APPENDIX 12.6: GEOTECHNICAL INTERPRETIVE REPORT

New Deer 2 400kV Substation

Geotechical Inteperative Report

Scottish & Southern Electricity Networks (SSEN)

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1. Project Background

1.1. Introduction

Tony Gee and Partners LLP (TGP) was commissioned by Scottish and Southern Energy Networks (SSEN) (the Employer) to act as the Investigation Supervisor for the Ground Investigation (GI) works at the proposed New Deer 2 400kV Substation, herein referred to as the Site, and subsequently develop a Geotechnical Interpretive Report (GIR).

The Site is located approximately 15km East of Turriff, Aberdeenshire. The Site Location is shown in Figure 1, with the site boundary shown in red.

Figure 1. Site Location Plan

The GI works, which were designed and specified by SSEN, were undertaken by the BAM Ritchies (the Contractor) between 21st August 2023 and 20th October 2023. The scope of GI works was presented within the GI Technical Specification (SSEN Document Reference LT379-GTS-CIV-002 Rev. 2.0).

1.2. Proposed Development

The project involves construction of a new 400kV double busbar air-insulated switchgear (AIS) substation to the North-West of the existing New Deer 275kV Substation. The requirement for the proposed development is driven by the need to increase the capacity of the onshore electricity transmission infrastructure, to deliver 2030 carbon reduction targets and a pathway to net zero.

Civil infrastructure associated with the development will comprise cut and fill earthworks to create the level substation platform, construction of site access tracks, foundations to support the proposed structures and substation equipment, sustainable drainage ponds and cabling works. It is possible that retaining structures and / or engineering slopes may also form part of the development.

A site layout plan is presented within Figure 2.

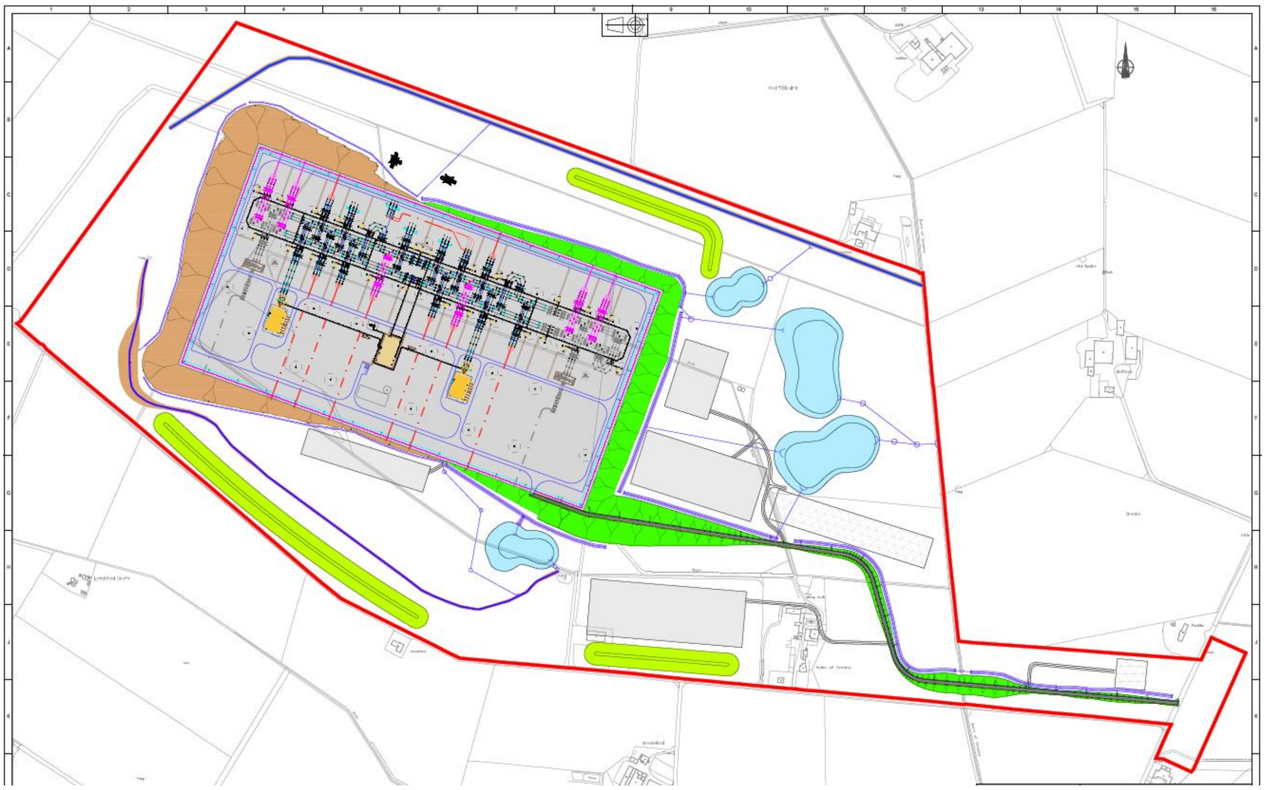

Figure 2. Site Layout Plan

1.3. Objectives and Methodology

This Geotechnical Interpretive Report (GIR) has been produced by TGP to comply with BS EN 1997-1:2004+A1:2013 ‘Eurocode 7: Geotechnical design – Part 1: General rules’ (British Standards Institution, 2013) and NA+A1:2004 to BS EN 1997-1:2004+A1:2013, the accompanying UK National Annex Eurocode 7: Geotechnical design - Part 1: General rules’ (British Standards Institution, 2014).

The objective of this GIR is to summarise and characterise the ground and groundwater conditions at the Site, to support SSEN in assessing the suitability of the Site and inform the design and planning of the proposed development.

This report includes:

- A summary of desk-based information relating to the Site;
- A description of the ground investigation works undertaken;
- A summary of the ground conditions encountered and an evaluation of their geotechnical and geo-environmental properties;
- A high-level engineering discussion in relation to the proposed development.

