Appendix 11

Geotechnical Investigation (GT12-156-001R REV 1)

Prepared by ETS Geotechnical

RATCH AUSTRALIA CORPORATION

GEOTECHNICAL INVESTIGATION

MOUNT EMERALD WIND FARM

REPORT NUMBER: GT12-156-001R REV 1

JULY 2013

REVISION 1

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Document Status							
Revision No.	*Author*	*Reviewer*	*Reason for Issue*	*Approved for Issue*			
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1.0 INTRODUCTION

Engineering Testing Services Pty Ltd (ETS) has conducted a geotechnical investigation for the proposed Mount Emerald Wind Farm on the Atherton Tablelands, Queensland. The proposed site is located approximately 7km west of the township of Walkamin, 21km south of Mareeba and 22km north of Atherton. The investigation was undertaken for RATCH Australia Corporation (RATCH).

It is understood that the proposed wind turbine farm will involves the construction of up to 75 turbines of 2-3MW generation capacity. The turbines will have a rotor diameter of 90m to 100m and these will be mounted on steel tube towers 80m to 90m high. RATCH has envisaged a raft footing system of 15m to 20m in diameter, 1.5m to 2m deep of a rock anchor type.

The scope of the work for the investigation comprised ten (10) boreholes to a depth of 10m or 6m of solid rock coring, followed by laboratory testing, engineering analysis and reporting.

The aim of the investigation was to identify materials, material properties and groundwater conditions to enable a geotechnical assessment and advice to be provided on the following:

- General geological description of the site;
- Provide descriptions of the soil and rock types encountered and comments on the water table depth if encountered;
- Provide engineering design parameters for the soil and rock types encountered;
- Advice on footing design options;
- Advice on bearing capacities;
- Provision of advice on any other geotechnical issues or hazards that come to light during the investigation which may impact on the design or construction.

2.0 FIELD WORK

Fieldwork was conducted by ETS from the 4th February 2013 to the 16th February 2013 and included a visual assessment of the various sites, their surrounds and subsurface investigations. The investigation consisted of drilling 10 boreholes to a maximum depth of 10m, or 6m of rock, which ever came first. The boreholes were drilled close to the locations highlighted by RATCH. Standard Penetration Testing (SPT) was carried out where possible to assist in assessing the consistency and density of the subsurface materials. Where rock was encountered core samples were collected and placed into core trays for rock logging and testing. Soil resistivity testing was carried out at each of the borehole locations to provide soil resistance parameters for the design of electrical earthing devices.

The results of the field work (borehole logs) are presented in Appendix B. The approximate locations at which the field work was conducted are displayed in Figure 1, Appendix A.

3.0 STANDARDS & GUIDELINES

The soil, rock classification descriptions, field and laboratory testing were carried out in general accordance with the following Australian Standards.

AS 1726-1993	Geotechnical Site Investigations
AS 1289	Methods of Testing Soils for Engineering Purposes
AS 4133	Methods for Testing Rocks for Engineering Purposes

4.0 LABORATORY TESTING

The following laboratory testing was conducted at NATA accredited laboratories on samples recovered during fieldwork in order to assist with the assessment of geotechnical design parameters:

- Atterberg Limits
- Particle Size Distribution Analysis
- Point Load Test
- Unconfined Compressive Strength

Results of laboratory testing are presented in Appendix C.

5.0 GEOLOGY

Numerous volcanic sequences have been identified in the Atherton area of which the majority have been subdivided into the Featherbed, Koolmoon, Sundown and Scardons Volcanic Groups. The Mount Emerald/ Walsh's Bluff topographic feature are believed to represent outflow deposits of the Glen Gordon Cauldron and are members of the Koolmoon Volcanic Group. This group comprises of Early Permian densely welded rhyolitic ignimbrite and minor rhyolitic lava. The lava flows are commonly flow banded and autobrecciated. The group has well developed columnar jointing and is up to 600m thick.

6.0 SITE CONDITIONS AND OBSERVATIONS

6.1 Visual Assessment

The proposed wind farm site is located to the north of Mount Emerald and is situated approximately 3.5km to the southwest of Walkamin. The site covers an area of approximately 15km^2 with elevation variations of up 550m throughout the site. Large relatively flat sections exist on the site between hilly peaks to the northeast and southwest of the site. The site was generally covered in medium dense bush consisting of small to large trees, long grass and shrubs. Several small creeks were encountered across the site.

Numerous large rock outcrops were observed throughout Mount Emerald and in the vicinity of all of the drill sites.

