may require relatively deep foundations. The cemented sand areas with Site 2 and buried trees (if encountered) may require pre-drilling for pile foundations.

Inverters may be supported on shallow foundations in areas underlain by sandy soils. Either piles or shallow foundations plus a hardfill raft may be needed in areas underlain by thick deposits of weak or organic soils.

Other structures are recommended to be positioned to avoid areas where thick deposits of weak or organic near surface soils are encountered. Shallow reinforced concrete foundations are expected to be suitable. These will need to be designed to consider earthquake loads and potentially liquefaction effects.

Some earthworks may be required to re-contour steeper parts of the solar farm site, especially in Site 1. The sandy soils are expected to be suitable for re-use as fill. Peat and organic soils including topsoil will not be suitable for use as structural fill, though may be used as a landscaping material or as part of potential new wetland areas for the development.

1 Introduction

1.1 Background

Meridian Energy Limited (Meridian) are undertaking the development of a new solar farm across three sites in Ruakākā, Northland, as shown in Figure 1-1.

Beca Limited (Beca) has been engaged by Meridian to prepare a resource consent level design and supporting technical documentation for the consent application.

This report summarises the preliminary geotechnical assessment completed for the three sites which comprises of the following scope of services:

- Review of existing geotechnical documentation supplied by Meridian.
- Desktop assessment of geotechnical conditions for 33kV transmission route.
- Site walkover by a geo-professional.
- Interpretation of existing data including preparation of a representative ground model of each site using available site data and high level liquefaction assessment.
- Preliminary assessment of geotechnical constraints and opportunities to inform design of solar farm and layout plan and 33kV transmission route and provide mitigation options for identified constraints.
- Identification of potential borrow sites (local quarries and known development sites) for import of off-site material.
- Recommendations for additional geotechnical investigation and testing to inform subsequent design phases.
- Preparation of a preliminary geotechnical assessment report summarising the results of the assessment outlined above.

This report does not address acid sulphate soils (ASS). An ASS desktop assessment has been reported separately.

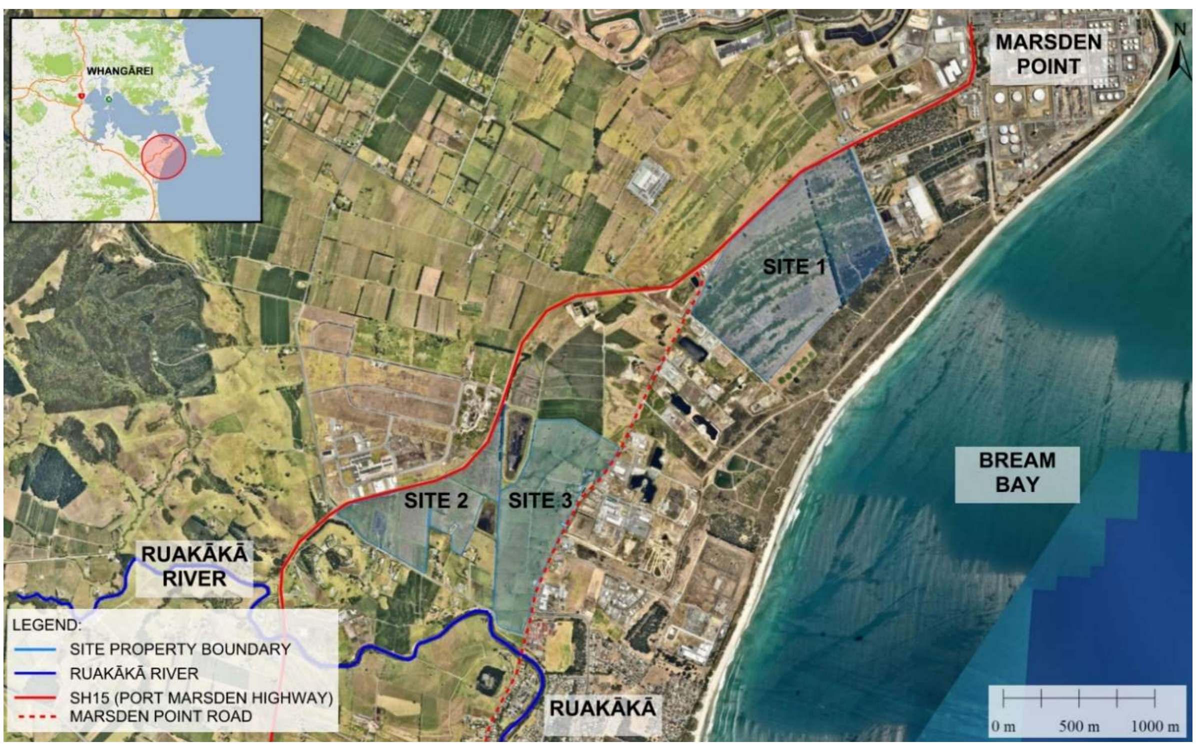

Figure 1-1: Site locations (image obtained from https://www.nearmap.com/nz/en, Nearmap Australia Pty Ltd (2023))

1.2 Existing Geotechnical Reports

Geotechnical reports have been previously issued as part of due diligence and site suitability assessments for the three sites and have been supplied by Meridian to inform this assessment.

The geotechnical reports for Site 1 include a suitability report prepared for Refining New Zealand and a site suitability report prepared for Meridian. The site suitability report focused on the northern corner of Site 1. The reports for Site 2 and Site 3 are due diligence reports prepared for Meridian and are focused on the solar farm development.