A further list of information sources referenced in this report are as follows:

- Clayton, C.R.I. (1995) “The Standard Penetration Test (SPT): Methods and Use”;

- M.A. Stroud (1989) “The Standard Penetration Test – It’s Application and Interpretation”;
- Carter, M. and Bentley, S. (2016) “Soil Properties and their Correlations”;
- The Manual of Contract Documents for Highway Works (MCHW) Specification for Highway Works (SHW) Series 600 – Earthworks.

The guidelines used to complete the geoenvironmental section of this report include the following:

- CIRIA C522- Contaminated Land Risk Assessment – A Guide to Good Practice;
- Scottish Water Quality Standards Factsheet 2;
- SEPA WAT-SG-53 – Environmental Quality Standards and Standards for Discharging to Surface Waters;

1.4. Limitations

The scope of this GIR is limited to the presentation and evaluation of geotechnical information obtained on the project to date and does not include quantitative design conclusions, only high-level recommendations.

To the extent that this document is based on information gathered during the recent ground investigation works, persons using or relying on it should recognise that any such investigation can examine only a fraction of the subsurface conditions which have inherent natural variability. Intrusive investigations are based on sampling at localised points and as such there remains a risk that unforeseen ground conditions may not be identified. An overview desk study has been undertaken as part of this report; however, it does not constitute a desk study meeting the requirements of BS EN 1997-1:2004+A1:2013 ‘Eurocode 7: Geotechnical design - Part 1: General rules’ (British Standards Institution, 2013).

The accuracy of information presented in this document is limited to the accuracy of the sources of information listed in Section 2, or as referenced throughout, and therefore this extends to the accuracy of interpretation and conclusions drawn from this information.

2. Desk Based Assessment

The following sections present the findings obtained by a high-level desk study. The information used for this study was found from online sources.

2.1. Site Setting

The Site is located to the West of New Deer in Aberdeenshire, Scotland and is to be used for a new 400kV AIS substation. The Site spans between the National Grid References NJ 82215 47155 and NJ 81442 47574. There is a single-track access farm road to the South of the site which connects to the existing A948 road.

Surrounding the North and East perimeters of the Site is farmland, while the West perimeter is bordered by forestry land which connects into the North-West corner. The South perimeter is bordered by the A948 road, and a small existing residential property is located within the southeast.

The Site is at its highest elevation of 155m at the West side, and then gradually slopes downwards to an elevation of 100m towards the East side. This elevation change will require a phase of earthworks to level the Site for the proposed substation.

2.2. Site History

Using historical mapping from 1888 to present day, the Site has been checked for any changes experienced in recent history. The historical maps from 1888 show that the roads surrounding the Site are consistent with those in the present day as well as the residential property to the South side. The Site appears to be made up of a combination of rough moorland and farmland and there are no structures present within it. There are no known operational or historical quarries or gravel pits present within the Site.

The next available historical mapping is from 1961 which shows the consistent roads and residential property from before. No quarries or gravel pits appear to be present within the Site and the moorland and farmland combination is like before. The Site continues to possess no structures within it.

The historical mapping of the Site indicates that no previous works have been undertaken due to the lack of present infrastructure. The residential property to the South of the Site is the only infrastructure that could affect the ground conditions of the Site.

Furthermore, some ditches are present throughout the Site and historical mapping shows these ditches to be present from 1888.

2.3. Geology

2.3.1. Superficial Geology

BGS online mapping[1] shows the Site to be underlain by Glacial Deposits (Till). The available GI typically shows a layer of Topsoil underlain by thin layers of Glacial Deposits comprising Clays, Gravels, Sands and Silts before typically reaching a shallow rockhead.

[1] BGS Geology Viewer [BGS Geology Viewer - British Geological Survey]

2.3.2. Solid Geology

The mapping shows the solid geology of the Site to comprise the MacDuff Formation containing micaceous Psammite, Semi-Pelite and Pelite. This metamorphic bedrock formed between 1000 and 541 million years ago. The available GI shows the MacDuff Formation of the Southern Highland Group comprising Psammite, Semi-Pelite and Pelite all ranging in strengths from extremely weak to strong which matches what is expected from the geological mapping. There is also a localised pocket of Conglomerate present within BH03 ranging in strength from moderately weak to medium strong and a potential pocket recovered as a Gravel in TP51.

2.4. Hydrology, Hydrogeology and Flooding

2.4.1. Hydrology

Using the SEPA Water Classification Hub[2], it is found that the Site contains no known classified surface water bodies. The only known surface water present within the Site is a small pond to the South border of the Site that should not affect the proposed plans. However, the presence of field drains is considered probable, given the Sites land use. There are also a few ditches present throughout the Site.

Furthermore, SEPA does not classify The Burn of Greens which is located outside the east site boundary. Despite this water body being located outside of the Site boundary, it sits at the bottom of the sloped site and may catch any run-off from the Site area.

2.4.2. Hydrogeology

The BGS Geoindex[3] was reviewed to obtain the expected groundwater conditions and information on the underlying aquifer. Using the online BGS hydrogeology mapping tool it was found that the Site is underlain by the Southern Highland Group which was classified as a low productivity aquifer. The classification is defined as an aquifer where there are small amounts of groundwater within any near surface weathered zones and secondary fractures.

2.4.3. Flooding

The Site is determined to be at low risk to flooding according to the SEPA Flood Map[4].

2.5. Mining and Quarrying

The Site is located outside the Coal Authority's[5] coal mining reporting area, so the risk from historical mining cannot be confirmed. However, there are no known or suspected areas of historical mining in the Site, or the surrounding area. Furthermore, review of historical mapping, as referenced previously, shows no sign of quarries from 1888 through to present day.

It should be noted that the MacDuff Formation is > 500Ma, meaning it pre-exists life. This means that the rock could therefore not host fossils and we can rule out coal mining on this basis.

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[2] SEPA Water Classification Hub Water Classification Hub (sepa.org.uk)
[3] BGS Geoindex GeoIndex (onshore) - British Geological Survey (bgs.ac.uk)
[4] SEPA Flood Mapping Flood Maps | SEPA - Flood Maps | SEPA
[5] Coal Authority Map Interactive Map Viewer | Coal Authority (bgs.ac.uk)

2.6. UXO

Unexploded Ordnances (UXOs) are explosive weapons such as bombs, bullets or landmines that did not detonate when first deployed. These weapons pose a risk in the present day as they could still potentially detonate. The Zetica[6] website is used to check the risk of UXO's, and it was found that the Site is believed to be at a low risk.

