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**WINDFARM AT DEREENACRINNIG WEST,
DRUMOLEAGUE, CO. CORK**

**SITE INVESTIGATION CONTRACT
GEOTECHNICAL REPORT
NO. P16177**

Client: Mr. George O'Mahony

Engineer: Jennings O'Donovan &
Partners

Consulting Engineers,

Finisklin,

Sligo,

Ireland.



21737-26/10/2021-EIA-Appendix J Site Investigations



REPORT CONTROL SHEET

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Employer's Representative	Jennings O'Donovan & Partners Consulting Engineers					
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1 INTRODUCTION

1.1 SCOPE OF WORKS

In November 2016, Jennings O'Donovan & Partners, Consulting Engineers acting as the Employer's Representative on behalf of their Client, Mr. G O'Mahony commissioned Priority Geotechnical (PGL), to carry out a site investigation contract for the proposed windfarm development at Dereenacrinnig West, Drumoleague, Co. Cork. The windfarm development comprised of seven (7) turbines, 3km of associated access roadways, upgrade of 1.3km of existing roadway and electrical sub-station infrastructure.

The purpose of the investigation is to provide geotechnical information for the provision and design of the windfarm development and associated works.

The scope of the ground investigation, which was specified by Mott MacDonald, as Tendered (initially) comprised of the following:

- 32Nr. Trial pits excavations;
- 7Nr. Rotary core boreholes;
- 8Nr. Dynamic probes;
- Standpipe well installations;
- All associated sampling;
- *In-situ* testing, standard penetration testing and California bearing ratio (CBR);
- Geophysical investigation, MASW and electrical resistivity;
- Laboratory testing of soil and rock samples;
- Factual and interpretative reporting.

The final works as completed are detailed in Section 3.2 of this factual report. The fieldworks were carried out between 22nd November and 22nd December, 2016.

This investigation was carried out in accordance with the contract specification: Specification and Related Documents for Ground Investigation in Ireland (Engineers Ireland, October 2006), Eurocode 7- Geotechnical Design Part 2, ground investigation and testing (BS EN 1997-2: 2007) and the relevant British Standards (BS 5930 (1999) Code of Practice for Site Investigation +A2:2010 and BS 1377, Method of Tests for Soil for Civil Engineering Purposes, *in situ* Tests Parts 1 to 9).

1.2 REPORTING

This geotechnical report (ref: P16177_RP_F01) presents the factual records of the fieldwork with respect to the site investigation works contract for the proposed windfarm development at Dereenacrinnig West, Drumoleague, Co. Cork and the geotechnical interpretation of same.

A non-intrusive geophysical survey was undertaken by PGL. A report titled "Dereenacrinnig Wind Farm, Geophysical Investigation" (ref: PGL P16177_GP_Rp_F01, March, 2017) is presented in **APPENDIX B** of this combined factual and interpretative report.

No responsibility can be held by PGL for ground conditions between exploratory locations. The exploratory logs provide for ground profiles and configuration of strata relevant to the investigation depths achieved during the fieldworks. Caution shall be taken when extrapolating between such exploratory locations. No liability is accepted for ground conditions extraneous to the exploratory locations. The geotechnical interpretation may be subject to change where further information becomes available.

No account has been taken of potential subsidence or ground movement due to mineral extraction, mining works or karstification below or in proximity to the site, unless specifically addressed.

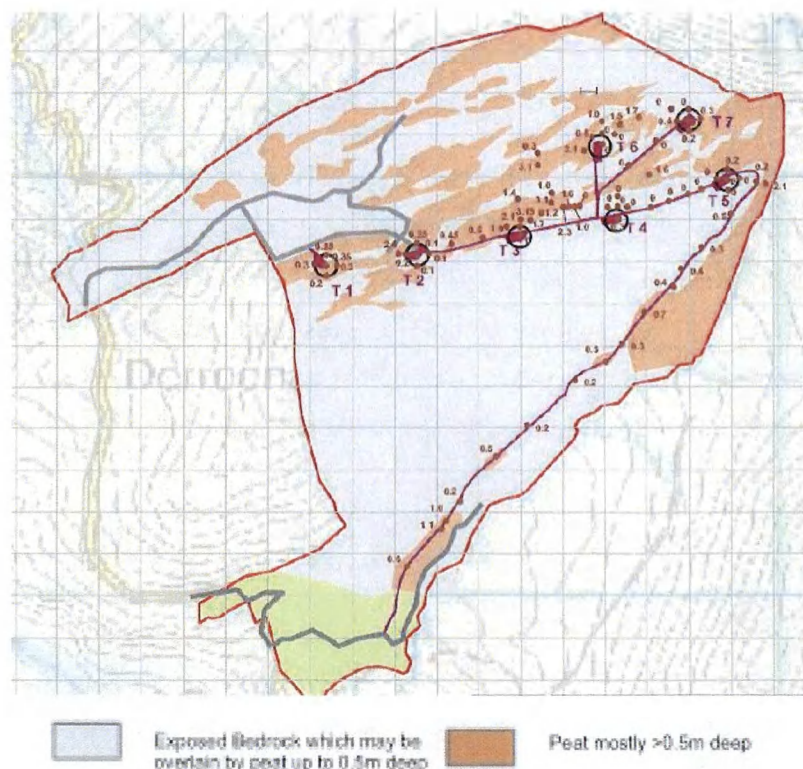
This report has been prepared for the Employer and their Representative as outline, herein. The information should not be used without their prior written permission. PGL accepts no responsibility or liability for this document being used other than for the purposes for which it was intended.

1.3 SITE LOCATION & DESCRIPTION

The up-land site approx' 1km² was north of Castledonovan in Co. Cork, at Dereenacrinning West, at elevations 200mOD to 400mOD Malin.

1.4 PUBLISHED GEOLOGY

The published geology maps; Geological Society of Ireland, GSI, Sheet 24 1:100,000 mapping shows the site to be underlain the Gun Point Formation (GP) described as green-grey Sandstones and purple Siltstones. This formation has been observed to be well folded and generally steeply dipping 50° to 60° across the site. The south of the site is underlain by the Castlehaven Formation (CE), described as purple Mudstone and Siltstone. The very north of the site is underlain by the Toe Head Formation (TH) described as cross-bedded sandstones and purple Siltstones with some Mudstones. There are two published bedrock faults within the site area. Both faults run in a generally northeast-southwest direction, one passing under the northern boundary of the site and one passing close to the southern boundary. The Teagasc mapping on the GSI website shows this area of the site to be underlain by blanket Peat, glacial tills derived from Devonian Sandstones and shallow bedrock. Peat deposits (H3- H7) were measured across the site by Others (2010) and found to be between 0.1m to 3.15m deep.



2 FIELDWORK

2.1 GENERAL

The fieldwork was carried out in general accordance with British Standards (BS 5930 (1999) Code of Practice for Site Investigation +A2:2010 and BS 1377, Method of Tests for Soil for Civil Engineering Purposes, *in situ* Tests Parts 1 to 9). Details of the equipment and plant used are presented below.

Operation	Equipment	Nominal diameter, mm	Flush	Comments
Rotary Coring and Open Hole Drilling	Delta Base 520 6t tracked rig	Symmetrix 131mm diameter open hole 76mm diameter core	Compressed Air mist	Standard Penetration Test, N values obtained in overburden. Visual observations of ground and groundwater conditions. Installation of standpipe monitoring wells.
Trial pit excavations	12t tracked excavator	N/A	N/A	Visual observations of ground and groundwater conditions. Bulk disturbed sampling.
Dynamic probe	TRL Competitor DP(H)	8kg/ 575mm drop ht. 50kg/ 500mm	N/A	Pen rate mm/blow recorded & CBR estimated. N _{100 H} blows/100mm

The exploratory locations were selected by JO'D and set out subject to work space restrictions and available access. The 'as constructed' exploratory locations were subsequently surveyed using Trimble V8 GPS equipment to the Ordinance Survey, Irish Transverse Mercator (ITM) system of co-ordinates and elevations to Malin Head datum. These locations are shown on the Exploration Location Plans (dwg. No.: P16177-SI-0A, P16177-SI-01 to P16177-SI-06) presented in **APPENDIX D** of this report.

2.2 EXPLORATORY HOLES

The exploratory holes as completed during the ground investigation are listed in the following table(s):

SUMMARY OF EXPLORATORY HOLES

Type	Quantity, Nr.	Depth Range, m bgl	Comments
Trial Pit Excavations	32	0.55 – 3.20	TP01A ¹ , TP02A, TP03A, TP04A, TP05A, TP06A and TP07A. TP01, TP02, TP03, TP04, TP05, TP06, TP07, TP08, TP09, TP10, TP11, TP12, TP13, TP14, TP15, TP16, TP17, TP18, TP19, TP20, TP21 and TP22. TPS1, TPS2 and TPS3.
Rotary Boreholes	4	6.70– 7.10	BHT01, BHT02, BHT03 and BHT04 ² .
Dynamic probes	8	0.60 – 3.30	DP01, DP02, DP03, DP04, DP05, DP06, DP07 and DP08 (existing road from Castledonovan).

2.3 SAMPLING

Fifty six (56) bulk disturbed samples (B), forty nine (49) small disturbed samples (D), and 21.25lin.m rotary core were recovered from the exploratory holes in accordance with Geotechnical Investigation and Sampling – Sampling Methods and Groundwater Measurements (EN ISO 22475-1:2006).

¹ Annotation A referenced turbine locations; TP01A = T01 (BHT01).

² Rotary boreholes at turbines T05 – T07 were removed from the scope of works. Trial pits TP05A, TP06A and TP07A were excavated to bedrock. Details of bedrock is also provided for in the geophysical survey report, **APPENDIX B**.

2.4 GROUNDWATER MONITORING

Groundwater was recorded when encountered/ observed during boring and trial pit excavations over a period of 20 minutes, noting any changes that may occur. Groundwater levels were also monitored at start and end of drilling shifts. Groundwater is presented in a table in Section 5 of this factual report.

It should be noted that the normal rate of boring may not permit the recording of equilibrium groundwater levels for any one groundwater water strike where casing may exclude low volume flows as the borehole progresses. Groundwater conditions observed in the borings or pits are those appertaining to the period of the investigation. Groundwater levels may be subject to diurnal, seasonal and climatic variations and can also be affected by drainage conditions, tidal variations etc. The groundwater regime should be assessed from standpipe well installations, where available.