The existing geotechnical reports are summarised per site below:

1.2.1 Site 1

- Engineering Suitability Report (Hawthorn Geddes, 2019)
- Ruakākā Energy Park – BESS. Site Suitability Report (WSP, 2022)

1.2.2 Site 2

- Pre-Purchase Assessment – Ruakaka Energy Park. Advanced Development Site (Tonkin + Taylor, 2022a)
- Ruakaka Energy Park – Advanced Development Site. Further Geotechnical Considerations (Tonkin + Taylor, 2022b)

1.2.3 Site 3

- Pre-Purchase Assessment – Ruakaka Energy Park. Batten Site (Tonkin + Taylor, 2022c)
- Pre-Purchase Assessment for Ruakaka Energy Park – Batten Site. Further Geotechnical Considerations (Tonkin + Taylor, 2022d)

2 Proposed Development

Beca understand that Meridian are undertaking the consent design for the development of a solar farm. The solar farm area is approximately 200ha split over three sites. The development of a Battery Energy Storage System (BESS) in the northern corner of Site 1 is separate to the proposed development of the solar farm and not considered in this report. The main control room located on Site 1 is being developed separately as part of the BESS development. The solar farm development is expected to include the following infrastructure:

- Photovoltaic arrays arranged in rows. These arrays will be supported on frames with potential to be either fixed or with single axis tracking to follow the sun.
- Inverter stations for converting the generated electricity from DC to AC.
- Internal reticulation cables between the inverters and substation, potentially a combination of buried or overhead.
- 33 kV transmission line connecting Site 1 to Site 2 and Site 3. The transmission line may consist of overhead lines or underground cables or a combination of the two.
- Satellite control room located at Site 2 or Site 3.
- Satellite O&M facility located on Site 3, to be developed by converting an existing house, with the main O&M facility location on Site 1 being developed separately as part of the BESS development.
- Access roads to allow all weather access around the site.
- Wetlands consisting of modifying and expanding existing wetlands in Site 1 and/or constructing new wetlands within low lying areas within Site 3.
- Earthworks including site clearance and re-contouring may be required to prepare the sites for the proposed infrastructure.

3 Assessment Basis

The geotechnical assessment was based on a review of available information and a site walkover undertaken by a Beca geo-professional. No new geotechnical investigations were completed by Beca as part of the current assessment.

The following information was used to support the assessment:

- Published geological maps (Edrooke and Brook, 2009) were reviewed to provide background information regarding the expected geology underlying the site.
- The New Zealand Geotechnical Database and Beca's in-house Geotechnical Reports Database were reviewed to determine if there were any historical geotechnical investigations near to the sites.
- Historical aerial photographs were reviewed to assess historical site use and changes within the photographic history of the sites.
- The Northland 2018-2022 1m LiDAR survey was reviewed to interpret site topography (LINZ CC BY 4.0).
- Existing geotechnical reports for the three sites were reviewed to provide background information regarding previous geotechnical assessments and considerations.
- Northland Regional Council and Whangarei District Council hazard maps were reviewed to assess site hazard susceptibility.

4 Site Description

The proposed solar farm sites are located in Ruakākā, south-east of Whangārei, Northland, refer to Figure 1-1. The three sites are located to the east of SH15, Port Marsden Highway. The general topography of the sites is relatively low-lying flat to undulating terrain (typically between 2m RL and 10m RL based on LIDAR DEM) and they are currently mostly used for pastoral farming.

A site walkover was undertaken by an Engineering Geologist on 10 February 2023.

4.1 Site 1

Site 1 is bounded by the SH15 Port Marsden Highway, Bens View Road, and Marsden Point Road along the northwest boundary of the site. The northeast boundary consists of Rama Road and the southwest boundary consists of a private accessway. The southeast boundary consists of vegetated sand dunes with adjoining beach access routes.

The site predominately consists of undulating sand dunes running in the southwest to northeast direction, approximately parallel to the current coastline. The sand dune crests peak between 5m RL to 7m RL with terrain elevation changes along the length of the sand dunes of up to 2m. The troughs located between the sand dunes are at approximately 3m RL to 4m RL. Typical slope angles are 5° to 10° on the windward northwest facing slopes and 10° to 15° on the leeward southeast facing slopes. Wetlands are common in the interdune troughs across the site. The surficial soils of the interdune wetlands where exposed were observed to consist of amorphous sandy peats.

The central southwest to northeast band of the site consists of a flatter lower lying area at about 3m RL with less elevation variability.

The southeast margin of the site is densely vegetated. Drainage channels and pastoral access tracks frequently intersect the site, including a central drain several metres wide and deep known as the Bercich Drain. Water and ponding was observed within the drainage channels and wetland areas during the site walkover.

Figure 4-1: Photo of Site 1 undulating topography.

Figure 4-2: Bercich Drain running through the centre of Site 1.

Figure 4-3: Natural wetlands within Site 1.

4.2 Site 2

Site 2 is bounded by the SH15 Port Marsden Highway on the northeast of the site, McCathie Road to the south, and several private properties towards the southeast. Site 2 borders Site 3 along an approximately 75m stretch on the eastern boundary.

The site primarily consists of an elevated sand belt between approximately 7m RL and 10m RL. There is a distinct break in slope along the southeastern boundary of the site to a lower lying estuarine flood plain between approximately 3m RL to 5m RL.

Several open drains run through the site. Site observations and LiDAR review indicate that the drains are typically 2m to 6m wide and between 1m to 2m deep. A larger main drain runs northwest to southeast near the northern end of the site, flowing from the Marsden City development north of SH15 to a stormwater pond adjacent Sites 2 and 3.. This drain is up to 10m wide and 4m deep, refer to Figure 4-5. The exposed faces in the drainage channels are typically inclined at about 30° to 40°, however were observed to be standing up to 60° to 70°, indicating cementation of the sand soils. Shallow failure of the excavated bank was observed in one area. A cemented sand hardpan was observed in several locations within the drain excavations.

Areas of fill were observed on the site adjacent the highway, and a large stockpile of sand fill was observed near the boundary with McCathie Rd which is understood to have been sourced from excavation of natural dunes to expand playing fields at the nearby Ruakaka recreation centre.

Figure 4-4: Soft peaty soils visible along access tracks

Figure 4-5: Main 4m deep drainage channel running through Site 2

4.3 Site 3

Site 3 is bounded by the McCathie Road to the south and Marsden Point Road to the east. The northern and western perimeter consist of several private properties. Site 3 borders Site 2 along an approximately 75m stretch on the western boundary, separated by a large open drain.

The site predominately consists of a low lying flood plain between approximately 1.5m RL to 4m RL. There is a distinct break in slope at typically 10° along the eastern boundary of the site to an elevated ridge at approximately 6m RL to 8m RL along which Marsden Point Rd runs.