2.7. Historical Ground Investigation Information

No historical ground investigation data was available for the Site prior to the completion of the ground investigation works.

2.8. Environment and Ecology

Regular walkover surveys of the Site were undertaken by an Ecological Clerk of Works (ECoW) provided by Envirocentre on behalf of the Contractor. The ECoW was on site three days per week during the GI works, and produced weekly reports on the ecological findings. The summary findings of these walkovers are as follows:

- Species identified on Site included roe deer, pheasant, rabbits, sparrows, crows, peacock butterflies and badgers.
- A private water supply was identified close to BH04 which required sampling and monitoring to satisfy the user that there will be no adverse effects from the works.
- The invasive species Monkey Flower was identified as being present in some drainage ditches across the Site. Site staff were made aware of these flowers and informed to practice caution when working close to these areas.

The ECOW weekly reports are included in the Factual Report in Appendix A.

[6] Zetica UXO Risk Map [Zetica | Reduce the Risk of the Unknown]

3. Ground Investigation Works

3.1. Purpose and Scope of Investigation

The GI works, which were designed and specified by SSEN, were undertaken by BAM Ritchies between 21st August and 20th October 2023 to inform the detailed design and construction of the proposed development, including but not limited to:

- Optimisation of the earthworks cut and fill balance, and determination of the suitability of material on Site to be reused during construction of the substation platform;
- The design of the foundations, required to support all structures, plant and equipment;
- Ground conditions for access roads/ other infrastructure;
- The presence of any contaminated ground;
- The drainage strategy;
- Slope stability assessment;
- Quantification of ground risk;
- Assist with proposing ground risk mitigation strategies;
- Highlighting the potential for variations in ground conditions;
- Inform proposed construction methodologies.

The findings of the GI works are summarised within the GI Factual Report, contained within Appendix A.

The GI works comprised 108 No. intrusive exploratory holes across the Site, complete with in-situ testing and soil sampling, which are intended to aid the development of the new substation and associated cables and ground levelling. The exploratory holes consisted of an array of 55 No. Trial Pits (TPs), 46 No. Boreholes (BHs) and 7 No. Hand Pits (HPs). Standpipe installations were installed into 16 No. of the boreholes across the Site. Following the Site works, a phase of geotechnical and geo-environmental laboratory testing on recovered soil and rock samples followed, along with a 12-week period of ground water and ground gas monitoring.

An as-built exploratory hole location plan is shown in Figure 3. A summary of the exploratory holes completed as part of the Site works is provided in Table 1, Table 2 and Table 3 below.