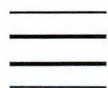
Arisings backfill to borehole



BENTONITE grout backfill to rotary boreholes/ installation



GRAVEL backfill to installation/ rotary borehole



uPVC slotted pipe

2.5 IN SITU TESTING

Standard Penetration Tests, N values, were typically carried out in the boreholes using the 60° solid cone (CPT) in place of the standard split barrel sampler. The Standard Penetration Test was carried out in accordance with Geotechnical Investigation and Testing, Part 3 Standard penetration test, BS EN ISO 22476-3:2005+A1:2011. The data was presented on the relevant logs in **APPENDIX A**.

The geophysical survey comprised of Resistivity Soundings, Seismic Refraction Profiling and Multi-Channel Analysis of Surface Wave (MASW) at 7 proposed turbine locations as well as a proposed substation location. The survey fieldwork was carried out by PGL between the 12th and 14th December, 2016. A separate report has been produced and is presented in **APPENDIX B** of this report.

TRL dynamic probes (8kg drop weight, 575mm drop height) were carried out in pit excavations to establish *in situ* California bearing ratio, CBR to refusal depths between 0.18m bgl to 2.30m bgl. The data from the testing was presented in **APPENDIX A** with the relevant trial pit records.

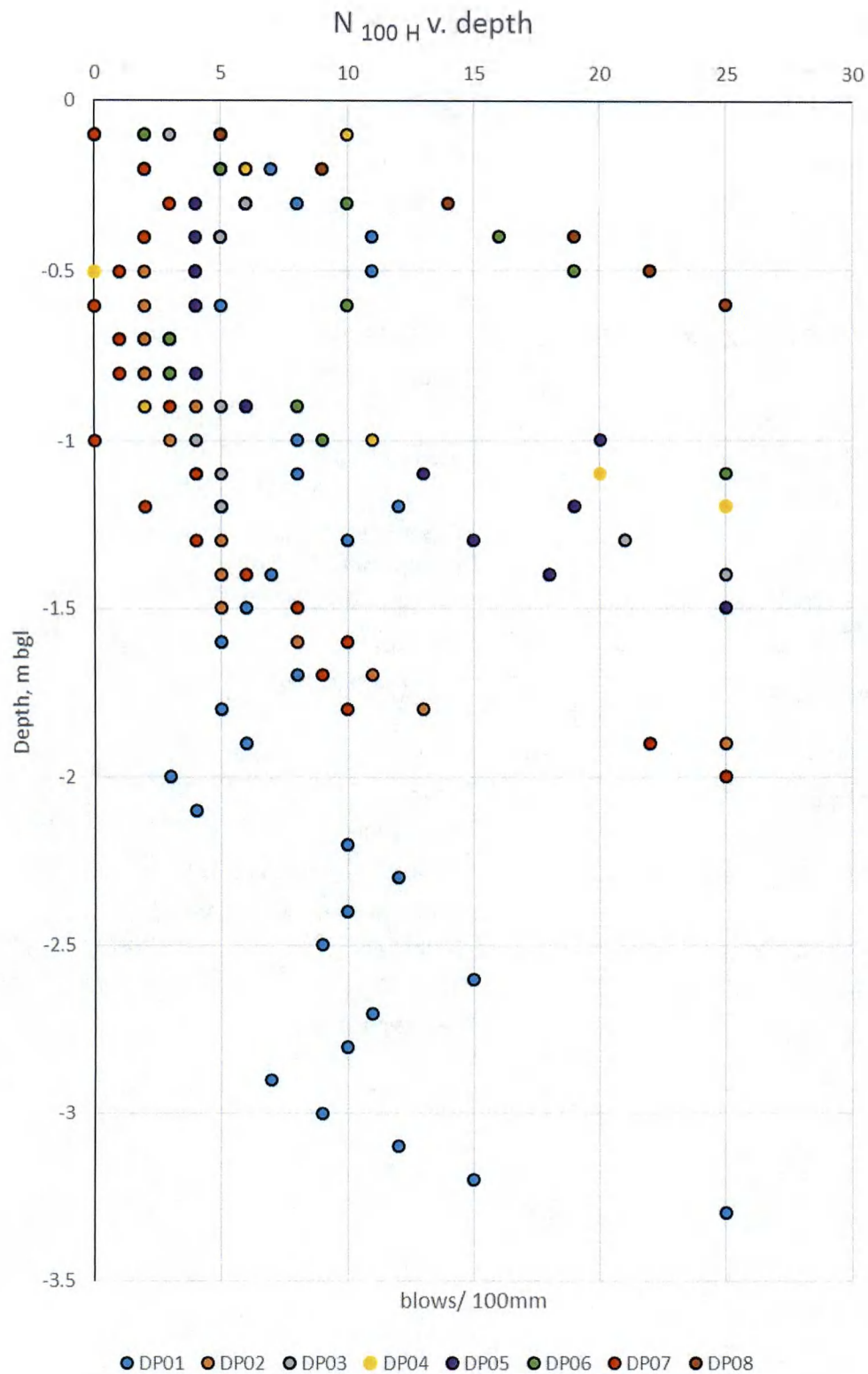
PGL's Competitor dynamic probing rig was to undertake dynamic probing (DP(H); heavy, 50kg drop weight, 500mm drop height) in general accordance with Geotechnical Investigation and Testing, Part 2, Dynamic probing, BS EN ISO 22476-2:2005. The blows per 100mm (N₁₀₀) were recorded to refusal being 25blows without progress over 100mm. Eight (8) number dynamic probes (DP01, DP02, DP03, DP04, DP05, DP06, DP07 and DP08) were advanced to refusal to depths between 0.6m below existing ground level (bgl) to 3.3m bgl.

SUMMARY OF IN-SITU TESTING

Type	Quantity	Remarks
Standard penetration test, N _{SPT} value	4Nr.	Nspt refusals N presented a numerical value 50; See APPENDIX A
Geophysical survey	840lin.m 2D res 161lin.m Seismic/ MASW	P16177_GP_Rp_F01, <i>March, 2017</i> See APPENDIX B
Dynamic probing (TRL)	32Nr.	DCP01A, DCP02A, DCP03A, DCP04A, DCP05A, DCP06A and DCP07A.

Type	Quantity	Remarks
		DCP01, DCP02, DCP03, DCP04, DCP05, DCP06, DCP07, DCP08, DCP09, DCP10, DCP11, DCP12, DCP13, DCP14, DCP15, DCP16, DCP17, DCP18, DCP19, DCP20, DCP21 and DCP22. DCPS1, DCPS2 and DCPS3. See APPENDIX A
Dynamic probing (DPH)	8Nr.	N ₁₀₀ H See APPENDIX A

The distribution of N_{100} values with depth (m below existing ground level, bgl) is presented below for dynamic probing DP(H). Refusals, $N_{100} = 25$, were plotted. The N_{100} values for the boreholes are presented on probe logs in **APPENDIX A**.



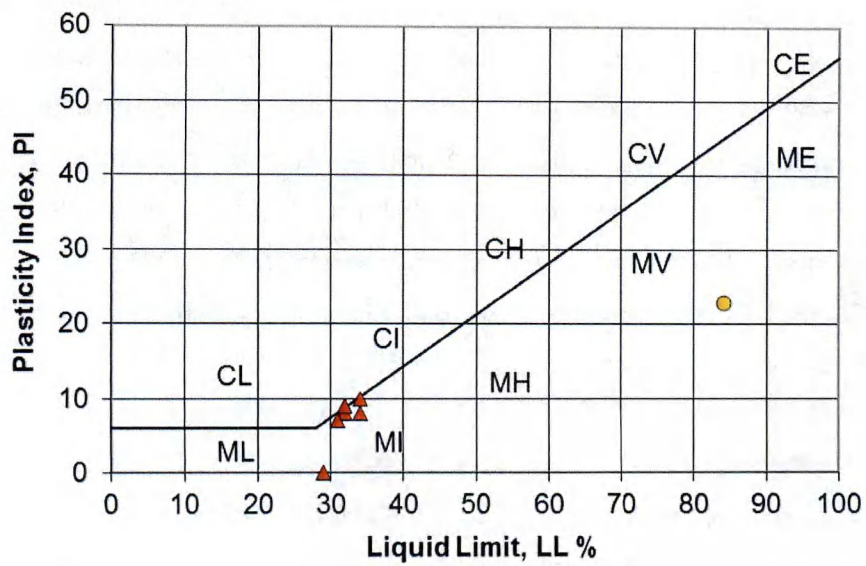
3 LABORATORY TESTING

All samples were transported to Priority Geotechnical's laboratory in Midleton, Co. Cork examined, logged and prepared for scheduled testing. Laboratory testing was proposed by PGL, being revised and approved by Jennings O'Donovan. Testing was carried out by PGL, in accordance with BS1377 (1990), Methods of test for soils for civil engineering purposes and the ISRM suggested methods for rock characterisation, testing and monitoring. Specialist chemical testing was undertaken by Chemtest Ltd. (UK) on behalf of PGL. Specialist rock testing was carried out by GSTL Ltd (UK) on behalf of PGL. The laboratory test results were presented in **APPENDIX C**. A summary of tests undertaken were detailed below.

SUMMARY OF LABORATORY TESTING UNDERTAKEN – SUPERFICIAL DEPOSITS

SOILS		
Type	No.	Remarks
Natural Moisture Content	39	9% to 795%
Atterberg Limits	08	Liquid limit, LL 29% to 187% Plastic limit, PL 23% to 102% and NP non-plastic soils Plasticity index, PI 7 to 85
Particle Size Distribution (grading)	15	See APPENDIX C
pH	03	7.4 to 8.1
SO ₄ water soluble	03	<0.010g/l to 0.013g/l
SO ₄ acid soluble	03	<0.01%
Loss on ignition	04	0.73% to 81%
Proctor compaction (Moisture content/dry density relationship)	01	TP06 0.75m; Optimum moisture content 6% Maximum dry density 2.02Mg/m ³
California Bearing Ratio, CBR	16	0.5% to 2.5%
CBR Moisture content relationship	01	TP06 0.75m See APPENDIX C

Summary of plasticity data



SUMMARY OF LABORATORY TESTING UNDERTAKEN – SOLID GEOLOGY

ROCK		
Type	No.	Remarks
UCS with Young's Modulus and Poisson's Ratio	01	16MPa 55.7GPa 0.215
Point Load Test (Ip_{50})	06	0.7MPa to 4.7MPa
Magnesium sulphate soundness value MSSV,	04	6% to 9%
pH	01	7.4 to 8.7
SO ₄ water soluble	01	<0.010g/l

4 GROUND CONDITIONS

The full details of the ground conditions encountered are provided for on the exploratory records accompanying this report. The records provide descriptions, in accordance with BS 5930 (1999) +A2: 2010 and Eurocode 7, Geotechnical Investigation and Testing, Identification and classification of soils, Part 1, Identification and description (EN ISO 14688-1: 2002),– Identification and Classification of Soil, Part 2: Classification Principles (EN ISO 14688-2:2004) and Identification and Classification of Rock, Part 1: Identification & Description (EN ISO 14689-1:2004) of the materials encountered, in situ testing and details of the samples taken, together with any observations made during the site investigation.