Several open drains run through the site. Site observations and LiDAR review indicate that the drains are typically 3m wide, 1m deep (up to 2m deep), and side slope angles typically around 50°. The site is known to be prone to flooding, and Figure 4-6 below shows the southern end of the site during Cyclone Gabrielle, February 2023. Observations of cut faces in drains during the site walkover indicated peaty soils across the flood plain, with the banks standing near vertically.

Transmission towers and distribution poles are located in the southern area of the site, and signage indicates that gas and fuel pipelines (understood to be the Refinery to Auckland Pipeline- RAP) run through the site.

Figure 4-6: Low lying estuarine flood plain on Site 3 during heavy rainfall

5 Geology

The published geological map of the Whangarei area (Edbrooke and Brook, 2009) indicates that the project area is immediately underlain by either Kariotahi Group sands or Tauranga Group alluvium.

The Ruakākā area is located on a coastal plain comprised of two lines of sand dunes (sand belts). One of the sand belts accumulated during the last interglacial when sea levels peaked at about 6m above present, 120,000 years ago. This is denoted as IQdt on Figure 5-1 and is mapped to be within most of Site 2 and the eastern boundary of Site 3. The other easternmost sand belt consisting of recent dunes that have been formed in approximately the last 5,000 years, this is denoted as Q1dp in Figure 5-1 and mapped to underlie the entirety of Site 1.

The lower lying area of Site 3 and the eastern boundary of Site 2 is mapped to be underlain by Holocene aged river deposits, consisting of unconsolidated to poorly consolidated mud, sand, gravel and peat deposits.

The sand dunes have been partially eroded across Site 2 and are less defined than on the outer coastal areas of Site 1 which are characterised by the more recent dunes. Peat lenses are found within the interdune sequence where natural wetlands have developed in the troughs between the dunes.

The sites are underlain by the several hundred million of years old (Permian-Jurassic) Waipapa Group greywacke at depth, generally considered to be bedrock. The Waipapa Group greywacke is mapped at the surface to the west of Site 2 and dips downwards below the sites from west to east.

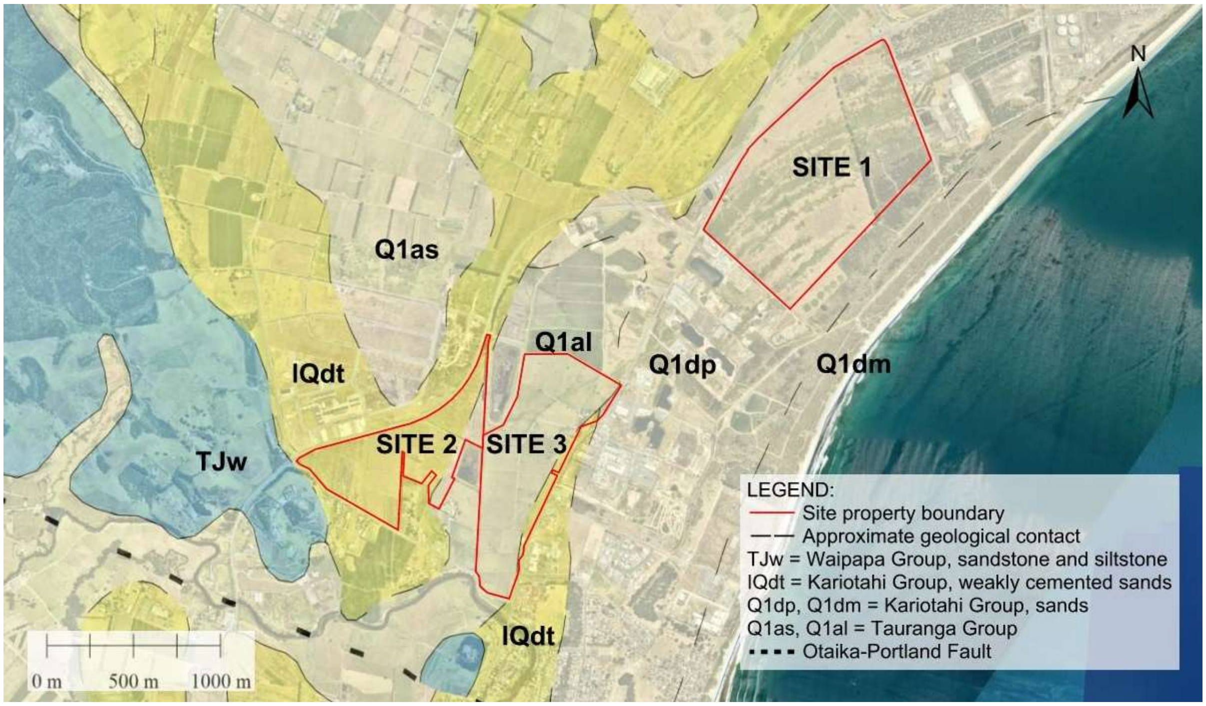

Figure 5-1: Geological Map of the Ruakākā Sites (Edbrooke and Brook, 2009) (image obtained from https://www.nearmap.com/nz/en, Nearmap Australia Pty Ltd (2023))

6 Previous Geotechnical Investigations

No new geotechnical investigations have been undertaken as part of the solar farm consent submission at the time of reporting. Previous geotechnical investigation data has been sourced from the geotechnical reports referenced in Section 1.2. An overview of these investigations is provided in Table 6-1.

Table 6-1: Total number of investigations and type undertaken per site

Investigation Type	Site 1	Site 2	Site 3
Machine borehole (BH)	-	2	3
Cone Penetration Test (CPT)	76	10	13
Dynamic Cone Penetration Test (DCP)	-	9	8
Test Pit (TP)	56	-	-

6.1.1 Site 1 – Hawthorn Geddes (2019)

Geotechnical investigations were undertaken by Hawthorn Geddes (2019) as a geotechnical due diligence assessment for the industrial development of a site for Refining New Zealand. The investigated site consisted of the same parcel boundaries as Site 1. The geotechnical investigations consisted of:

- 56 TPs: undertaken to depths between 1m and 1.5m below ground level
- 51 CPTs: undertaken to depths between 6.06m and 20.94m below ground level

Review of the testing location plan indicated that the wetter areas of the site with softer soils and/or thicker peat may not have been tested due to accessibility constraints. Inconsistencies were observed in the raw CPT data, with missing data and conflicts in the Hawthorn Geddes (2019) report. To mitigate against data inconsistencies, a holistic approach was adopted when reviewing the CPT data.