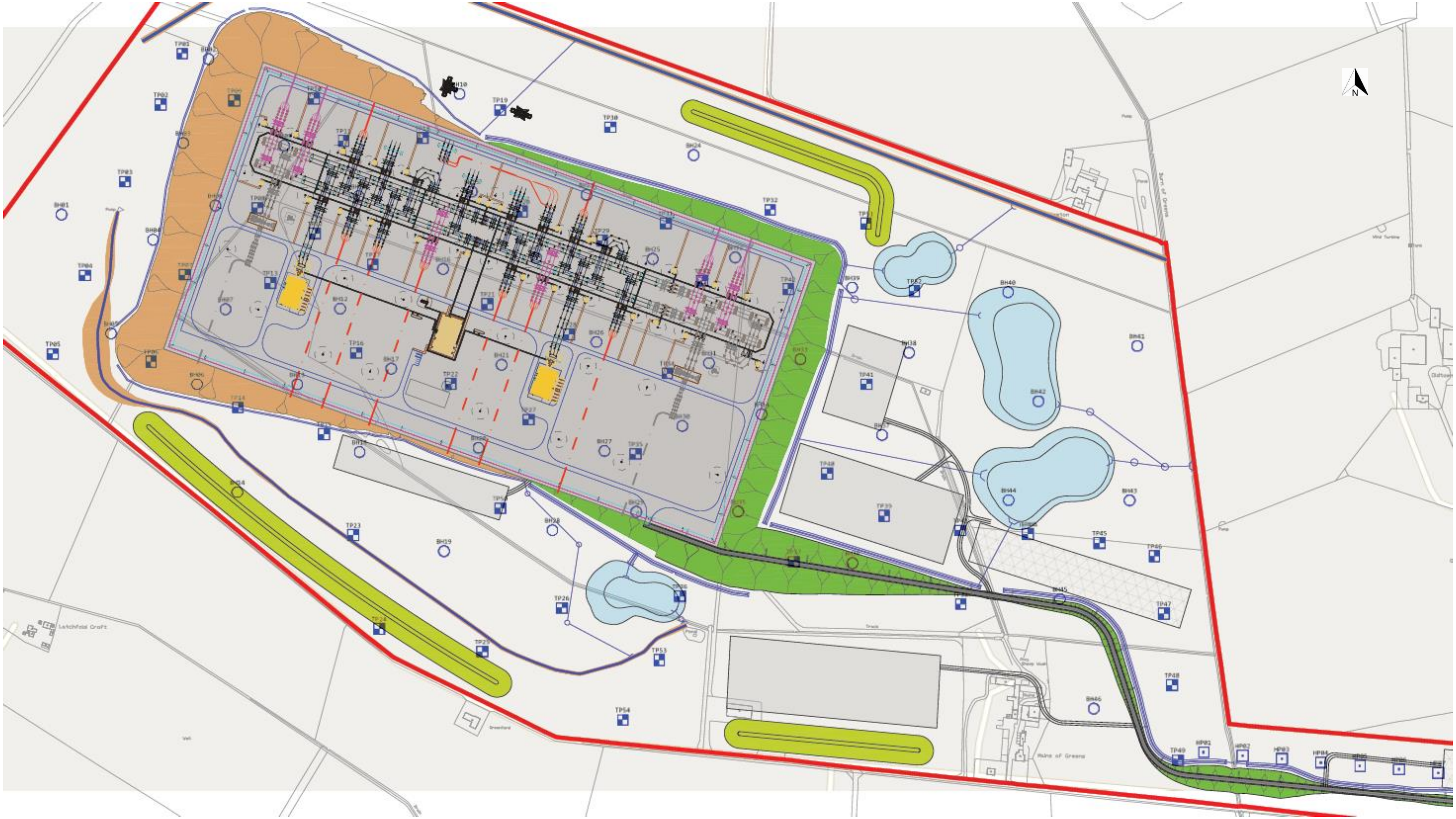

Figure 3. Exploratory Hole Location Plan

Table 1. Borehole Summary

Hole ID	Type	Final Depth (m bgl) [m OD]	SPTs (No.) [S / C]
BH01	SNC + RC	20.20 [131.63]	4 [S]
BH02	SNC + RC	20.00 [129.48]	2 [S]
BH03	SNC + RC	20.60 [128.95]	3 [S]
BH04	SNC + RC	20.00 [125.22]	7 [S]
BH05	SNC + RC	20.00 [130.55]	3 [S]
BH06	SNC + RC	20.00 [123.98]	5 [S], 1 [C]
BH07	SNC	20.00 [122.17]	13 [S]
BH08	SNC + RC	20.30 [127.77]	1 [S]
BH09	SNC + RC	20.00 [126.35]	2 [S]
BH10	DS + RC	15.00 [118.31	1 [S]
BH11	DS + RC	15.00 [125.29]	1 [S]
BH12	DS + RC	11.00 [128.91]	2 [S]
BH13	SNC + RC	15.00 [120.75]	3 [S]
BH14	SNC + RC	15.20 [126.72]	2 [S]
BH15	DS + RC	15.00 [118.89]	1 [S]
BH16	SNC + RC	15.00 [122.12]	2 [S]
BH33	DS + RC	10.00 [104.91]	1 [S]
BH34	DS + RC	10.00 [106.95]	1 [S]
BH35	DS + RC	10.00 [107.24]	1 [S]
BH36	DS + RC	10.00 [100.77]	1 [S]
BH37	DS + RC	10.00 [100.97]	1 [S]
BH38	DS + RC	10.00 [104.26]	1 [S]

Table 1. Borehole Summary cont.

Hole ID	Type	Final Depth (m bgl) [m OD]	SPTs (No.) [S / C]
BH39	DS + RC	10.00 [107.37]	1 [S]
BH40	DS + RC	15.20 [95.25]	2 [S]
BH41	DS + RC	14.00 [90.75]	1 [S]
BH42	DS + RC	16.00 [91.53]	7 [S]
BH43	DS + RC	8.00 [95.37]	1 [S]
BH44	DS + RC	10.00 [96.16]	2 [S]
BH45	DS + RC	10.30 [93.27]	3 [S]
BH46	DS + RC	7.20 [96.61]	1 [S]

<u>Notes</u>

1. 1. All boreholes commenced with a hand-dug inspection pit to a depth of 1.2 m bgl.
2. 2. BH = Borehole; DS = Dynamic Sampling; RC = Rotary Coring
3. 3. S = Split-spoon SPT sampler; C = Cone SPT sampler

Table 2. Machine Excavated Trial Pit (METP) Summary

Hole ID	Type	Final Depth (m bgl) [m oD]
TP01	METP	2.00 [148.20]
TP02	METP	3.00 [149.44]
TP03	METP	3.30 [147.51]
TP04	METP	4.00 [144.71]
TP05	METP	1.20 [151.97]
TP06	METP	3.00 [144.20]
TP07	METP	3.60 [139.89]
TP08	METP	3.00 [143.08]
TP09	METP	2.00 [147.47]
TP10	METP	1.10 [143.01]
TP11	METP	3.40 [138.76]
TP12	METP	2.90 [139.73]
TP13	METP	3.20 [139.83]
TP14	METP	3.80 [137.80]
TP15	METP	3.50 [132.24]
TP16	METP	3.50 [132.89]
TP17	METP	3.30 [135.49]
TP18	METP	3.40 [131.94]
TP19	METP	3.30 [127.71]
TP20	METP	3.60 [126.35]
TP21	METP	3.20 [131.05]
TP22	METP	2.90 [132.81]

Hole ID	Type	Final Depth (m bgl) [m oD]
TP23	METP	2.30 [133.53]
TP24	METP	3.30 [130.89]
TP25	METP	3.50 [125.86]
TP26	METP	3.60 [118.63]
TP27	METP	3.60 [127.83]
TP28	METP	3.40 [126.02]
TP29	METP	3.60 [120.92]
TP30	METP	3.90 [125.70]
TP31	METP	3.60 [116.79]
TP32	METP	3.60 [118.69]
TP33	METP	3.60 [115.31]
TP34	METP	3.40 [119.02]
TP35	METP	3.30 [120.10]
TP36	METP	2.00 [114.41]
TP37	METP	3.90 [109.12]
TP38	METP	3.90 [102.50]
TP39	METP	3.10 [106.73]
TP40	METP	3.60 [109.24]
TP41	METP	3.40 [108.82]
TP42	METP	2.60 [115.27]
TP43	METP	3.60 [103.46]
TP44	METP	1.00 [104.57]

Hole ID	Type	Final Depth (m bgl) [m oD]
TP44	METP	1.00 [104.57]
TP44A	METP	3.90 [101.67]
TP45	METP	3.70 [99.58]
TP46	METP	3.30 [98.87]
TP47	METP	3.30 [98.00]
TP48	METP	3.00 [96.96]
TP49	METP	3.70 [95.13]
TP50	METP	3.20 [124.09]
TP51	METP	3.90 [115.06]
TP52	METP	3.90 [112.49]
TP53	METP	3.30 [115.72]
TP54	METP	2.20 [120.49]

Notes

1. METP = Machine Excavated Trial Pit.
2. TP01 – TP35, TP38 – TP43 and TP49 – TP54 all terminated early due to encountering obstruction, possible bedrock.
3. TP36 - TP37 and TP44A – TP48 terminated due to sidewall collapse.
4. TP44 encountered a field drain and was micro-sited (TP44A).