The turbine sites were characterised by dark brown to black PEAT and peaty CLAY 0.22m to 0.6m thick and slightly sandy gravelly SILT with Cobble content deposits 0.7m thick to 1.1m thick overlying bedrock BHTP01 to BHTP04 (T01 – T04). Based on tactile assessment, the Silt was described as soft. Medium strong to strong SILTSTONE was encountered 1.55m bgl (BHT03) to 1.65m bgl (BHT01). Sandstone was found to underlie the Siltstone at BHT01 and BHT04, below 4.5m bgl to 5.05m bgl. Strong SANDSTONE was encountered at 1.55m bgl (BHT02). Peaty CLAY 0.8m thick was encountered at TP05A (T05) overlying 0.7m of firm slightly sandy gravelly SILT with Cobble content to a depth 1.5m bgl. Peaty CLAY 0.17m thick was encountered at TP06A (T06) overlying 0.63m of firm slightly sandy slightly gravelly SILT with Cobble content to a depth 0.8m bgl. PEAT 0.28m thick was encountered at TP07A (T07) overlying 0.92m of firm to stiff slightly sandy gravelly SILT with Cobble content to a depth 1.2m bgl.

The access road was characterised by mixed deposits. Topsoil; slightly sandy slightly gravelly SILT, 180mm to 300mm thick; PEAT 170mm to 600mm thick and peaty CLAY 150mm to 900mm overlay silty sandy GRAVEL (TP01, TP02, TP03 and TPS1) with varied Cobble content deposits 600mm to 1.55m thick and soft becoming firm to stiff, slightly sandy slightly gravelly SILT with varied Cobble content, 650mm to 2.7m thick. The Gravels and Silts were underlain by shallow bedrock 0.75m bgl to 2.3m bgl. Clayey PEAT was encountered 2.0m deep at TP12.

5 GROUNDWATER CONDITIONS

Groundwater was not encountered during rotary drilling. Groundwater was encountered in trial excavations between ground level, 0.00m bgl and 2.00m bgl. Details of the ground water and installations are presented graphically on the relevant exploratory logs within **APPENDIX A** and are summarised below. See also section 3.4 for general details.

No standpipe well, installations were constructed.

SUMMARY OF GROUNDWATER INFORMATION OBTAINED DURING SITE WORKS

Location	Depth water strike, m bgl	Comments
TPS1	2.00	Water strike at 2.0m bgl, trickle flow rate.
TP01	0.80	Water strike at 0.80m bgl. Trickle flow rate.
TP05A	0.30	Trickle flow rate.
TP07	0.10	Water strike at 0.10m bgl. Steady flow rate.
TP09	1.40	Water strike at 1.40m. Trickle flow rate.
TP12	0.10	Water strike at 0.10m bgl. Trickle flowrate.
TP20	1.20	Waterstrike at 1.20m bgl. Trickle flow rate.
TP21	0.00	Water encountered at ground level.

6 GEOTECHNICAL REVIEW

6.1 GROUND MODEL

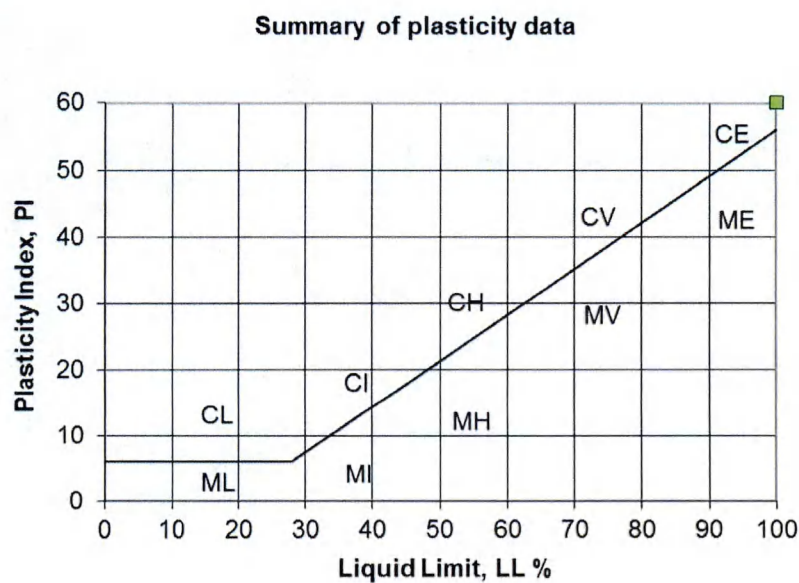
The site was characterised by shallow bedrock overlain by organic (shallow blanket PEAT) and mixed glacial deposits slightly sandy slightly gravelly SILT and silty sandy GRAVEL.

Groundwater was assumed at the boundary of the rock and superficial deposits with perched groundwater within the shallow peat deposits.

6.2 CHARACTERISTIC PROPERTIES

6.2.1 PEAT

The Peat was characterised by extremely high plasticity (liquid limit, LL 187%), high organic content 81% and natural moisture content, w 248% and 765%.



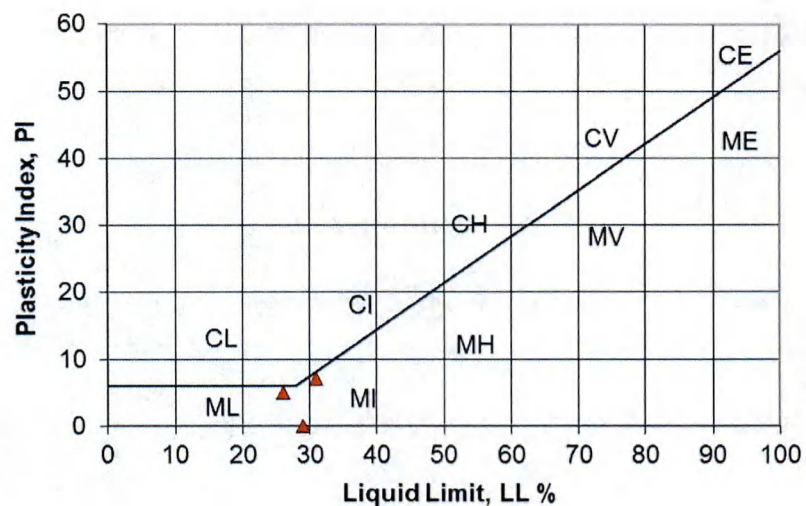
The Peat deposits are expected to be highly compressible, noting Peat was 2m thick at location TP12.

Historical in situ hand vane tests (Others, 2010) indicated undrained shear strengths 16kPa to 34kPa describing very soft to soft deposits.

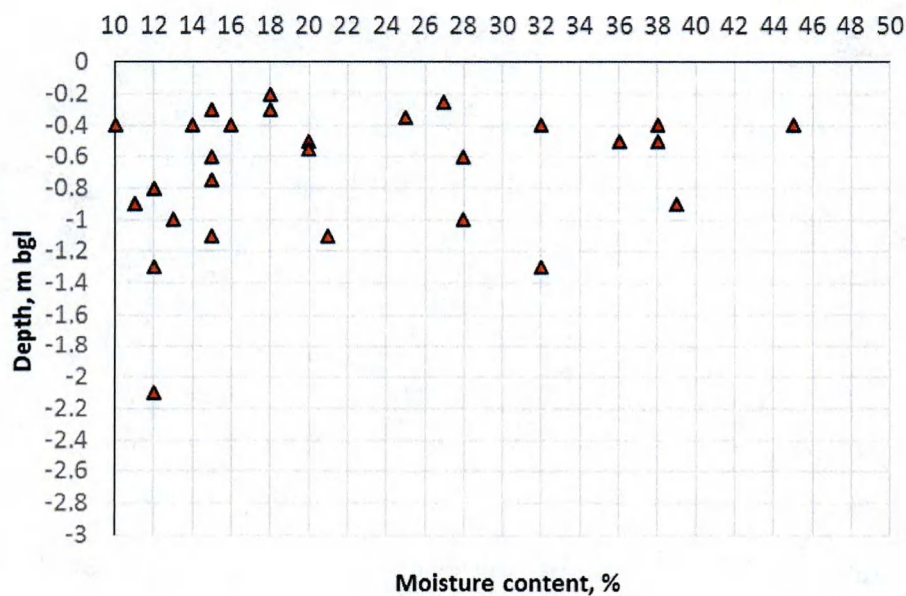
6.2.2 SILT

The Silt was characterised by natural moisture content, w 10% to 45%, low plasticity (ML) and low organic content (<6%). Grading analysis indicated 22% to 42% silt fraction with 29% to 52% gravel fraction, 5% to 34% sand fraction with up to 20% coarse Cobble content. The deposits are described as mixed glacial tills.