6.1.2 Site 1 – WSP (2022)

Geotechnical investigations were undertaken by WSP (2022) to assess geotechnical suitability for the BESS on the northern corner of Site 1. While the BESS is separate to the proposed development captured in this report, the geotechnical investigations provide insight to the local soil types and their strength characteristics. The geotechnical investigations consisted of:

- 20 CPTs: undertaken to depths between 4.29m and 10.29m below ground level

Only the raw CPT data for CPT01 to CPT07 was accessible for review. The remaining CPTs were reviewed based on the information supplied in the WSP (2022) report.

6.1.3 Site 2 – Tonkin + Taylor (2022a)

Geotechnical investigations were undertaken by Tonkin + Taylor (2022a) as part of a geotechnical due diligence pre-purchase assessment of Site 2 and Site 3. The geotechnical investigations within Site 2 consisted of:

- 2 BHs: undertaken to depths between 9m and 13.56m below ground level
- 9 DCPs: undertaken to depths between 0.8m and 3.9m below ground level
- 10 CPTs: undertaken to depths between 0.43m and 24.81m below ground level

6.1.4 Site 3 – Tonkin + Taylor (2022c)

Geotechnical investigations were undertaken by Tonkin + Taylor (2022c) as part of a geotechnical due diligence pre-purchase assessment of Site 2 and Site 3. The geotechnical investigations within Site 3 consisted of:

- 3 BHs: undertaken to depths between 8.3m and 16.95m below ground level
- 8 DCPs: undertaken to depths between 1.9m and 2.95m below ground level
- 13 CPTs: undertaken to depths between 1.7m and 24.72m below ground level

7 Ground Model

7.1 Ground Conditions

The ground conditions have been assessed based on the geotechnical investigation data available. The ground conditions encountered in the reviewed geotechnical investigations were consistent with the mapped geology.

The sites outlined in Section 4 have been characterised into different areas based on similarities in the encountered ground conditions. Figure 7-1 gives an overview of the Solar Farm sites and the areas with similar typical ground conditions. The ground condition areas presented in Figure 7-1 for Site 2 and Site 3 were found to be consistent with the ground models reported by Tonkin + Taylor (2022b, 2022d).

Figure 7-1: Site overview showing areas with similar ground conditions (image obtained from https://www.nearmap.com/nz/en, Nearmap Australia Pty Ltd (2023)).

7.1.1 Area A Ground Conditions

The entirety of Site 1 has been categorised as Area A due to the high level of spatial variability in the ground conditions. The variability in the Area A surficial ground conditions is due to the undulating terrain from the interdune sequences.

The Area A terrain predominately consists of recent sand dunes aligned from southwest to northeast. The sand dune crests are elevated at approximately 5m RL to 7m RL with the troughs located between 3m RL to 4m RL. The sand dunes undulate along their length.

The underlying materials are generally consistent with the mapped geology, with surficial soils observed within the interdune troughs consisting of variably thick peat and organic silts. As noted in Section 6.1.1 the thickness of these materials is not likely to have been adequately determined by previous investigations and may exceed that presented in Table 7-1 below. Thinner layers of organic soils are found along the dune crests, before transitioning to the Kariotahi Group sands.

Interpretation of CPT data indicates that the Kariotahi Group sands consist of medium dense sands overlying a variably thick dense to very dense sand zone. Below the dense to very dense zone, the soils transition back to a medium dense sand. Refer to Table 7-1 for simplified ground model for Area A.

Table 7-1: Area A simplified ground model

Unit No.	Geological Unit	Typical Description	Depth to Top of Layer (m)	Typical thickness (m)	In situ testing[1]
1a	Tauranga Group	Soft to firm PEAT / organic SILT	0	0.3	CPT q_c: 0.2MPa to 2MPa (0.5MPa)
3a	Kariotahi Group	Medium dense SAND and silty SAND	0 to 3.7	2	CPT q_c: 3MPa to 10MPa (5MPa)
3b	Kariotahi Group	Dense to very dense SAND	0.5 to 16	8	CPT q_c: 10MPa to 40MPa (20MPa)
3a	Kariotahi Group	Medium dense SAND and silty SAND	≥8	-	CPT q_c: 2MPa to 10MPa (5MPa)

[1]: Typical values shown in brackets

7.1.2 Area B Ground Conditions

Area B covers the majority of Site 2 and consists of an elevated belt of Kariotahi Group sands, typically at 7m RL to 10m RL. The underlying materials are primarily sand dune deposits and are consistent with the mapped geology. The Kariotahi Group sands in this area included a variably iron cemented hardpan layer that was difficult to penetrate. This layer was found to be up to 3m thick at one borehole location and may vary across the site area (other tests refused on the top of this layer). Within the wider region the hardpan is noted be variable in extent, thickness and strength, but typically in the order of 1m thick and encountered near surface. The material underlying the cemented sand layer comprised a loose to medium dense sand.

A hill formed of Waipapa Group greywacke is visible to the west of the Site 2/Area B extents. Waipapa Group materials were not observed at the ground surface within the Area B extents and the Waipapa Group is interpreted to dip steeply from the southwest to the northeast below the site. Where encountered, the Waipapa Group is observed to have a weathering profile from highly/moderately weathered rock to unweathered rock at depth.

Stockpiled fill is located on the western portion of the Area B. This material is understood to consist of an imported clean sand sourced from a nearby sand dune. Fill was also observed along the boundary with SH15, possibly from road construction or drainage channel excavation.