Table 3. Hand Pit (HP) Summary

Hole ID	Type	Final Depth (m bgl)	Remarks / Comments
HP01	HP	0.60 [98.33]	Terminated on possible boulder obstruction.
HP02	HP	1.00 [98.83]	Terminated on possible boulder obstruction.
HP03	HP	0.55 [103.98]	Terminated on possible boulder obstruction.
HP04	HP	0.50 [104.00]	Terminated on possible boulder obstruction.
HP05	HP	0.70 [102.87]	Terminated on possible boulder obstruction.
HP06	HP	1.20 [101.35]	Terminated at scheduled depth.
HP07	HP	1.20 [101.05]	Terminated at scheduled depth.

3.2. Geotechnical Testing

3.2.1. In-Situ Testing

The in-situ testing undertaken during the GI works comprised a combination of Standard Penetration Tests (STPs) within Boreholes, Soakaway Tests in Trial Pits and DCP Tests adjacent to Trial Pits. The in-situ tests are summarised within Table 4 below.

Table 4. Summary of in-situ testing undertaken

Test Type	Location	Quantity	Remarks / Comments
SPTs	At regular depth intervals within all boreholes	112	-
Soakaway Tests	TP51, TP52, TP53 and TP54	4	Undertaken at 1.5m bgl in specified Trial Pits.
DCP Tests	HP01, HP02, HP03, HP04, HP05, HP06, HP07, TP44, TP46 and TP48.	10	Undertaken adjacent to specified Trial Pits and Hand Pits

3.2.2. Sampling and Laboratory Testing

The geotechnical laboratory tests undertaken are detailed in Table 5 and Table 6 below.

Table 5. Geotechnical Laboratory Tests - Soils

Test Type	Test Method	Quantity
Classification	Moisture Content	213
	Atterberg Limits	30
	Particle Size Distribution (PSD)	174 (160 with sedimentation)
Compaction	2.5 kg Compaction	10
	4.5 kg Compaction	31
	Single Point Moisture Condition Value (MCV)	9
	Moisture Condition Value (MCV) Calibration Line	4
Soil Strength	Standard Shear Box	31

Table 6. Geotechnical Laboratory Tests – Rock and Aggregate

Test Type	Test Method	Quantity
Classification	Rock Moisture Content	14
Aggregate	Magnesium Sulphates	6
	Los Angeles Abrasion	3
Rock Strength	Point Load Test (PLT)	17
	Uniaxial Compressive Strength (UCS)	1

Notes

1. PLT tests included 9 No. irregular lump tests.
2. 9 No. PLT test were undertaken as a replacement for a UCS test due to non-conformance.

3.3. Geo-chemical Testing

3.3.1. Sampling and Laboratory Testing

The geo-chemical laboratory testing undertaken as part of the project is summarised within Table 7.

Table 7. Geo-chemical Laboratory Testing

Test Type	Test Method	Quantity
Soil Chemical	BRE Suite A	4
	BRE Suite B	13
	BRE Suite D	6
	Total Sulphates	16
	pH	15
	Organic Matter	7
Water Contaminant	Suite F	10

No in-situ geo-environmental testing was undertaken in any of the exploratory holes during the fieldwork.

3.4. Groundwater and Ground Gas Monitoring

50mm diameter standpipes complete with valve taps for gas monitoring and removable caps for water level monitoring and sampling, were installed in 16 No. boreholes during the fieldwork. Details of these installations are shown in 8 below.

Table 8. Standpipe Installation Details

Exploratory Hole	Plain Standpipe		Slotted Standpipe	
	Response Zone Top (m bgl)	Response Zone Bottom (m bgl)	Response Zone Top (m bgl)	Response Zone Bottom (m bgl)
BH01	0.00	1.00	1.00	5.00
BH02	0.00	0.50	0.50	3.50
BH05	0.00	1.00	1.00	8.00
BH07	0.00	0.50	0.50	2.50
BH08	0.00	5.00	5.00	7.50
BH10	0.00	4.30	4.30	7.30

Exploratory Hole	Plain Standpipe		Slotted Standpipe	
	Response Zone Top (m bgl)	Response Zone Bottom (m bgl)	Response Zone Top (m bgl)	Response Zone Bottom (m bgl)
BH16	0.00	1.20	1.20	2.70
BH18	0.00	1.00	1.00	2.50
BH20	0.00	0.50	0.50	2.30
BH24	0.00	4.00	4.00	8.00
BH28	0.00	1.00	1.00	5.00
BH31	0.00	1.00	1.00	4.00
BH36	0.00	0.50	0.50	2.30
BH39	0.00	0.50	0.50	3.50
BH42	0.00	1.20	1.20	5.00
BH46	0.00	0.50	0.50	2.00

4. Ground Conditions Encountered

This section presents an evaluation of the ground and groundwater conditions encountered during the Ground Investigation which has resulted in the development of a Site-wide ground model for the substation Site, presented in tabular format. Geological cross-sections, through and across the Site, have also then been developed to illustrate the ground model and the interpretation of ground conditions across the Site.

4.1. General Ground Conditions

The ground conditions at the Site were found to typically comprise of 3 No. different geological units, as follows:

- Topsoil;
- Glacial Deposits (Granular and Cohesive Till);
- MacDuff Formation Bedrock.

4.1.1. Topsoil

Topsoil was recorded across the site from existing ground level, in 105 No. exploratory holes and has a thickness ranging between 0.1 - 0.6m, with an average thickness of 0.28m. It is generally described as either dark brown gravelly sandy Clay or dark brown clayey gravelly fine to medium Sand. Some exploratory holes identify a low cobble content within the Sand to the East of the Site.

4.1.2. Glacial Deposits

Glacial Deposits were recorded in 39 No. exploratory holes and comprises a mix of Clay, Silt, Gravel and Sand. The thickness of Glacial Deposits layers ranges between 0.25m – 2.7m with an average thickness of 0.95m. The locations that did not encounter Glacial Deposits all comprises Topsoil underlain by Weathered Rock.