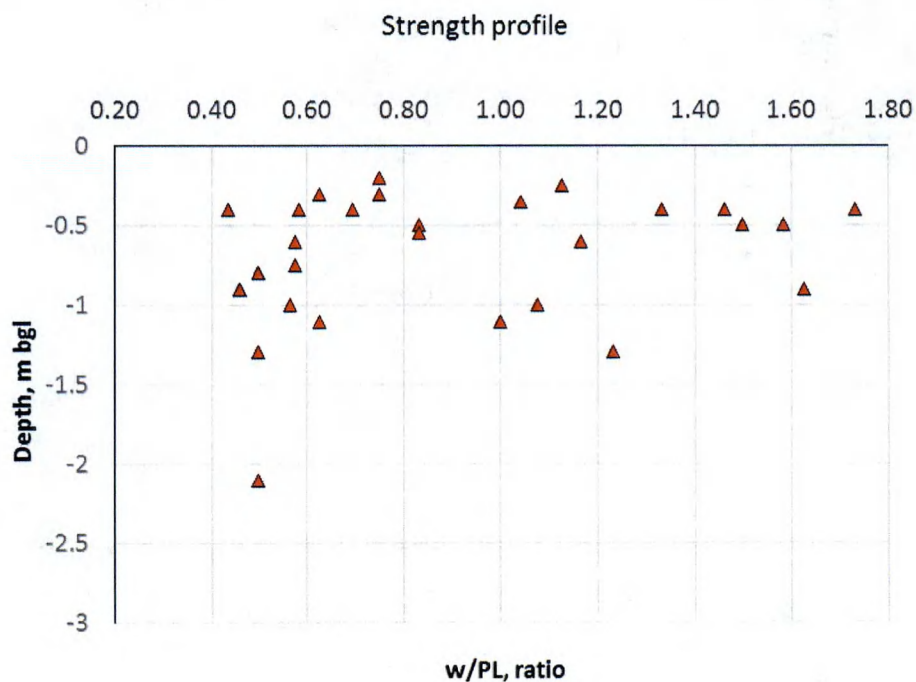
Summary of plasticity data



Moisture content profile - Silt



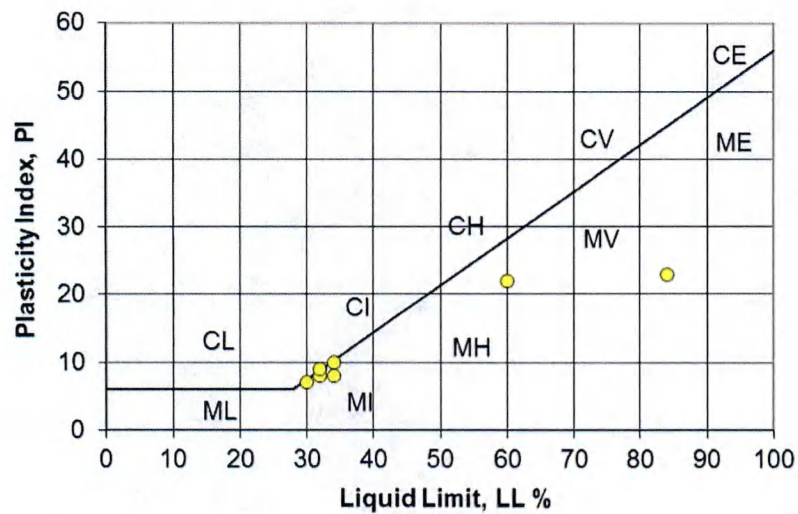
The ratio of natural moisture content to plastic limit indicated soft ($w/PL > 1.2$) to 'firm to stiff' deposits ($w/PL < 1.0$) (C504 Engineering in glacial tills, Figure 5.19). It can be seen that soft deposits were present to depths up to 1.3m bgl at locations; TP04, TP05, TP06, TP07, TP11, TP13, TP15, TP17, TP18, TP19, TP03A and TPS3. Undrained shear strengths are expected to vary between 20kPa and >75kPa ('very soft to soft' to stiff, BS5930; 1999). The soft deposits were typically in the upper 0.5 -1.0 with exceptions of TP04, TP07, TP19 and TPS3 where the soft deposits extended deeper. It is noted the *in situ* tactile assessment indicated firm deposits (40kPa – 75kPa BS5930; 1999). Plasticity data indicated a friction angle, $\phi = 26^\circ$ (C504 figure 5.1).



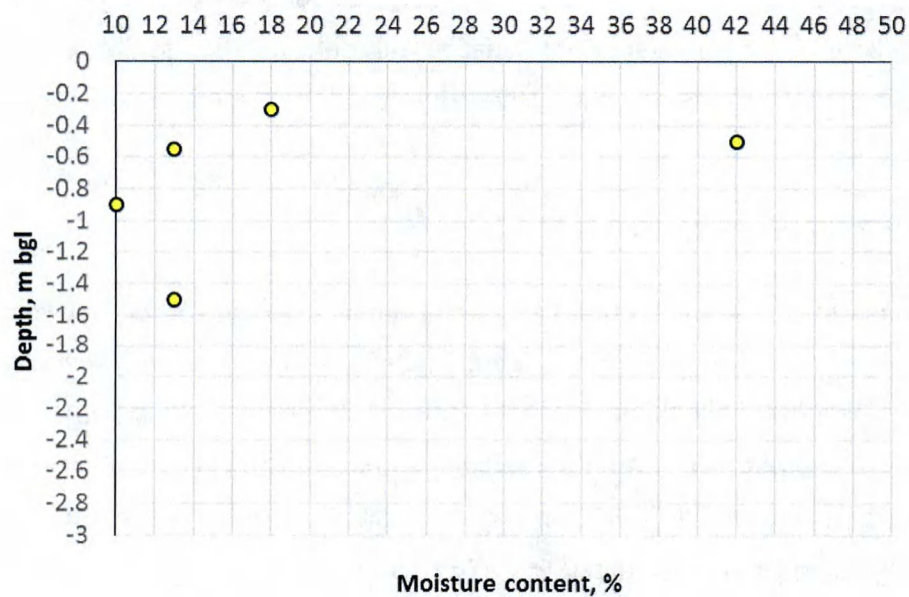
6.2.3 GRAVEL

The Gravel was characterised by natural moisture content, w 10% to 42% and intermediate to very high plasticity (MI – MV). Grading analysis indicated 34% to 66% gravel fraction with 14% to 25% sand fraction, 7% to 27% silt fraction with 4% to 24% coarse Cobble content. The deposits are described as mixed glacial tills.

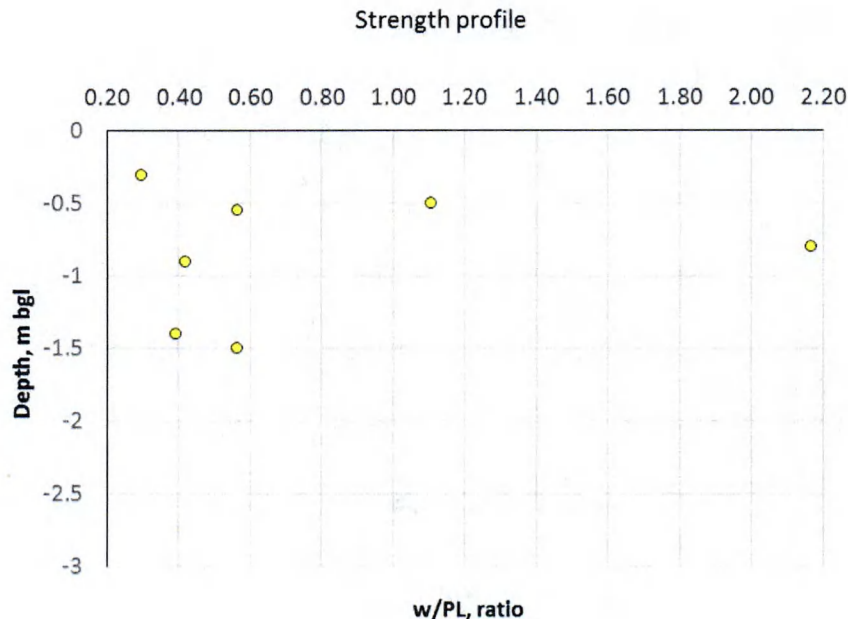
Summary of plasticity data



Moisture content profile - Gravel



The ratio of natural moisture content to plastic limit indicated soft ($w/PL > 1.2$) to 'firm to stiff' deposits ($w/PL < 1.0$). It can be seen that soft deposits were present to depths up to 0.8m bgl at locations; TP08 and TPS3.



No assessment of the relative density of the Gravels was provided. The higher plasticity of the Gravels were attributed to the overlying organic deposits.

The SILT and GRAVEL shall be considered a singular deposit - mixed glacial till with varied granular fractions, having similar characteristics with only variation in the proportions of the varied constituents/ fractions and characterised by varied undrained shear strengths.

6.2.4 SANDSTONE and SILTSTONE

The rock mass characterization has been established using the Rock Quality Designation, (RQD, Deere, 1964), Rock Mass Rating (RMR) using the Geo-mechanics System (Bieniawski, 1989) and Geologic Strength Index (GSI, Hoek and Brown 1997, 2002). A review of the rock properties, strength (medium strong to strong I_{P50} , 1.1MPa – 4.7MPa, UCS 16MPa), fracture spacing (NI– 400mm) and condition (slightly weathered no infill), Rock Quality Designation (RQD non-intact/ 9% - 78%) and groundwater (assumed 'dry' within the zone of influence of the expected foundations) was undertaken. The rock mass rating, RMR range was 44 - 64.

The Siltstone and Sandstone bedrock was classified as Class III-II fair to good rock with unit weight 2.71Mg/m^3 . A Poisson's ratio, ν of 0.15 - 0.20 can be expected and was measured as $\nu = 0.215$. The intact Young's Modulus, E was 55.7GPa. Allowing for the non-intact nature of the rock mass this was reduced by a factor of 0.15 yielding a design modulus of 8.4GPa.

The rock characteristic properties were summarized as follows:

	T01		T02		T03				T04	
RMR	44	52	62	64	59	64	54	63	54	57
Class	III		II		III-II		III-II		III	
Cohesion	250kPa		300kPa		300kPa		275kPa		250kPa	
Friction	30°		35°		35°		32°		30°	
Rock	Siltstone		Sandstone		Siltstone		Sandstone		Siltstone	
E^3 , MPa	5093	9530	18430	20761	15303	20761	10979	19570	10979	13446
	7079	11220	19953	22387	16788	22387	12589	21135	12589	14962

Note there was a reasonable correlation between the Young's Modulus, E derived from direct measurement (BHT03) the assessment of rock mass rating, RMR and the indirect geophysical measurements (MASW).

6.3 PAVEMENT AND HARDSTANDING CONSTRUCTION

It is recommended to construct pavement and hardstanding from a formation level in weathered bedrock were practical. Where the depth of superficial deposits exceeds 1.0m 600mm capping is recommended. Laboratory test data and moisture content data suggested an adjustment factor of 0.5 for the *in situ* determined CBR values. CBR values of 22% were attributed to coarse particles or transition to the weathered rock mass. It is recommended that plate loading tests, using a 600mm plate, be used to control CBR during the construction phase for both the roadway and also hardstanding associate with crane platforms. The risk associated with pavement failure are high during construction and commissioning phased of windfarm development.