In some areas the Kariotahi Group sands are overlain by a thin layer of Tauranga Group alluvium. This material was observed to be variable, ranging from firm to stiff and consisting of an organic clayey silt or a silty peat. Tauranga Group alluvium is observed to be thicker along the eastern boundary below a distinct change in slope at the transition to Area C. Refer to Table 7-2 for simplified ground model for Area B.

Table 7-2: Area B simplified ground model.

Unit No.	Geological Unit	Typical Description	Depth to Top of Layer (m)	Typical thickness (m)	In situ testing[1]
1b	Tauranga Group	Firm to stiff organic SILT / PEAT	0	1	CPT q_c: 0.2MPa to 2MPa (1MPa)
3c	Kariotahi Group	Cemented, dense to very dense SAND	0 to 1.4	-	SPT N: ≥ 50 CPT q_c: 10MPa to 30MPa+ (CPT refusal typically at q_c = 30MPa)
3d	Kariotahi Group	Loose to medium dense SAND	3 to 5	-	SPT N: 19 to ≥50
4a	Waipapa Group	Extremely weak, highly weathered SILTSTONE	≥9.5	-	SPT N: ≥ 50
4b	Waipapa Group	Very weak, slightly weathered SILTSTONE	≥12	-	-

[1]: Typical values shown in brackets

7.1.3 Area C Ground Conditions

Area C consists of an estuarine flood plain elevated between 1m RL to 5m RL. The Area C extents cover the majority of Site 3 and the eastern boundary of Site 2. The underlying materials are consistent with the mapped geology, consisting of Tauranga Group alluvium. The Tauranga Group alluvium typically ranges from soft to stiff and consists of an organic clayey silt or a fibrous/amorphous peat. Underlying the Tauranga Group alluvium are weakly cemented/uncemented medium dense and loose Kariotahi Group sand deposits. Waipapa Group rock is expected at depth. Refer to Table 7-3 for simplified ground model for Area C.

The thickness of Tauranga Group soils reduces along the eastern boundary of Area C at a distinct change in slope at the transition to Area D consisting of an elevated dune of Kariotahi Group sands.

Tonkin + Taylor (2022c) report that large organic debris obstructions were found throughout the Tauranga Group soils. These obstructions are interpreted to consist of logs or branches, likely kauri that has been buried prior to decomposition occurring.

Table 7-3: Area C simplified ground model.

Unit No.	Geological Unit	Typical Description	Depth to Top of Layer (m)	Typical thickness (m)	In situ testing[1]
2	Tauranga Group Alluvium	Soft to Firm PEAT / organic SILT	0	3	CPT q_c: 0.1MPa to 2MPa (0.2MPa) SV: 37kPa to 64kPa (50kPa) SPT N: 0
3e	Kariotahi Group	Medium dense SAND	0 to 3	-	CPT q_c: 5MPa to 15MPa SPT N: 15 to ≥50
3f	Kariotahi Group	Loose to medium dense SAND	≥6	-	CPT q_c: 5MPa to 15MPa SPT N: 2 to 15
4a	Waipapa Group	Extremely weak, highly weathered SILTSTONE	-	-	-
4b	Waipapa Group	Very weak, slightly weathered SILTSTONE	-	-	-

[1]: Typical values shown in brackets

7.1.4 Area D Ground Conditions

Area D consists of an elevated Kariotahi Group sand belt along the elevated eastern boundary of Site 3 at 6m RL to 8m RL. Geotechnical investigations indicate that the area is underlain by medium dense sands of the Kariotahi Group, overlying dense to very dense sands.

The iron cemented hardpan found within Area B was not encountered in the investigations within the Kariotahi Group sands within Area D above the groundwater level.

While not encountered, there may be Tauranga Group alluvium soils along parts of Area D. Refer to Table 7-4 for simplified ground model for Area D.

Table 7-4: Area D simplified ground model.

Unit No.	Geological Unit	Typical Description	Depth to Top of Layer (m)	Typical thickness (m)	In situ testing[1]
3d	Kariotahi Group	Loose to medium dense SAND	0	2	SPT N: 11 to 38 CPT q_c: 2MPa to 15MPa (5MPa)
3e	Kariotahi Group	Medium dense to dense SAND	≥2	-	SPT N: 25 to ≥50 CPT q_c: 7MPa to 25MPa (10MPa)

[1]: Typical values shown in brackets

7.1.5 Area E Ground Conditions

Area E is comprised of the northern portion of Site 3 on the low-lying estuarine plain Tauranga Group alluvium soils. The Tauranga Group alluvium soils in Area E are confined by Kariotahi Group sands. The Kariotahi Group sands are interpreted to be medium dense silty sands with the Tauranga Group consisting of a soft peat or organic SILT.

Due to similarities between Area C and Area E, there is uncertainty in the boundary transition between the two Areas. The thickness of the upper Kariotahi Group sands and Tauranga Group soils are expected to reduce towards the east. Refer to Table 7-5 for simplified ground model for Area 5.

Table 7-5: Area E simplified ground model.

Unit No.	Geological Unit	Typical Description	Depth to Top of Layer (m)	Typical thickness (m)	In situ testing[1]
3f	Kariotahi Group	Loose to medium dense SAND	0	4	CPT q_c: 1MPa to 9MPa (3MPa)
2	Tauranga Group Alluvium	Soft PEAT / organic SILT	≥4	7	CPT q_c: 0.1MPa to 1MPa (0.2MPa) SPT N: 0
3e	Kariotahi Group	Medium dense silty SAND	≥11	-	CPT q_c: 2MPa to 12MPa (5MPa)

[1]: Typical values shown in brackets

7.2 Groundwater

7.2.1 Area A Groundwater

Groundwater was typically encountered between 0.5m and 3m below ground level across Area A. Due to changes in the terrain from the sand dunes, the groundwater level may be deeper along dune crests and shallower along dune troughs. Groundwater level was estimated to be at 3m RL within the Bercich drain based on LiDAR observation.

Groundwater level measurements are based on pore water pressure CPT data from September 2022.

Groundwater level site observations indicate that groundwater level is at the ground surface along drainage channels and within low lying wetland areas.

7.2.2 Area B Groundwater

Groundwater was typically encountered approximately 3m to 4m below ground level across Area B (approximately between 7m RL to 4m RL). The groundwater level is expected to follow the topography of Area B and gently slope down from the northwest to the southeast.