Cohesive Glacial Deposits

The Glacial Deposits encountered throughout the Site predominately comprise cohesive material of Clay and Silt. Clay was encountered between 0.2 – 1.5m bgl in a total of 12 No. exploratory holes and has an average thickness of 0.72m. Silt was encountered between 0.2 – 3.3m bgl in a total of 10 No. exploratory holes and has an average thickness of 1.1m.

Granular Glacial Deposits

Granular Glacial Deposits were also identified to be present across the Site, which comprise Gravel and Sand. Gravel was encountered between 0.1 – 1.8m bgl in a total of 11 No. exploratory holes with an average thickness of 0.8m. Sand was encountered between 0.15 -2.1m bgl in a total of 6 No. exploratory holes with an average thickness of 1.4m.

4.1.3. Bedrock

Bedrock was encountered in all BHs and the majority of TPs. The TPs where bedrock was not encountered was due to either collapse of the pit or reaching the scheduled depth before encountering bedrock. The bedrock encountered was of the MacDuff Formation, comprising extremely weak to strong Pelite, extremely weak to medium strong Semi-Pelite and extremely weak to strong Psammite.

When rockhead is encountered, the rock is typically weathered or non-intact. Rockhead was encountered between 0.25 – 9.7m bgl with an average depth of 2.5m bgl. BH03 and TP51 identified "Conglomerate", which was recovered as non-intact Gravel, which also expected to belong to the MacDuff Formation.

4.2. General Groundwater Conditions

Groundwater strikes were recorded in 21 No. holes. Groundwater strike levels ranged from 0.6 – 3.3 m bgl with an average depth of 2.3m bgl. There is no noticeable trend with depth observed regarding groundwater strikes. However, the locations of the exploratory holes where groundwater strikes occurred are typically found to the East side of the Site. It should also be noted that limited groundwater strikes do occur in other areas of the Site.

Standpipes were installed in 16 No. boreholes to allow for monitoring of groundwater levels over a six fortnightly period commencing in November 2023 and is currently ongoing at the time of reporting. The available groundwater monitoring results show groundwater levels across the Site to fluctuate between ground level and 8.9m bgl and the largest variance of groundwater levels observed across the monitoring period was 6.75 m within BH08. There is no noticeable trend with depth observed regarding groundwater monitoring results as groundwater levels fluctuate across the entire Site.

A summary of the standpipe installation is presented in Table 8. The results from the groundwater monitoring to date are presented in Table 9 below.

Table 9. Groundwater Monitoring Results

Exploratory Hole	Depth of Water Level Range (m bgl)												
	29/08/2023	01/09/2024	07/09/2023	14/09/2023	30/09/2023	07/10/2023	14/10/2023	18/10/2023	01/11/2023	20/11/2023	05/12/2023	22/01/2024	07/02/2024
BH01	-	-	-	Dry	3.98	3.88	3.80	3.84	1.95	1.60	1.19	0.55	1.18
BH02	-	-	-	Dry	Dry	Dry	Dry	Dry	4.15	Dry	Dry	2.46	3.30
BH05	8.90	-	-	-	8.26	8.26	8.23	8.20	7.63	Dry	Dry	7.15	7.76
BH07	-	-	-	-	Dry	Dry	Dry	Dry	2.20	Dry	Dry	1.88	1.97
BH08	-	-	-	-	Dry	Dry	Dry	Dry	6.75	6.70	6.71	5.86	6.63
BH10	-	-	-	3.5	2.44	2.3	2.28	2.21	0.2	2.15	1.05	0.86	1.22
BH16	-	-	-	-	6.35	Dry	Dry	Dry	Dry	Dry	Dry	Dry	Dry
BH18	-	-	-	-	1.84	Dry	Dry	Dry	1.04	1.25	1.2	1.2	1.26
BH20		Dry	-	-	2.17	2.20	2.23	2.27	2.30	Dry	Dry	2.29	2.39
BH24	-	-	-	-	6.81	6.74	6.66	6.35	4.1	4.7	4.66	4.4	4.64
BH28	3.7	Dry	Dry	Dry	Dry	Dry	Dry	Dry	3.00	3.62	3.57	3.78	3.90
BH31	-	-	-	Dry	Dry	Dry	Dry	Dry	2.52	3.25	3.20	3.04	3.24
BH36	-	-	-	Dry	Dry	0.48	0.45	0.49	0.31	0.8	0.7	0.52	0.35
BH39	-	-	-	Dry	Dry	Dry	Dry	Dry	2.95	3.25	3.27	3.13	Dry
BH42	-	-	-	-	-	-	0.00	-	0.00	0.25	0.29	0.32	0.40
BH46	-	-	-	-	-	-	-	-	1.30	1.80	1.74	1.01	1.77

4.3. Ground Model

4.3.1. Tabulated Ground Model

A tabulated ground model has been developed for the Site and is presented in Table 10 below.

Table 10. Ground Model Summary

Stratum	Top of Stratum (m OD)	Average thickness (m)	Description
Topsoil	98.93 – 153.17	0.34	Grass over soft dark brown slightly gravelly sandy Clay.
	98.83 – 150.55	0.23	Grass over dark brown gravelly clayey Sand.
Glacial Deposits	103.61 – 150.05	0.72	Very soft to firm sandy gravelly Clay. Some localised areas where the Clay contains a low to high cobble content with some boulders.
	105.37 – 136.52	1.11	Soft to stiff sandy gravelly Silt.
	103.37 – 150.51	0.82	Sandy silty fine to coarse Gravel. Some localised areas where the Gravel contained a low cobble content.
	112.02 – 148.98	1.39	Medium dense silty gravelly Sand.
MacDuff Formation Bedrock	95.13 – 149.94	Unproven	Extremely weak to strong Pelite recovered predominately non-intact. Typically overlain by a layer of weathered Pelite or Semi-Pelite often recovered as a sandy gravel.
	95.13 – 152.87		Extremely weak to strong Psammite recovered predominately non-intact. Typically overlain by a layer of weathered Psammite often recovered as a sandy gravel.