Site access was via a dirt track suitable for four wheeled drive vehicles, the track crossed the site from north and south. A second major track was used to gain access to borehole locations BH1-BH4, this track was low-lying and become impassable after periods of rainfall due to boggy ground.

6.2 Subsurface Conditions

The investigation consisted of drilling 10 boreholes to a maximum depth of 10 metres, or 6 metres of rock, whichever came first. A shallow soil profile existed at the borehole locations and was generally found to be less than 1 metre thick. The soil consisted of a Sandy SILT (ML) of low plasticity, with fine to coarse sand and of a stiff consistency. The soil was underlain by members of the Koolmoon Volcanic Group which generally consisted of rhyolitic ignimbrite. The ignimbrite was slightly weathered to fresh with minor moderately and highly weathered zones. Generally the weathering was limited to iron oxide discolouration without any significant affect to the rock strength. A preliminary field assessment of rock strength indicated the rock was generally within the high to very high strength range which was confirmed with laboratory strength testing.

Discontinuities in the core were generally confined to joints dipping at high angles relative to the core axis. Infill was found in some of the joints and consisted as either iron oxide or clay. Infill generally occurred as veneer thickness or less than 5mm. Minor faults were also identified in some of the boreholes and were found to dip between 0° and 26° relative to the core axis. Infill was clay and/or gouge material.

7.0 ENGINEERING ASSESSMENT AND RECOMMENDATIONS

7.1 Design Parameters

Rock strength has been determined using a variety of methods including field strength correlation assessments as outlined in AS1726, Point Load Testing and Unconfined Compressive Strength Testing. Tables 1 and 2 present the findings of those assessments.

TABLE 1. ROCK STRENGTH PARAMETERS- UNCONFINED COMPRESSIVE STRENGTH

Borehole	Depth (m)	Unconfined Compressive Strength	AS1726 Strength Term
BH1	0.67 -0.85	74.8 MPa	Very High
BH2	0.96 - 1.16	164 MPa	Very High
BH4	0.50 - 0.70	40.5 MPa	High
BH7	1.30 - 1.55	221 MPa	Very High
BH9	3.05 - 3.29	56.3 MPa	High
BH10	0.40 - 0.53	37.8 MPa	High

TABLE 2. ROCK STRENGTH PARAMETERS- POINT LOAD TEST

Borehole	Depth (m)	Point Load Test $I_{s(50)}$	AS1726 Strength Term
BH2	1.21 – 1.45	13 MPa	Extremely High
BH4	1.5 – 1.74	7 MPa	Very High
BH5	5.85 – 6.03	1.7 MPa	High
BH8	2.39 – 2.57	2.6 MPa	High
BH9	0.22 – 0.48	6.3 MPa	Very High
BH10	3.93 – 4.18	4.1 MPa	Very High

Laboratory testing was completed to determine modulus parameters required for detailed footing design. A summary of these results is presented in Table 3. The laboratory reports are presented in Appendix C.

TABLE 3. MODULUS PARAMETERS

Borehole	Depth (m)	Tangent Modulus	Poisson Ratio	Secant Modulus	Poisson Ratio
BH1	0.67 -0.85	43 GPa	0.074	42 GPa	0.074
BH2	0.98 - 1.16	58.6 GPa	0.084	55.6 GPa	0.084
BH4	0.50 - 0.70	33.5 GPa	0.118	29 GPa	0.128
BH7	1.30 - 1.55	58.9 GPa	0.088	56.3 GPa	0.088
BH9	3.06 - 3.29	53.6 GPa	0.055	52.5 GPa	0.055
BH10	0.40 - 0.53	40.3 GPa	0.073	43 GPa	0.073

7.2 Footing Recommendations

Based on the information collected during the preliminary subsurface investigation, the subsurface conditions are suitable for support of the turbine foundations on a raft foundation system. Since shallow bedrock was encountered at the borehole locations, other potential alternatives for supporting the proposed turbines include rock anchors and rock socketed piers.

It is understood that the customer would prefer to use a raft or rock anchor system at this site. The brief provided to ETS outlines a proposed raft design founded approximately 1.5m to 2.0m deep and 15m to 20m in diameter.

Shallow foundations will likely require rock excavation or blasting at most locations to achieve the planned foundation depth, or placing the foundations at the bedrock surface and raising the site grade to provide the required soil cover over the foundations for overturning stability. If the bedrock surface slopes significantly, or a combination of soil and rock is encountered at the planned foundation elevation, lean concrete will be required to create a level, uniform bearing surface to support the foundations.