³ $E = 3.5 \times \text{RMR}^{3.75}/1000$ and 10^n where $n = (\text{RMR}-10)/40$ (Boyd, 1993; BS8004)

Natural moisture content data and a single California bearing ratio, CBR/ moisture content relationship, suggested CBR_{2%} at moisture content, $w > 10\%$. Typically the superficial deposits were wet of optimum moisture content (6%/ CBR_{50%}) and so low CBR values were measured. It is not seen as practical to 'dry' the deposits *in situ* and so a design capping thickness of 600mm is proposed within the sandy gravelly SILT and silty sandy GRAVEL deposits with 150mm sub-base (cl.804 or similar) unbound surfacing. A summary assessment of CBR data is presented in **APPENDIX E**.

Where the sub-formation, the underside of capping (subgrade) is on weathered rock, no capping is required and 150mm sub-base (cl.804 or similar) shall be provided subject to a review by the Engineer.

At natural moisture content $< 15\%$, compaction levels of 95% maximum dry density can be achieved. It is recommended that the sub- grade deposits (slightly) sandy (slightly) gravelly SILT, be compacted to increase stiffness. Between moisture content 5% to 19%, CBR $> 20\%$ is expected.

'Reversal road' construction shall be considered where the weathered rock mass was at depths $< 1.0\text{m}$ bgl. Shallow road cuttings are also recommended. A minimum road width at formation of 7m is expected in shallow cutting and through Peat deposits.

Drainage shall be provided a minimum of 450mm below sub-formation.

6.3.1 Existing roadway

Dynamic probing DPH indicated varied strength to depths between 0.6m bgl and 3.3m bgl. Undrained shear strength was between 20kPa and 100kPa indicting soft deposit typically becoming firm to stiff with depth below 1.0m bgl.

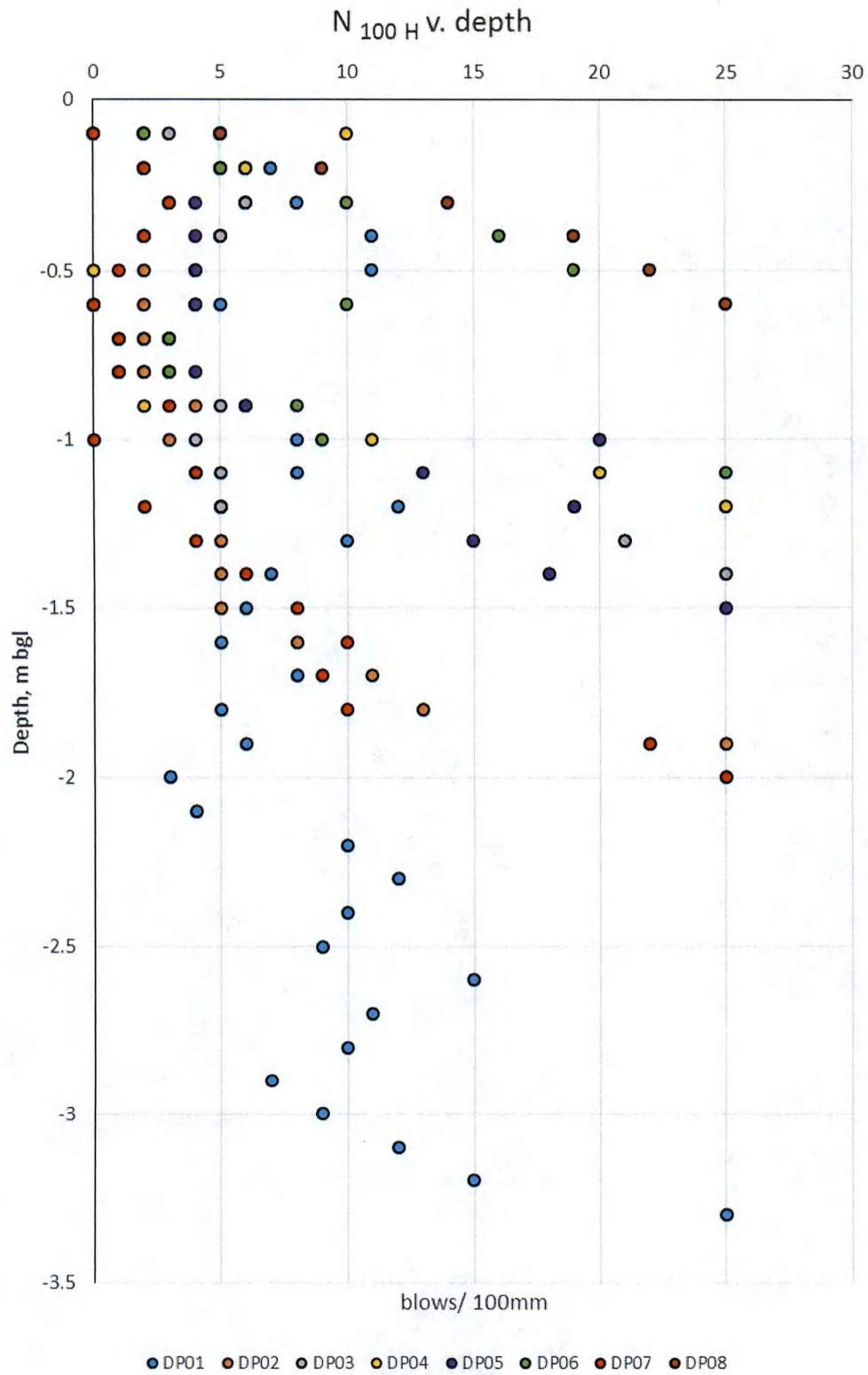
California bearing ratio, CBR was derived based on the relationship;

$$\text{CBR (\%)} = s_u \text{ (kPa) } / 15.$$

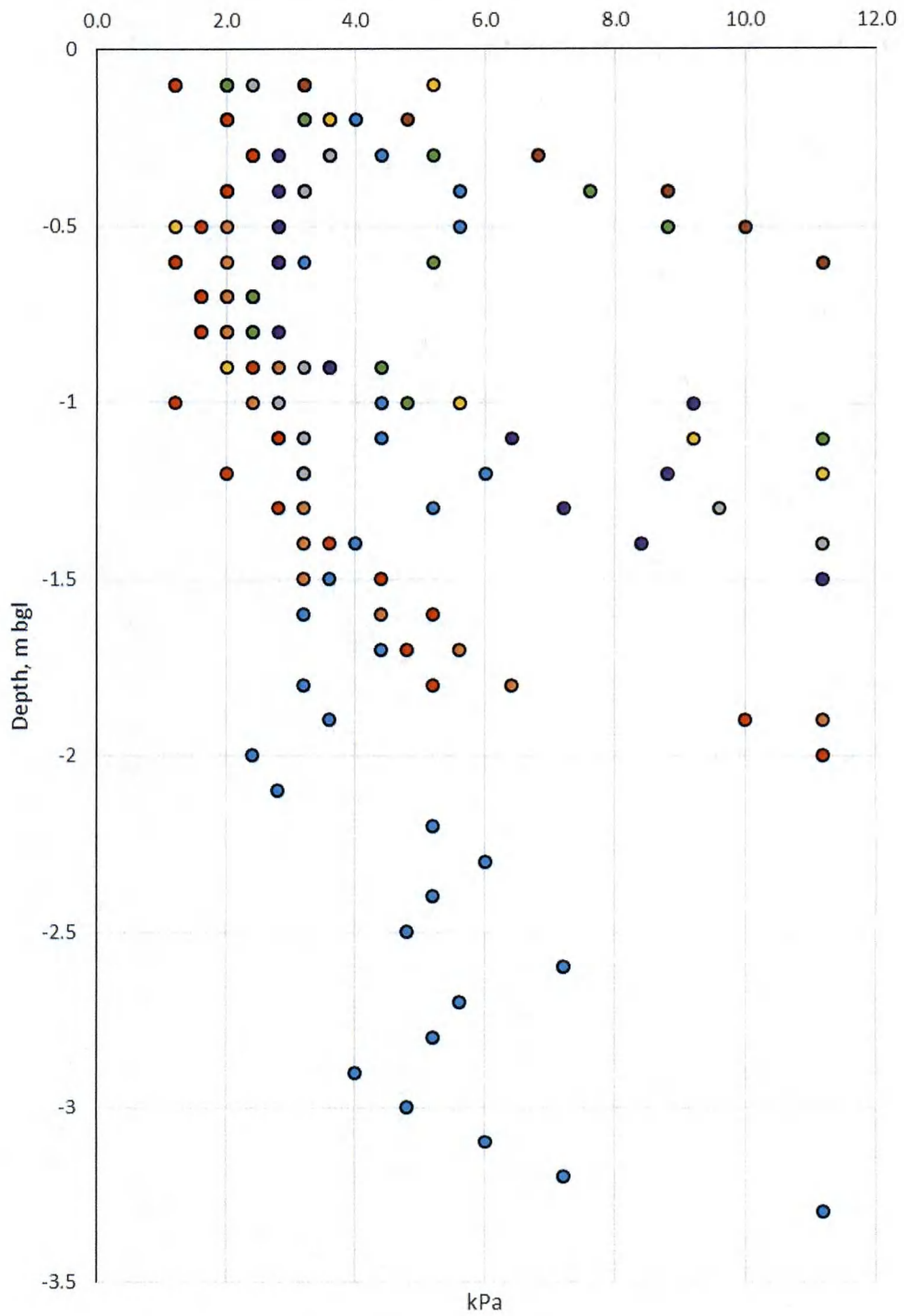
It can be seen that CBR_{2%} is expected to varied depths between 1.0m bgl and 2.0m bgl.

In order to fully assess the existing pavement the full construction detail, sub-base and surfacing thicknesses should be reviewed.