There is potential for perched groundwater where the cemented sand hardpan layer is present.

Groundwater level measurements are based on one off dipping of the CPTs undertaken in February 2022.

Groundwater level measurements align with site observations of the drainage channels across Area B.

7.2.3 Area C Groundwater

Groundwater was typically encountered approximately 1.5m below ground level across Area C (approximately between 0.5m RL to 2.5m RL).

Groundwater level measurements are based on one off dipping of the BHs and CPTs undertaken in February 2022.

Groundwater level measurements on the low lying estuarine plain align with Area C observations of the drainage channels across the site.

7.2.4 Area D Groundwater

Groundwater was typically encountered approximately 4m below ground level across Area D (approximately between 3m RL to 2m RL).

While not encountered, there is potential for perched groundwater where a cemented sand hardpan layer is present.

Groundwater level measurements are based on one off dipping of the BHs and CPTs undertaken in February 2022.

Groundwater level measurements on the low lying estuarine plain align with site observations of the drainage channels across Area D.

7.2.5 Area E Groundwater

Groundwater was typically encountered approximately 1.5m below ground level across Area E (approximately between 4m RL to 2m RL).

Groundwater level measurements are based on one off dipping of CPTs undertaken in February 2022.

Groundwater level measurements on the low lying estuarine plain align with Area E observations of the drainage channels across the site.

8 Potential Geotechnical Hazards

Identified geotechnical hazards are discussed below. Refer to Section 9 for recommendations for development to mitigate these geotechnical hazards.

8.1 Soft Ground

Soft and compressible ground is expected to be encountered across the low lying areas of the site. Geotechnical hazards associated with soft ground include low bearing capacity for shallow and pile foundations, low lateral capacity for pile foundations and high settlement for shallow foundations and fill loads.

Soft ground is expected to be present across much of the solar farm sites, with a significant variation in thickness. This primarily includes organic clayey silts and peaty deposits of the Tauranga Group. Thick layers of soft ground that will influence development were predominately encountered within the low-lying estuarine plain within Unit 2 in Area C and Area E, and Unit 1a in the low-lying inter-dune areas of Area A.

8.2 Cemented Sand

Cemented sands are a hardpan layer encountered above the groundwater level within Kariotahi Group sands within Unit 3c in Area B. The top of this layer was encountered between the ground surface and 1.4m deep. The thickness in one borehole was 3m, though this may vary across the area.

Cemented sand layers could also potentially be encountered with Area 1, noting that some locations met CPT refusal at around 6m depth.

These cemented materials could influence the design and construction methodology for driven pile foundations (e.g. difficult to install a driven pile).

Historical developments near the site encountering these materials are understood to have required excavators fitted with ripping tynes and hydraulic breakers to break up the cemented sand material, where encountered.

8.3 Seismic Hazards

8.3.1 Ground Shaking

The Ruakākā area has relatively low seismicity within New Zealand. These loads will need to be considered but are unlikely to have a significant influence on the solar structures relative to wind loading. Seismic loads are likely to be more influential on the design of the stations and control rooms.

8.3.2 Liquefaction and Cyclic Softening

a. Liquefaction

Liquefaction is a phenomenon where saturated granular soils temporarily lose strength due to high pore pressure development during and after earthquake shaking. Liquefaction predominately occurs in loose non plastic silts and sands below the groundwater table. Liquefaction is expected to occur in a ULS seismic event. Geotechnical effects associated with liquefaction include differential settlement, surficial settlement, negative pile skin friction loads, and surface ejecta breakthrough.

A quantitative liquefaction assessment has not been undertaken. A liquefaction susceptibility review has been undertaken on a qualitative basis and review of the previous geotechnical reports, refer to Section 1.2. The liquefaction susceptibility review is reported based on the ground condition areas reported in Section 7.1:

- Area A Unit 3a is considered to have a medium susceptibility to liquefaction, due to a relatively high groundwater and expected variability in the sand density. The liquefaction effects are anticipated to vary across Area A due to localised variations in soil density and groundwater level.
- Area B is considered to have relatively low susceptibility to liquefaction due to the thick layer of cemented Kariotahi Group sands (Unit 3c) above the groundwater table. Some parts of Unit 3d, typically below 6m below ground level, could result in differential foundation settlement, surficial settlement, and ejecta breakthrough. Liquefaction effects are expected to vary across the site due to localised variations in density, groundwater level, and cementation across the site.
- Area C and Area E (Unit 3f and parts of Unit 3e) are considered to have a relatively high susceptibility to liquefaction due to the high groundwater level and presence of loose sand below the Tauranga Group soils. This is expected to result in widespread differential settlement, surficial settlement, and ejecta breakthrough. There will be localised variability in the liquefaction effects across the site.
- Area D is expected to have low susceptibility to liquefaction at the near surface due to the lower groundwater level and denser sands encountered.

b. Cyclic Softening

Cyclic softening can occur in weak cohesive soils under earthquake shaking. Cyclic softening causes the shear strength of cohesive soils to reduce under repeated cyclic actions.

Cyclic softening has not been specifically evaluated for the sites due to the encountered geology. Soils susceptible to cyclic softening that may be present across the site are Unit 2 in Area C and Area E.

8.3.3 Fault Rupture

No active faults are known to exist within the Northland region. The nearest inactive fault is the Otaika-Portland Fault, located approximately 1km southeast of the solar farm extents (Edbrooke and Brook, 2009 and GNS Science, 2022).

8.4 Acid Sulphate Soils

Acid sulphate soils (ASS) are naturally occurring soils that are commonly found in coast or near-coastal sediments deposited in the last 10,000 years. ASS can be highly acidic in situ or can become acidic when excavated and exposed to air.

Where present, ASS can increase the corrosivity of the soil to concrete and steel. Any excavated ASS can cause adverse environmental effects, such as acidic run-off, and could require neutralisation prior to disposal or re-use as a fill material.

Refer to Beca, 2023, *Acid Sulphate Soils Desktop Information Review* for recommendations to address this hazard.