The rippability characteristics of the rock encountered during the investigation are discussed in Section 7.3 of this report. Should blasting be considered at this site a pre-blast survey and blasting plan should be prepared prior to performing any blasting activities.

7.3 Excavation Characteristics and Rippability

The excavatability of rock depends on the geotechnical properties of the material, on the method of excavation and on the type and size of excavating equipment used. In mechanical excavation the cutting parts of the equipment must be forced into discontinuities in the rock mass or into the fabric of weak rock. It is generally accepted that the discontinuity (fracture) spacing and the strength of the rock are particularly important properties and general ripping depths can be significantly increased when the rock is bedded or foliated, or highly fractured. The nature of the rock and inherent planes of weakness therefore play an important part in rock excavation assessment.

A rippability assessment for the rock encountered during the geotechnical investigation has been carried out using Weaver's Rock Classification System for

Rippability. Based on Bieniawki's Geomechanics Classification of Rock Masses, this rippability classification system allocates a weighted value to each input parameter. The total rating is given a rippability assessment. Tables 4 and 5 below, show the ratings obtained after the assessment of each grade of weathered rock encountered on site.

TABLE 4. RIPPABILITY RATING

Parameters	Fresh Rock		Slightly Weathered	
	Assessed Classification	Rating	Assessed Classification	Rating
Est. Seismic Velocity	>2150	26	2150-1850	24
Rock Hardness	Very Hard Rock	5	Hard Rock	3
Rock Weathering	Unweathered	9	Slightly	7
Defect Spacing (mm)	300-50	10	300-50	10
Defect Continuity	Slightly Continuous	5	Slightly Continuous	5
Defect Gouge	Ave. <5mm	3	Ave. <5mm	3
Strike and Dip Orientation	Unfavourable	13	Unfavourable	13
Total Rating		71		65

TABLE 5. RIPPABILITY ASSESSMENT

Total Rating	Weaver's Classification (modified)		AS2868-1986
	Tractor Selection	Power (kw)	Class
< 25	D7	135	105 C
25 - 50	D7/D8	135 – 200	105 C – 150 C
50 – 70	D8/D9	200 – 290	150 C – 200 C
70 - 90	D11/ Hydraulic Breaking plus D9	290 – 575	200 C – 500 C
90 - 100	Blasting		

Notes:
Weaver J. M. (1975), Geological Factor Significant in the Assessment of Rippability, Die Siviele Ingenieur (South Africa)
AS 2868 – 1986. "Classification of machinery for earthmoving, construction, surface mining and agricultural purposes"

Weaver noted that for values above 75 should be considered unrippable without using pre-blasting or other techniques.

Excavation of the slightly weathered rock can be carried out by bull dozers (D8/D9) in bulk excavations and excavators with ripping tynes in confined excavations. Excavation of the fresh rock will need to be carried out with a large bull dozer (D11) or with a combination of hydraulic fracturing plus ripping with a D9 dozer.

Given the limited investigation and high strength of the rock it is suggested that an allowance should be budgeted for rock breaking works in addition to minor blasting. Consideration could be given to lightly blasting of the rock mass to loosen it and to facilitate excavation.

7.4 Bearing Capacity

Bearing capacity has been assessed on an understanding that the proposed footings will be founded approximately 1.0m below the ground surface into rock and will be a minimum of 15m wide. Based on this and an assessment of the unconfined compressive strength, fracture spacing and allowable defect range, an allowable bearing capacity of 3.0 MPa may be adopted.

The quoted bearing capacity allows for <1% settlement of the minimum footing dimension.

It is recommended that upon completion of a footing design the bearing capacity is reviewed to ensure the design conforms to our assumptions.

As subsoil conditions may vary from location to location, a qualified geotechnical engineer shall inspect the foundation prior to pouring of concrete. The geotechnical engineer should confirm that the allowable bearing pressures have been obtained. The foundation should be cleaned and free of loose caved in material prior to pouring the concrete.

7.5 Sliding Stability

A coefficient of friction value of 0.58 may be used for preliminary foundation design to evaluate sliding stability of the turbine foundations for concrete footings founded on the underlying bedrock.

7.6 Site Classification

Site classification in accordance with AS2870 has may have little relevance if the footings are to be founded on rock. However is provided to assist with footing designs for structures which may be founded on the soil or at shallow depths.