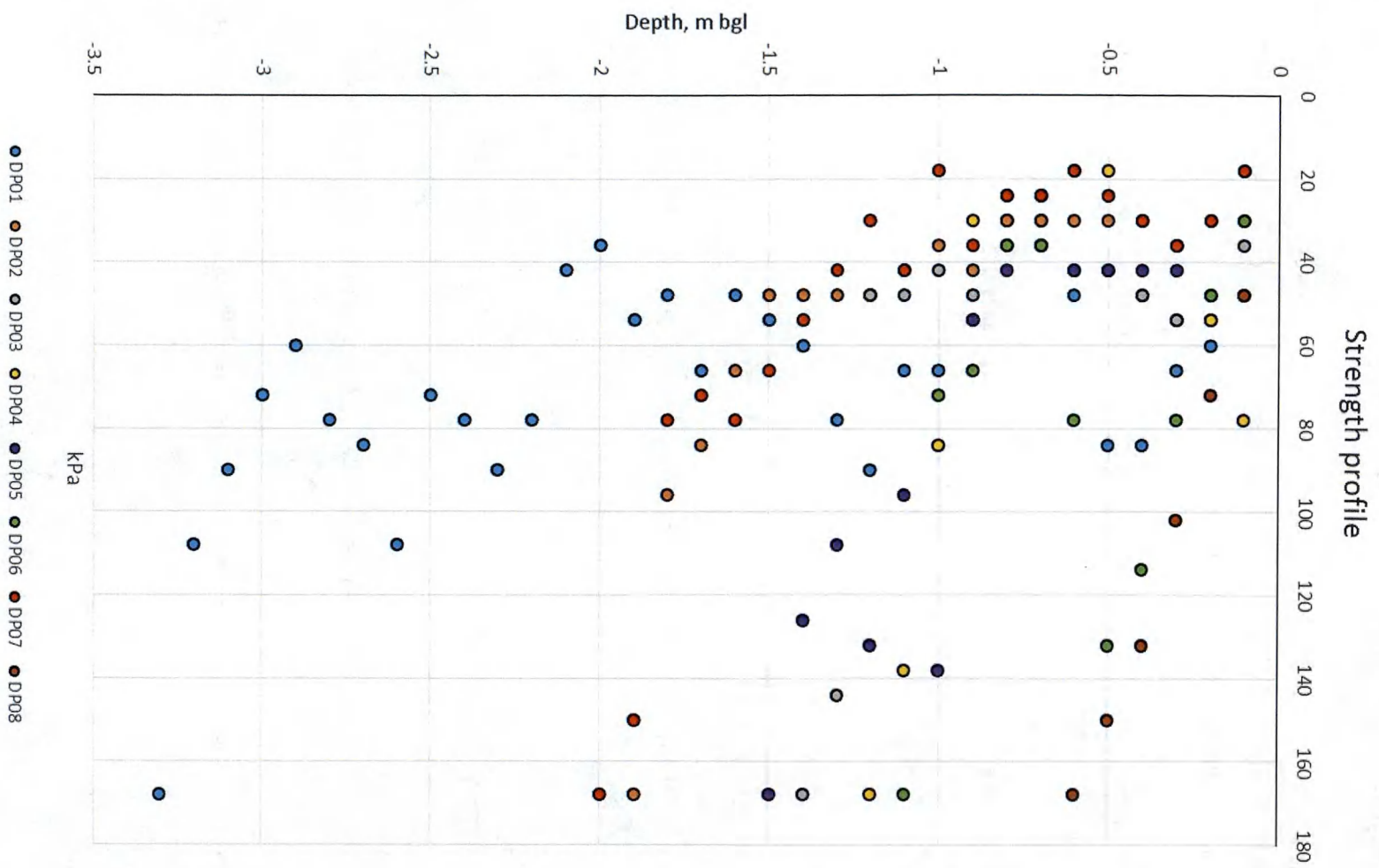
A pavement condition survey and Falling Weight Deflectometer FWD analysis should be undertaken to assess the structural integrity of the existing roadway to take the temporary trafficking associated with turbine deliveries.



Assumed CBR profile



● DP01 ● DP02 ● DP03 ● DP04 ● DP05 ● DP06 ● DP07 ● DP08



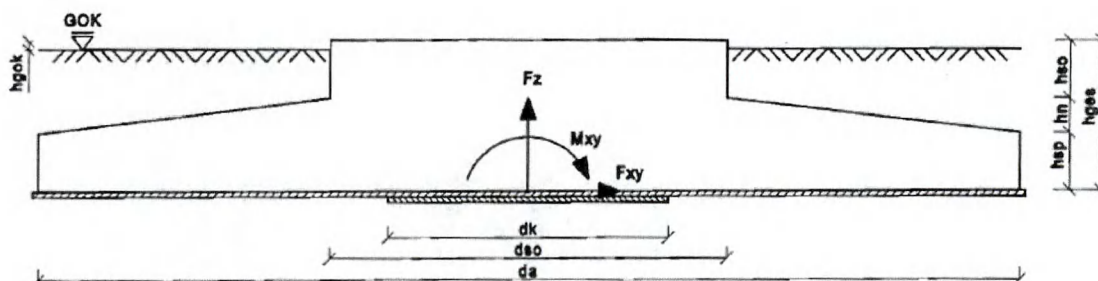
6.4 RE-USE OF AGGREGATE MATERIALS

With a 10% fines value of 102kN to 117kN and magnesium sulphate soundness values, MSSV 6% to 9%, the Sandstone and Siltstone is expected to be suitable for re-use as capping with $50\text{kN} < 10\%FV < 130\text{kN}$ and $\text{MSSV} < 25\%$.

No direct assessment was made of borrow pits and so further assessment is recommended where aggregate is produced on site. Water absorption capacity WAC (limit $< 2\%$) is recommended along with aggregate impact value AIV (limit $< 35\%$, soaked) to assess strength and durability. Flakiness index (FI < 40), grading ($< 6\%$ fines content) and plasticity (Liquid limit $< 21\%$) of aggregate shall also be assessed.

6.5 FOUNDATIONS

Foundations for the proposed turbines are within the range 13.2m to 16.8m outer diameter (d_a) and 1.6m thick (h_{ges});

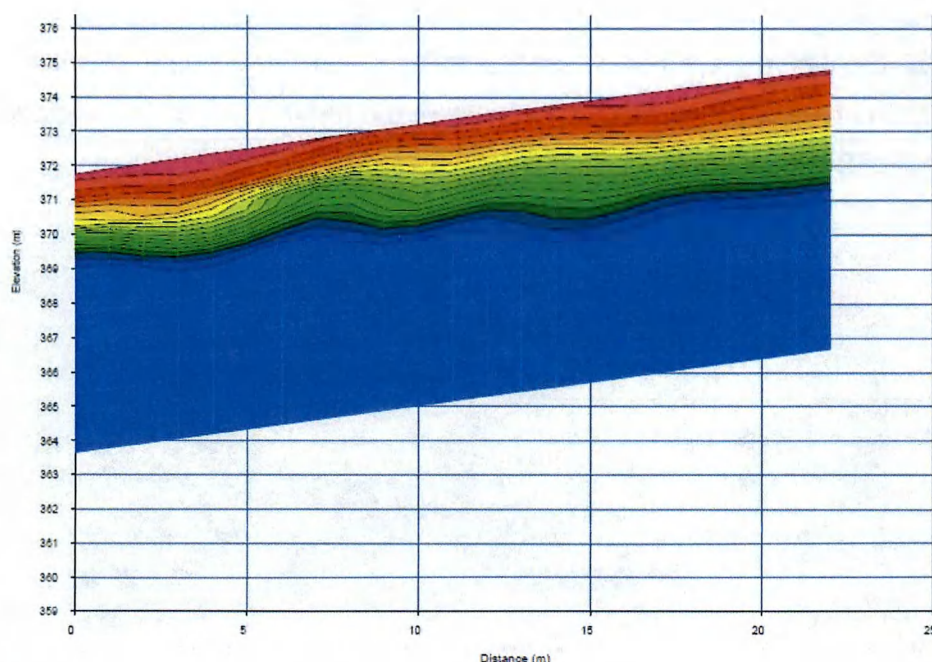


Minimum allowable bearing pressure of 77kPa to 127kPa is required for settlements not exceeding 3mm/m over a period of 20 years.

6.5.1 Turbine sites

Turbines are expected to be founded within the bedrock at least below 1.7m bgl. Based on the core information for the upper 5.5m of bedrock the rock was described as Class 3, weakly cemented Sandstone/ Siltstone ⁴ in accordance with Figure 4. Weak and broken rock of the Code of practice for Foundations, BS8004, 1986. An allowable bearing capacity of 250kPa is provided for settlement up to 0.5% foundation width (Figure 1, BS8004) where the bedrock is non-intact in the upper zone weathered zone.

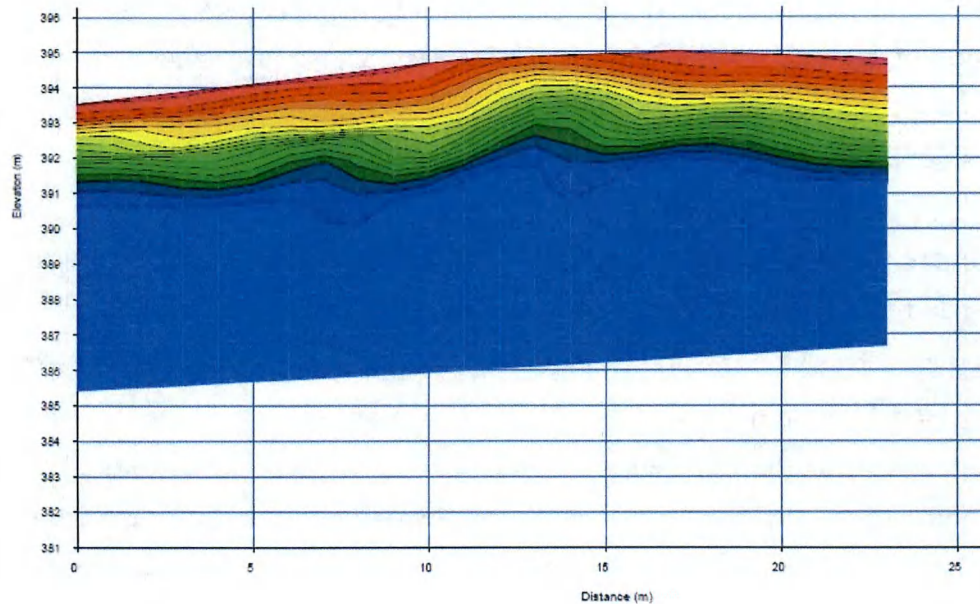
For settlement of the order 3mm/m an allowable bearing pressure of 130kPa to 255kPa is recommended taking a closely spaced, fracture spacing, j 100mm to 400mm and a single unconfined compressive strength, q_c of 16MPa ³. The geophysical survey data indicated an increasing Modulus with depths and so the proposed allowable bearing pressure is likely to be conservative over the full depth of influence of the foundations and where the rock strength is stronger. Shear failure of the foundation is not expected at the level of loading applied where an ultimate bearing pressure of 4MPa is expected. Settlement, particularly differential settlement, will be the limiting factor.