8.5 Shrink Swell Soils

Seasonal shrink swell movements at the ground surface due to changes in moisture content can occur in in near surface soils. This can result in strain on structures and can lead to cracking and differential settlement.

Shrink swell effects are not expected to affect the sandy soils in Area B and Area D. There may be localised clayey Tauranga Group soils in Unit 1 that may require further testing to assess their susceptibility. Where peat and organic silts are encountered, shrink swell effects are not expected to govern design due to high compressibility, discussed in Section 8.1.

8.6 Buried Obstructions

Tauranga Group soils (Unit 2) within Area C and Area E (parts of Site 2 and most of Site 3) may contain large buried obstructions (such as fallen kauri trees or other organic matter) that may present an obstruction during construction of foundations or excavation of trenches.

Underground services, such as the RAP and associated gas line should be positively located and avoided, protected or relocated in accordance with utility owner requirements where required.

8.7 Slope Instability

No active or historic landslides are shown on published maps anywhere within, or close to, the site boundaries.

The low-lying parts of the site are not anticipated to be affected by slope instability under static or non-seismic loads, other than potential small scale surficial instability within a few metres of existing farm drains or stormwater ponds.

Liquefaction with near surface sandy soils could potentially trigger lateral spreading movements near the farm drains, with potential for some lateral ground movements over a wide distance.

8.8 Tsunami

Tsunamis are generated by sudden movements of the sea floor caused by local or distal earthquakes, volcanic eruptions, or local submarine or coastal landslides.

The solar farm sites are mapped within the 500-year and/or 2,500-year return period tsunami inundation zones on the Northland Regional Council hazard portal (Northland Regional Council, 2023).

In the event of inundation of the site, structures would likely experience significant hydro dynamic forces. Performance requirements for the solar farm development under hydro dynamic forces will need to be agreed.

8.9 Coastal Erosion

The solar farm sites are mapped outside of the 100-year coastal erosion extent (Northland Regional Council, 2023).

9 Recommendations for Development

9.1 Solar Farm Development Area

Most of the site area is considered suitable for development as a solar farm, provided it is designed to accommodate the geotechnical hazards summarised in Section 8. The risk of damage during a significant seismic event may require acceptance by Meridian (ground deformation).

Development within areas where deep organic and soft soils are encountered (Area C, Area E and the inter-dune parts of Area A) will need specific foundation design to manage the effects and will require larger deeper foundations to support the solar loads in soils below these weak layers. These foundations will increase costs relative to a site underlain by better ground conditions, with potential to influence the economic feasibility of development in these areas.

The cemented sands encountered in Area B will likely increase construction costs by requiring pre-drilling or another technique to allow pile foundations to support the photovoltaic panels. This may influence the economic feasibility of development in these areas.

Liquefaction and the associated effects in a moderate to large earthquake event would likely damage the proposed development and require repairs. Most structures are expected to be able to be designed to accommodate the effects without collapse, though this will require design to confirm.

9.2 Solar Foundations

Photovoltaic panels will be set at levels above a design flood event, resulting in a varying height above ground levels across the site and an associated variation in the foundation loads.

Pile foundations are expected to be suitable for all sites, with potential to adopted driven piles, screw piles or possibly bored piles (noting these are not usually preferred for solar developments). Piles will be supported in sandy soils with little support provided by peat and organic soils where encountered. Pile foundations in Area C, Area E and the swampy inter-dune areas of Area A will be longer than those in other areas to allow for the thick layers of peat and organic soils at these locations.

Cemented sands were encountered in Area B that may be difficult to install a driven or screw pile through. Piles in these areas may require pre-drilling to loosen these materials or some other modification of the construction technique.

Pile foundations supported in medium dense sands would likely be affected by liquefaction induced strength loss in a moderate to large earthquake event, and associated post earthquake vertical and lateral movements. Foundations are expected to be able to be designed to accommodate these movements without collapse, though this will require confirmation during design.

Shallow foundations (e.g. a reinforced concrete footing) could be considered to support the photovoltaic panels in areas where sandy soils are encountered close to the surface. These would not be suitable in areas where deep organic soils were encountered (Area C and Area E, inter-dune parts of Area A). Localised dig outs would be needed in other areas to undercut and remove weak soils. Shallow foundation design will need to consider season shrink-swell movements and the effects of liquefaction in a moderate to large earthquake event (reduced bearing capacity and post -earthquake ground movements) .

9.3 Inverter Foundations

Inverters will be set at levels above a design flood event, resulting in a varying height above ground levels across the site and an associated variation in the foundation loads.

Shallow or pile foundations could be considered to support inverters in Area B, Area D, and elevated parts of Area A. Refer to Section 9.2 for potential pile foundation options and identified constraints.

Inverters within Area C, Area E, and low-lying parts of Area A may require pile foundations or shallow foundations supported on improved ground (e.g. excavate and replace organic and weak soils). Refer to Section 9.2 for potential shallow foundation constraints.

9.4 Earthworks

Earthworks willy be required to contour the solar farm sites for development and excavate and replace unsuitable materials at selected locations (e.g. beneath foundations and access tracks).

Earthworks will need to be managed in accordance with any requirements of an Acid Sulphate Soil Management Plan

Site 1 may require significant levelling earthworks to construct the proposed solar farm infrastructure due to the undulating sand dune terrain, or the use of terrain tracking type framing. Site 2 and Site 3 are flatter and are likely to require less extensive earthworks to contour ground levels.

9.4.1 Excavation

Excavation with sandy soils within Area B, if required, is expected to encounter a cemented hardpan as discussed in Section 8.2 above. There is also potential for cemented materials to be encountered elsewhere across the site, though earthworks in these areas is not currently envisaged to be required. The cemented sand hardpan may require ripping tynes or hydraulic breakers to break up prior to excavation.

Excavations in sandy materials are recommended to be formed at grades of 3:1(H:V) or flatter, noting that the solar development would likely prefer gentler grades within the solar development areas.

Excavation of unsuitable soils, including removal of topsoil and undercut of any underlying organic soils or uncontrolled fill will be required where shallow foundations or access tracks are proposed. These could be challenging in Area C and low-laying inter-dune parts of Area A given the depth of weak materials expected. Development is recommended to avoid or minimise excavation within these areas.