Table 10 shows the typical ground model across the Site comprises a layer of Topsoil underlain by a layer of Glacial Deposits consisting of either Clay, Silt, Sand or Gravel. Beneath the Glacial Deposits, a layer of weathered rock typically overlays bedrock which is of the MacDuff Formation and consists of either Pelite or Psammite with strength varying from extremely weak to strong. The rock recovered throughout the Site is predominately weathered or non-intact, with minimal amounts of intact rock recovered.

Appendix B shows the ground models compared to the existing ground level of the Site and the proposed platform level, at a range of cross sections through the Site. The top of weathered rock and intact rock are shown on the ground models, however, intact rock was rarely encountered throughout the Site so the level of intact bedrock is not always known.

5. Evaluation of Geotechnical Information

5.1. Introduction

Geotechnical in-situ and laboratory test results for the Site are summarised and discussed in this section. Full test data is provided in the Factual Report contained in Appendix A.

5.2. In-Situ Test Results

5.2.1. SPT

The recorded SPT blow counts have been corrected to N_{60} values in accordance with the requirements of BS EN 1997 to account for energy loss due to frictional effects of the SPT hammer. Four different hammers were used all with differing energy ratio values. For these results the energy ratio of the hammer ranges between 56 – 81%, meaning the number of blows prior to refusal ranges between 47- 68 (N_{60}) , depending on the energy ratio of the hammer. The hammer energy ratio certificates are contained within the Factual Report in Appendix A.

The corrected N_{60} values are summarised in Table 11 below and plotted in Figure 3.

Table 11. Summary of SPT N_{60} values

Strata	Quantity	N_{60}			
		Min	Max	Average	No. of Refusals
Cohesive Glacial Deposits	3	35	42	38	0
Granular Glacial Deposits	2	27	68	47	1
MacDuff Formation	107	7	68	53	73

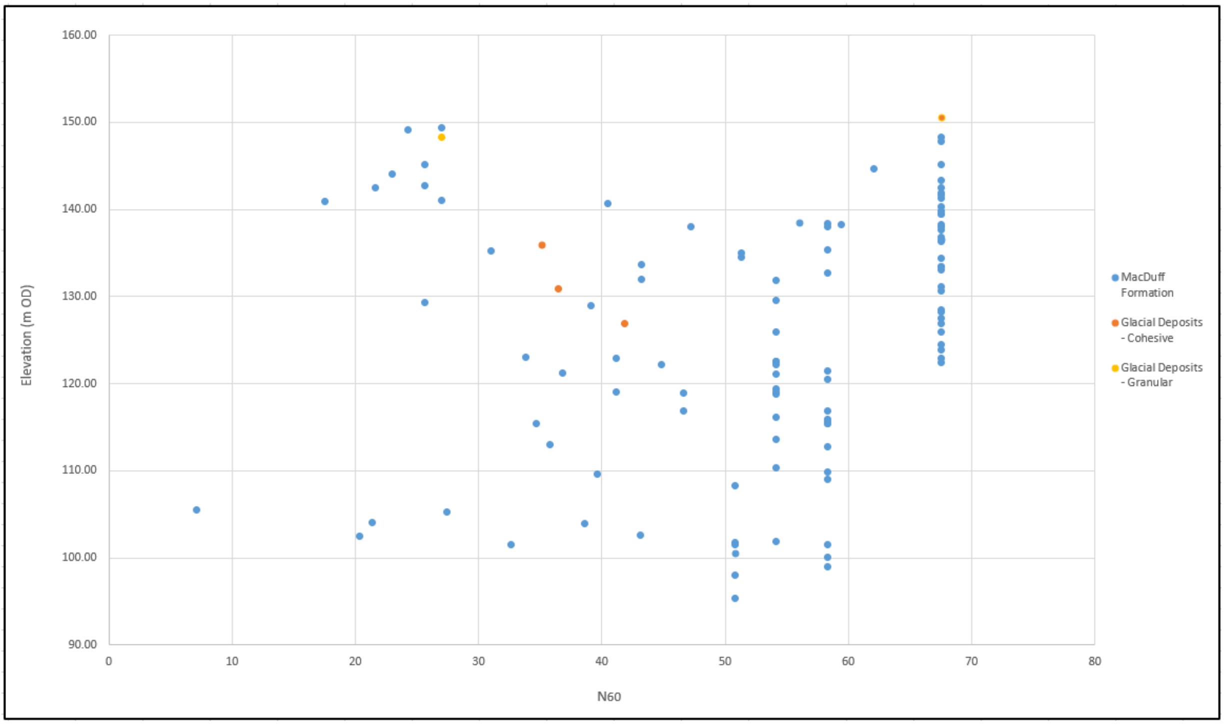

Figure 3. SPT vs. Depth Plot

5.2.2. Soakaway Test

The in-situ testing also comprised soakaway (infiltration) tests, which are summarised in Table 12 below.

Table 12. Soakaway Test Results

Test Type	Hole ID	Test Depth (m bgl)	Remarks / Comments
Soakaway	TP51	1.5	Test failed – water level dropped 54cm in test time (only t75 was achieved).
	TP52	1.5	Test failed – water level dropped 90cm in test time (only t75 was achieved).
	TP53	1.5	Test failed – water level dropped 52cm in test time (only t75 was achieved).
	TP54	1.5	Test failed – water level dropped 36cm in test time (only t75 was achieved).

All soakaway tests were recorded as 'Fails'. These results were recorded as fails as a result of only the t75 line being crossed during the test (defined as the time taken for the water volume to reach 75% of the pit storage capacity) with the t25 line not being reached (defined as the time taken for the water volume to reach 25% of the pit storage capacity). This result indicates low permeability ground; however, it should be noted that heavy rainfall was experienced during testing meaning that the results may have been affected by these conditions.