Seismic profile T01

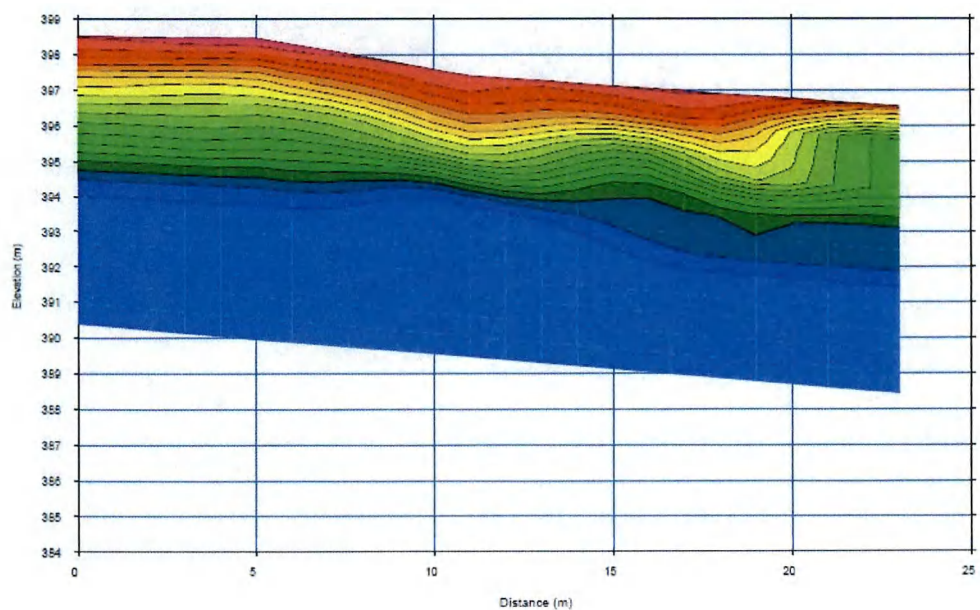
⁴ A UCS of 16MPa was measured conservatively indicating a weak rock in contrast with high point load indices up to 4.7MPa indicating strong rock.

Turbine; T01 foundations are recommended to be constructed below 371mOD within the bedrock. Bases on the rock mass characteristic at the turbine location a lower limit of allowable bearing pressure of 160kPa is recommended.



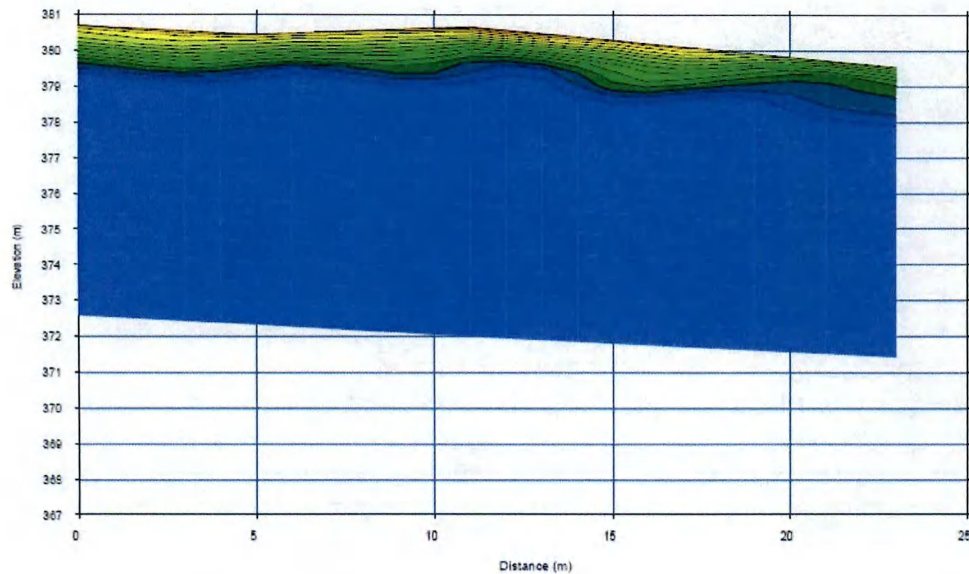
Seismic profile T02

Turbine; T02 foundations are recommended to be constructed below 393mOD within the bedrock. Bases on the rock mass characteristic at the turbine location an allowable bearing pressure of 255kPa is recommended.



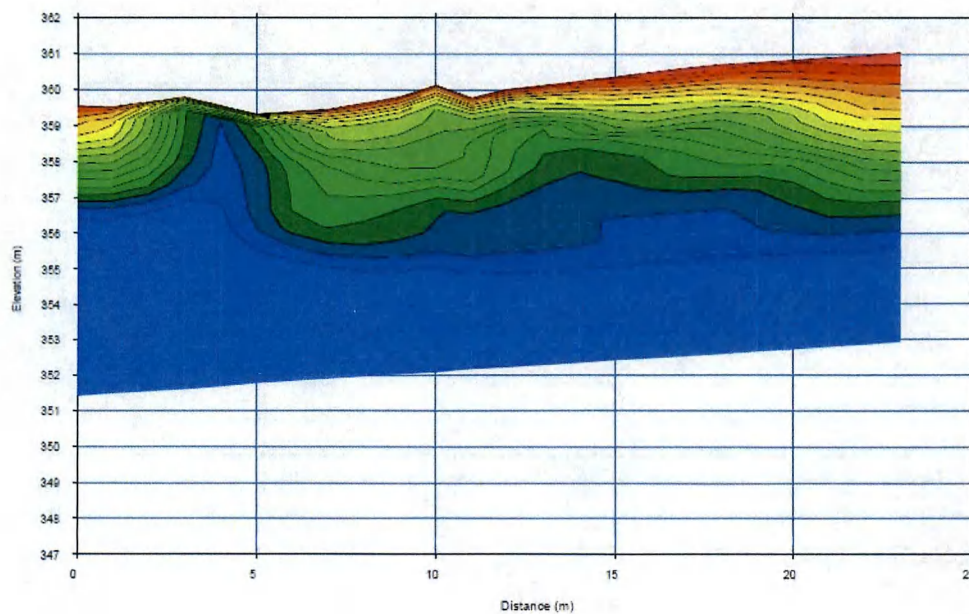
Seismic profile T03

Turbine; T03 foundations are recommended to be constructed below 396mOD within the bedrock. Bases on the rock mass characteristic at the turbine location an allowable bearing pressure of 215kPa is recommended.



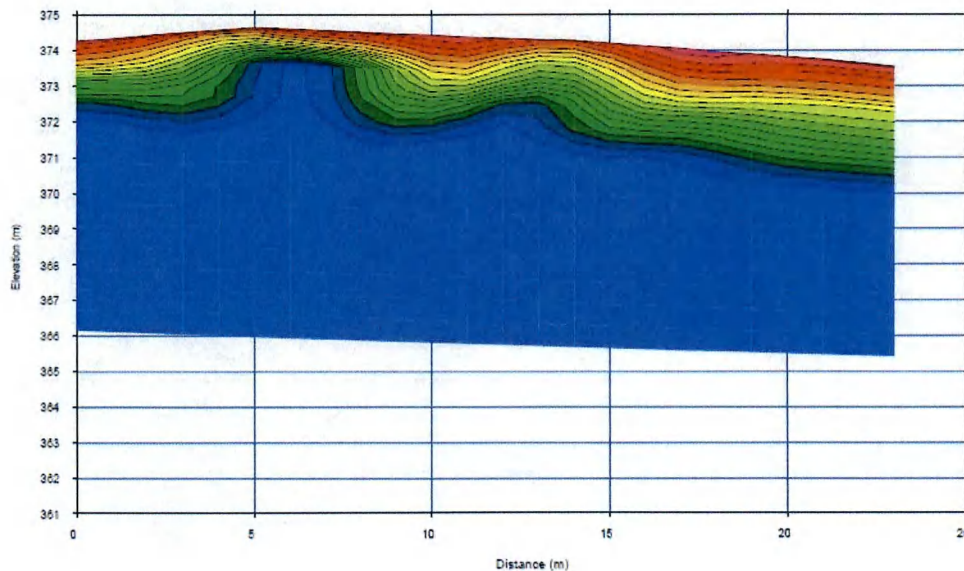
Seismic profile T04

Turbine; T04 foundations are recommended to be constructed below 379.5mOD within the bedrock. Note the rotary borehole appears to be at a location at a higher elevation. Bases on the rock mass characteristic at the turbine location an allowable bearing pressure of 130kPa is recommended.



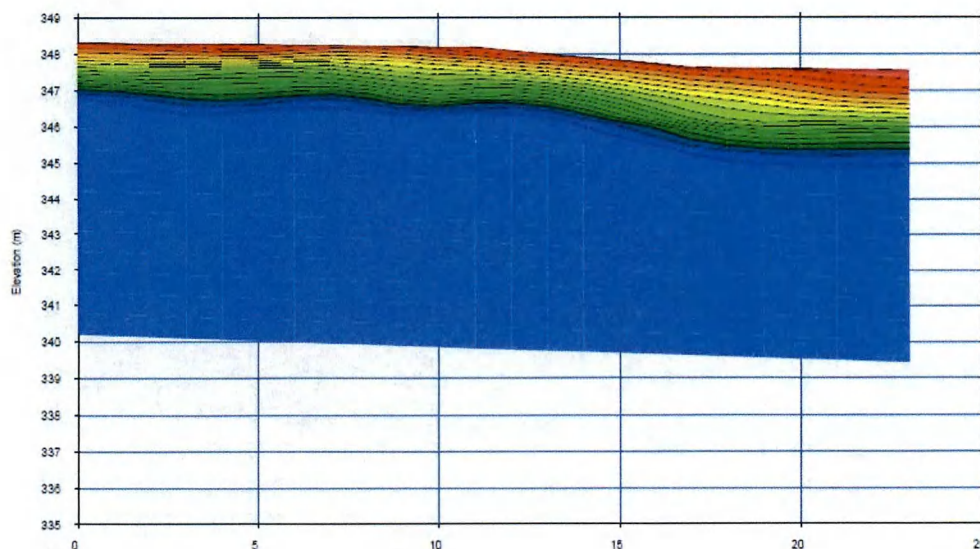
Seismic profile T05

Turbine; T05 foundations are recommended to be constructed below 359mOD within the bedrock. Bases on the rock mass characteristic derived from geophysical investigation at the turbine location an allowable bearing pressure of 130kPa is recommended.