Excavations within low lying parts of Area C to form new wetlands (if undertaken) are likely to encounter weak soils.

Excavations for new wetlands in weak soil are recommended to be formed at a grade of 4:1(H:V) or flatter to allow for planting and access.

9.4.2 Settlement

Placement of fill over areas with soft Tauranga Group peat or alluvium soils is expected to cause ground settlement (Area C and Area E, and low-lying inter-dune parts of Area A). Settlements are expected to be variable and occur over an extended amount of time. Peat and organic soils can experience creep settlement, occurring over years to decades.

Development within these areas is recommended to avoid or reduce fill placement where reasonable to do so to reduce settlement effects.

Post construction settlement effects on the development can be reduced by a combination of excavation and replacement of near surface materials, preloading (overfilling) and allowing time for settlement to substantially complete before constructing other infrastructure.

9.4.3 Site-won fill

Site sourced sand from Area B, Area D, and elevated parts of Area A is likely to be suitable for reuse as engineered fill. Due to the potential sorting of grain sizes during dune deposition blending of excavated sand from a wide area across the site may create a more suitable graded material enabling a higher level of compaction. Site compaction trials and laboratory testing are recommended to inform achievable compaction.

Soft alluvium and/or peaty soils (i.e. Area C, Area E, and low-lying parts of Area A) are not expected to be suitable for use as engineered fill. Some of these materials may be able to be used as landscaping fill for example bunds, otherwise will likely require offsite disposal.

9.4.4 Imported Fill

Should additional material be required for bulk fill, suitable materials may be able to be sourced from other developments in the area or dredging activities. As these developments change over time, enquiries with local contractors are recommended once a construction date for the solar farm is known and quantity of fill required determined.

Imported aggregates will be needed for pavement layers to form permanent all weather access tracks.

There are a number of nearby quarries supplying a range of materials suitable for bulk fill through to higher quality aggregates for hardstands and roading. The nearest known quarry to the site is Sea View Quarry in the Takahiwai Hills supplying greywacke aggregates. Higher quality greywacke aggregates are available from Mountfield Quarry and Huband’s Millbrook Road Quarry in Waipu.

9.5 Satellite Control Room

The satellite control room is understood to be proposed for Site 2 or Site 3. The structure is recommended to be positioned to avoid areas underlain by deep organic and soft soils (i.e. Area C and Area E).

Foundation options for the satellite control room will depend on the selected site and ground conditions. A site underlain by deep weak soils may require either pile foundations or shallow foundations supported on a thick hardfill raft.

Shallow foundations are likely to be suitable for a site underlain by sandy soils. The foundation design will need to consider liquefaction effects which may require strengthening to accommodate the temporary strength reduction and post earthquake ground movements.

9.6 Satellite O&M Room

The satellite O&M room is understood to be retrofitted into an existing house within Site 3.

Should a new satellite O&M room be proposed, the geotechnical recommendations for the satellite control room would apply (refer Section 9.5).

9.7 Access Roads

It is understood internal site access roads are likely to consist of unbound granular pavement supported on low embankments set at or above the surrounding ground level. Earthworks to support new access roads are discussed above.

New access roads over weak soils will need to allow for settlement and for low subgrade strength. Geosynthetic fabrics may be suitable to provide additional strength and a separation layer over weak subgrade areas.

Construction and maintenance traffic (e.g. heavy trucks and construction plant) may control the design of access road pavements. Additional operation costs may be incurred to periodically top up the pavements as a result of ongoing settlement within areas underlain by weak soils to avoid ponding and flooding.

9.8 33kV Transmission Line

The 33kV transmission line is proposed between Site 1 to Sites 2 and 3. The route may be a combination of overhead and underground transmission. It is anticipated this route will be from Site 3 along Marsden Point Road and Bens View Road to Site 1. The lines/cable may then run through Site 1 towards the BESS, near the boundary with SH15.

Ground conditions along the route between Sites 3 and 1 are anticipated to be Kariotahi Group sands, similar to Area D described in Section 7.1.4. Site specific geotechnical investigation is recommended to confirm.

Pile foundations (e.g. bored piles) are expected to be suitable for overhead pole structures. These will need specific design, including consideration of weak soils at the foundation location and the liquefaction hazard.

Underground cables could encounter materials of variable thermal resistivity (e.g. sands and organic soils), requiring further geotechnical investigation and testing to confirm.

10 Recommendations for Future Work

Recommended geotechnical investigations for subsequent stages of the project are set out below. This proposed testing will assist to better characterise site constraints and provide more accurate site data for design. A detailed scope of work can be prepared once the development and key structure locations have been determined.

- Cone penetration tests (CPTs) are recommended to be completed to assess the thickness of weak sediments across the site and to provide soil strength data to assess appropriate foundations. CPTs will also be used to assess the liquefaction hazard. Where weak clayey sediments are encountered, CPTs will likely require supporting testing (e.g. shear vane testing in hand augers) to confirm soil strength.
- Test pits may also be used to characterise the near surface ground conditions. Test pits will be useful characterise the depth and potentially the thickness of the cemented sand layer within Area B.
- Machine boreholes are expected to be required at structure locations to assess both settlement potential and strength of soils. Soil sampling and laboratory testing are likely to be needed to determine representative soil stiffness parameters.
- Machine boreholes will be needed in Area B to measure the thickness and variability of the cemented sand, if test pits cannot penetrate this material, and to characterise the underlying materials. Laboratory testing comprising UCS tests may be needed to characterise the material strength.
- Installation of groundwater monitoring instrumentation e.g. standpipes to measure seasonal fluctuation in groundwater levels.
- Boreholes are recommended along the 33kV transmission line at proposed overhead structure locations. CPTs may be adopted to assess the liquefaction hazard. Underground sections of the transmission cable will require test pits plus samples to complete electrical resistivity and thermal conductivity testing.
- Additional sampling and testing to assess site soil corrosivity for below ground foundations will also be needed.

11 References

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12 Applicability Statement

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