The Atterberg Limits tests indicate the Sandy SILT (ML) is slightly reactive to changes in moisture content with an estimated predicted ground surface movement (y_s) within the **Class S** category (0 to 20mm) for footings designed in accordance with Australian Standard 2870 'Residential Slabs and Footings – Construction'. It is recommended that any high level footing and slab systems be designed to accommodate the anticipated ground surface movement and the designers should satisfy themselves that the use of AS2870 is applicable for the proposed design.

The ground surface movement (y_s) estimated in this report does not take into account the effects of future trees planted or removed.

7.7 Excavation Conditions, Batter Angles & Dewatering

It is anticipated that excavations will consist of the following:

- Trenching – Upper level footings and services.

Excavation through the sandy silty soils is expected to be readily undertaken using conventional earthmoving equipment (i.e. backhoe or excavator).

Table 6 presents short and long term recommended maximum batter angles for excavations through the soil profile.

TABLE 6: BATTER ANGLES

Material	Short Term	Long Term
Controlled Fill	1H:1V	2H:1V
Sandy SILT	1.5H:1V	2H:1V

These values apply to dry slopes and batter heights of up to 3.5 metres in soil above the water table. Surcharge loads should not be placed near the crest of batter slopes as they may initiate slope failure. Flatter batter slopes would be required where seepage is encountered and further geotechnical advice should be requested if this occurs. It is essential that batters be suitably protected from erosion and scour by the establishment of ground cover and shrubs, installation of surface drains, etc. Runoff should not be allowed to discharge directly across the batters.

Groundwater was not encountered during the investigations but if required, sump and pump dewatering methods are expected to be suitable only when catering for minor seepage through the less permeable silty clay / clay soils.

7.8 Soil Resistivity

Soil resistivity testing was completed at the borehole locations using a FLUKE 1625 Earth Ground Tester. Testing was carried out to determine the soils resistive properties for use in the design of electrical earthing systems associated with the proposed wind turbines. Testing was carried out in two (2) directions perpendicular to each other in the vicinity of the boreholes. At the time of the investigation the ground was saturated at all of the tested locations. The depth which the earth stakes were placed into the ground was limited due to the shallow soil profile at many of the locations.

Soil resistivity was been calculated using the Wenner Method.

Table 7 presents the results of the field testing.

TABLE 7. SOIL RESISTIVITY RESULTS

Borehole	a Electrode Spacing (m)	R_a Wenner Resistance (Ω)	Strike	ρ_a Apparent Soil Resistance (Ωm)
BH1	0.8	258.3	East-West	1298
BH1	0.8	244.9	North-South	1231
BH2	0.8	198.1	East-West	995
BH2	0.8	205.5	North-South	1032
BH3	0.8	181.9	East-West	914
BH3	0.8	176.0	North-South	884
BH4	0.8	65	East-West	326
BH4	0.8	63.9	North-South	321
BH5	0.8	187.3	East-West	941
BH5	0.8	163.2	North-South	820
BH6	0.8	168	East-West	844
BH6	0.8	179.9	North-South	904
BH7	0.8	283.3	East-West	1424
BH7	0.8	252	North-South	1266
BH8	0.8	143.8	East-West	722
BH8	0.8	118	North-South	593
BH9	0.8	269	East-West	1355
BH9	0.8	268	North-South	1448
BH10	0.8	304	East-West	1528

BH10	0.8	295	North-South	1485

8.0 RECOMMENDATIONS

The current study presents an initial appraisal of likely conditions across the Mount Emerald wind farm site. Access at this relatively early stage in the project has been limited, to the extent that a fully representative sample of site conditions may not have been obtained. Further subsurface investigation and analysis may be necessary.

It is recommended that an ETS geotechnical engineer inspect all footing excavations to confirm the subsurface conditions.

9.0 LIMITATIONS

We have prepared this report for the use of **RATCH AUSTRALIA CORPORATION** for design purposes in accordance with generally accepted geotechnical engineering practices. No other warranty, expressed or implied, is made as to the professional advice included in this report. This report has not been prepared for use by parties other than **RATCH AUSTRALIA CORPORATION** or their design consultants, i.e. Architect & Civil/Structural Engineers. It may not contain sufficient information for purposes of other parties or for other uses. Your attention is drawn to the document - "Understand the Limitations of Your Geotechnical Report", which is included in Appendix D of this report. This document has been prepared to advise you of what your realistic expectations of this report should be, and to present you with recommendations on how to minimise the risks associated with the ground works for this project. The document is not intended to reduce the level of responsibility accepted by ETS, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.