Seismic profile T06

Turbine; T06 foundations are recommended to be constructed below 373mOD within the bedrock. Bases on the rock mass characteristic derived from geophysical investigation at the turbine location an allowable bearing pressure of 195kPa is recommended.



Seismic profile T07

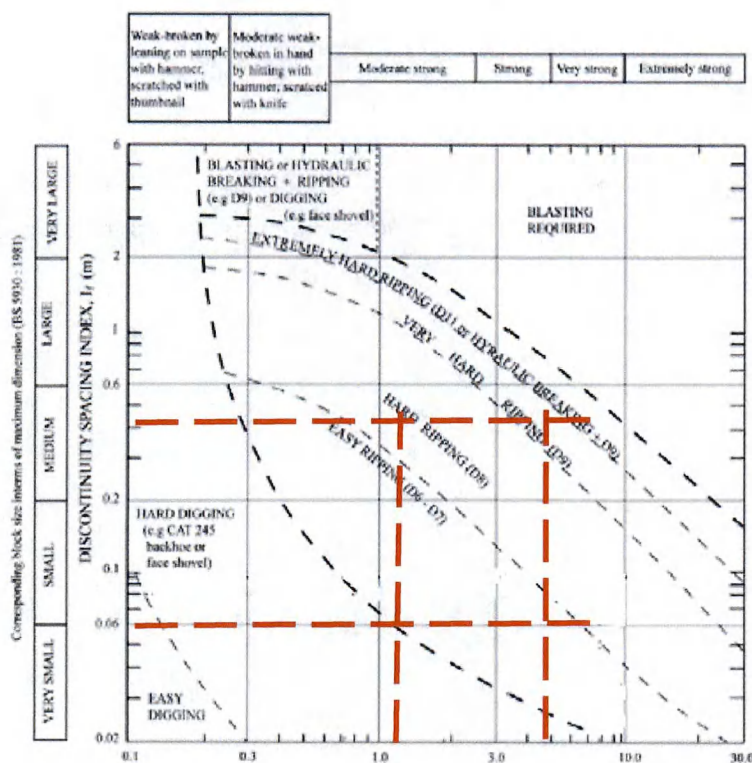
Turbine; T07 foundations are recommended to be constructed below 347mOD within the bedrock. Bases on the rock mass characteristic derived from geophysical investigation at the turbine location an allowable bearing pressure of 215kPa is recommended.

In all cases T01 – T07, the recommended lower limit of allowable bearing pressure exceeds the minimum requirement for the proposed foundation design where the foundations are constructed within the rock mass.

It is recommended at construction phase that plate bearing tests are carried out to verify the bearing pressure and assess the Modulus of the rock mass at foundation level.

6.5.1.1 Excavation

Excavation where required shall be by means of hydraulic excavator with hydraulic ripping and some hydraulic breaking in the upper zone of the slightly weathered rock. The energy requirement has not been assessed but can be expected to vary with location and depth. The rates of excavation at turbines; T02 and T03 will be reduced where fracture spacing and RQD was greater.



6.5.2 Substation

Silty sandy GRAVEL/ slightly sandy gravelly SILT was identified below a depth 0.55m bgl to 1.1m bgl TRL probing indicated CBR22% indicative of medium dense granular deposits or firm to stiff cohesive deposits.

TPS3 at a depth 1.1m bgl yielded a $w/PL = 1$ indicative of undrained shear strength 75kPa (C504 figure 5.19). An ultimate bearing pressure of 385kPa is expected. This should be verified by direct measurement using plate loading tests to determine soil strength. A safe bearing pressure of 75kPa is recommended (BS8004; Code of practice for Foundations; 1986) for settlements up to 25mm below a depth 1.0m bgl where the Silt fraction (17% to 44%) is considered dominant in terms of soil behaviour. Traditional shallow foundations will be suitable for the sub-station structure.

6.6 GROUNDWATER

Control of groundwater will be required during road construction. Appropriate drainage shall be provided to keep groundwater below formation level. Perched groundwater flow can be expected within the peaty deposits over the Silt.

6.7 CHEMICAL

pH and sulphate data indicated the site to be characterised by design sulphate class DS-1 in accordance with BRE special digest 1 (2003) concrete in aggressive ground or XA1 in accordance with IS EN 206.

7 REGISTER OF GEOTECHNICAL HAZARDS

Geotechnical hazards comprise of;

- Poor formation in the existing roadway;
- Localised soft extremely high plasticity compressible organic deposits, excessive rutting, long term settlement;
- Perched groundwater flow;
- Localised shallow bedrock, excavation in rock and
- Varied weathering profile of the rock mass.

The exact extents associated with the geo-hazards identified are not fully defined specifically the deeper Peat deposit associate with TP12.

8 SUMMARY

1. The turbine sites were characterised by dark brown to black PEAT, peaty CLAY and slightly sandy (slightly) gravelly SILT with varied Cobble content deposits 1.55m thick to 1.65m thick overlying bedrock. Medium strong to strong SILTSTONE was encountered 1.55m bgl (T03) to 1.65m bgl (T01). Sandstone was found to underlie the Siltstone at T01 and T04, below 4.5m bgl to 5.05m bgl. Strong SANDSTONE was encountered at 1.55m bgl (BHT02). Firm to stiff slightly sandy gravelly SILT overlay Bedrock 0.8m to 1.5m bgl at turbines; T05 to T07.
2. The access road was characterised by mixed deposits. Topsoil (slightly sandy slightly gravelly SILT/ clayey PEAT and PEAT, 150mm to 900mm thick, overlay silty sandy with varied Cobble content deposits 600mm to 1.55m thick and soft becoming firm to stiff, slightly sandy slightly gravelly SILT with varied Cobble content, 650mm to 2.7m thick. The Gravels and Silts were underlain by shallow bedrock 0.75m bgl to 2.3m bgl.
3. Clayey PEAT was encountered 2.0m deep at location TP12.
4. TRL dynamic probing was carried out at the thirty two (32) number trial pit locations providing for an estimate of unadjusted California bearing ratio, CBR. The data is presented in **APPENDIX A** of this report.
5. Dynamic probing DPH was carried out at eight (8) number locations along the existing access roadway from Castledonovan. The data is presented in **APPENDIX A** of this report.
6. A non-intrusive geophysical survey utilising Resistivity Soundings, Seismic Refraction Profiling and Multi-Channel Analysis of Surface Wave (MASW) at 7 proposed turbine locations techniques were carried out by PGL. A report ref: P16177_GP_Rp_F01, *March, 2017*, is presented within **APPENDIX B** of this report.
7. Groundwater was encountered between depths of 0.0 m bgl (ground level) and 2.0m bgl in trial pit excavations. Details are summarised herein and presented on the relevant trial pit logs presented in **APPENDIX A** of this report.
8. No groundwater was encountered during rotary drilling at the turbine locations. No standpipe wells were installed, see Section 5.

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9. Perched groundwater flow can be expected within the peaty deposits over the Silt.
 10. Detailed records of the ground and groundwater conditions can be found on the exploratory logs and photographic records presented within **APPENDIX A** of this report. Further details of the ground conditions can be found in the geophysical survey report presented within **APPENDIX B** of this report.
 11. Laboratory testing was undertaken to determine the classification, engineering properties of the soil and rock encountered during the site investigation. The data is presented in **APPENDIX C** of this report.
 12. The exploratory locations are presented on the location plans presented within **APPENDIX D** of this report.
 13. The Peat (CE) deposits are expected to be highly compressible. Historical in situ hand vane tests (*Others, 2010*) indicated undrained shear strengths 16kPa to 34kPa describing very soft to soft deposits.
 14. The Silt (ML) was described as soft to 'firm to stiff' deposits. The soft deposits were present to depths up to 1.3m bgl at locations; TP04, TP05, TP06, TP07, TP11, TP13, TP15, TP17, TP18, TP19, TP03A and TPS3. Undrained shear strengths are expected to vary between 20kPa and >75kPa. Plasticity data indicated a friction angle, $\phi = 26^\circ$.
 15. The SILT and GRAVEL shall be considered a singular deposit - mixed glacial till.
 16. The rock mass rating, RMR range was 44 - 64. The Siltstone and Sandstone bedrock was classified as Class III-II fair to good rock with unit weight 2.71Mg/m^3 . A Poisson's ratio, ν was measured = 0.215. The intact Young's Modulus, E was 55.7GPa. Allowing for the non-intact nature of the rock mass this was reduced by a factor of 0.15 yielding a design modulus of 8.4GPa. A friction angle between 30° and 35° with cohesion 250kPa to 300kPa are recommended.
 17. The Sandstone and Siltstone is expected to be suitable for re-use as capping with $50\text{kN} < 10\%\text{FV} < 130\text{kN}$ and $\text{MSSV} < 25\%$.

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18. For settlement of the order 3mm/m at the proposed turbine locations, an allowable bearing pressure of 130kPa to 255kPa is recommended.
 19. A safe bearing pressure of 75kPa is recommended below a depth 1.0m bgl for foundations associated with the sub-station infrastructure.
 20. Excavation shall be by means of hydraulic excavator with hydraulic ripping and some hydraulic breaking in the upper zone of the slightly weathered rock. The rates of excavation at turbines; T02 and T03 will be reduced where fracture spacing and RQD was greater.
 21. A capping requirement of 600mm is recommended for pavement design. No capping is required where the subgrade is on weathered rock, 150mm of surfacing cl. 804 sub-base or similar is recommended. A summary of the CBR assessment is provided in **APPENDIX E**.
 22. 'Reversal road' construction shall be considered where the weathered rock mass was at depths <1.0m bgl. Shallow road cuttings are also recommended. A minimum road width at formation of 7m is expected in shallow cutting and through Peat deposits. Drainage shall be provided a minimum of 450mm below sub-formation, the underside of capping
 23. A CBR_{2%} is expected to varied depths between 1.0m bgl and 2.0m bgl along the existing roadway as assessed by dynamic probes DP(H); DP01 to DP08.
 24. The site was characterised by design sulphate class DS-1 in accordance with BRE special digest 1 (2003) concrete in aggressive ground or XA1 in accordance with IS EN 206.
 25. A summary register of geotechnical hazards has been provided.
 26. A number of recommendations are made with regard to quality control and verifying design details at construction phase, typically requiring *in situ* plate loading testing.