5.2.3. Dynamic Cone Penetration Test

10 No. Dynamic Cone Penetration (DCP) tests were conducted adjacent to specified Trial Pits and Hand Pits along the location of the proposed access track. The results of the DCP tests are shown in Table 13 below, with lower bound CBR values presented based on the data obtained. These CBR values represent the material for which the initial layers of penetration were achieved, typically through topsoil and superficial deposits, and not of the underlying rock material which recorded

Table 13. DCP Test Results

Test Type	Hole ID	Test Depth Range (m bgl)	Lower Bound CBR Value (%) [Representative Depth Range]	Strata
DCP	HP01	0.13 – 1.74	5.90 [0.00 – 0.42m]	Topsoil / MacDuff Formation
	HP02	0.00 – 0.67	7.00 – 9.90 [0.00 – 0.62m]	Topsoil / MacDuff Formation
	HP03	0.00 – 1.96	3.60 – 6.60 [0.00 – 1.87m]	Topsoil / MacDuff Formation
	HP04	0.00 – 0.87	2.90 – 3.60 [0.00 – 0.73m]	Topsoil / MacDuff Formation
	HP05	0.00 – 0.58	6.20 - 11.00 [0.00 – 0.45m]	Topsoil / MacDuff Formation
	HP06	0.00 – 0.79	5.00 – 6.70 [0.00 – 0.57m]	Topsoil / MacDuff Formation
	HP07	0.00 – 0.38	7.00 – 14.00 [0.00 – 0.24m]	Topsoil / MacDuff Formation
	TP44	0.10 – 2.10	7.00 – 16.00 [0.00 – 0.86m]	Topsoil / Cohesive Glacial Deposits
	TP46	0.08 – 1.71	7.00 [0.00 – 4.7m]	Topsoil / MacDuff Formation
	TP48	0.12 – 1.57	5.90 [0.00 – 4.2m]	Topsoil / MacDuff Formation

DCP tests were carried out from the surface immediately adjacent to the corresponding Trial Pit using the hand operated DCP. The CBR values are estimated using the TRL[7] correlation equation;

$$log_{10}(CBR) = 2.48 - 1.057 * log_{10}(mm/blow)$$

[7] Transport and Road Research Laboratory (1990) Overseas Road Note 8. A users manual for a program to analyse dynamic cone penetrometer data.

5.3. Geotechnical Laboratory Test Results

5.3.1. Atterberg Limit Tests

30 No. Atterberg Limit tests have been carried out on samples across the Site. Table 14 summarises the test results. Figure 4 shows the plot of the Atterberg Limit Test Results.

Table 14. Atterberg Limit Test Results

Strata	Quantity	Liquid Limit		Plastic Limit		Plasticity Index	
		Min	Max	Min	Max	Min	Max
Topsoil	1	54.00	54.00	35.00	25.00	19.00	19.00
Cohesive Glacial Deposits	15	22.00	53.00	16.00	34.00	6.00	34.00
MacDuff Formation	11	24.00	48.00	17.00	29.00	7.00	20.00

Notes

1. 2 No. sample from the MacDuff Formation were classified as being non-plastic.
2. 1 No. sample from the Cohesive Glacial Deposits was classified as being non-plastic.

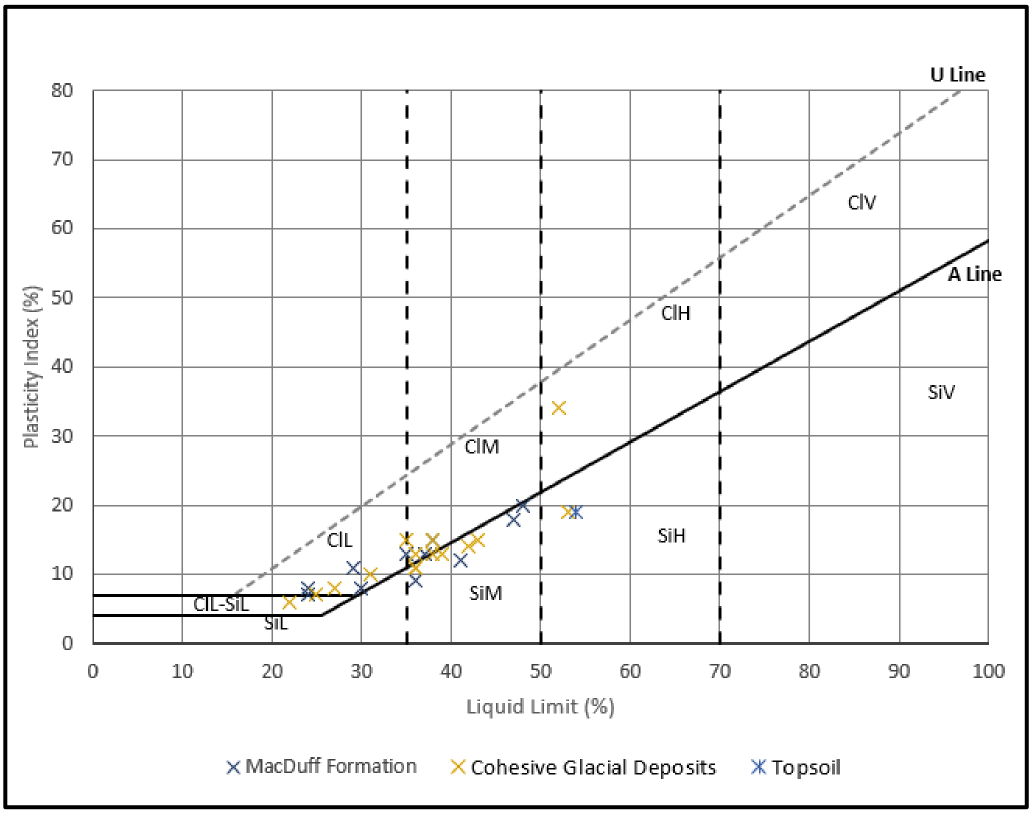

Figure 4. Atterberg Limit Test Results

5.3.2. Moisture Content (MC) Tests

221 No. moisture content tests have been conducted on samples across the Site. Table 15 summarises the test results.

Table 15. Moisture Content Test Results

Strata	Quantity	Moisture Content (%)		
		Min	Max	Average
Topsoil	2	27.00	37.00	32.00
Cohesive Glacial Deposits	7	8.80	15.00	12.11
Granular Glacial Deposits	43	3.00	38.00	16.86
MacDuff Formation	169	2.50	35.00	13.90

Figure 5 below presents the moisture content percentage against the elevation for each strata.

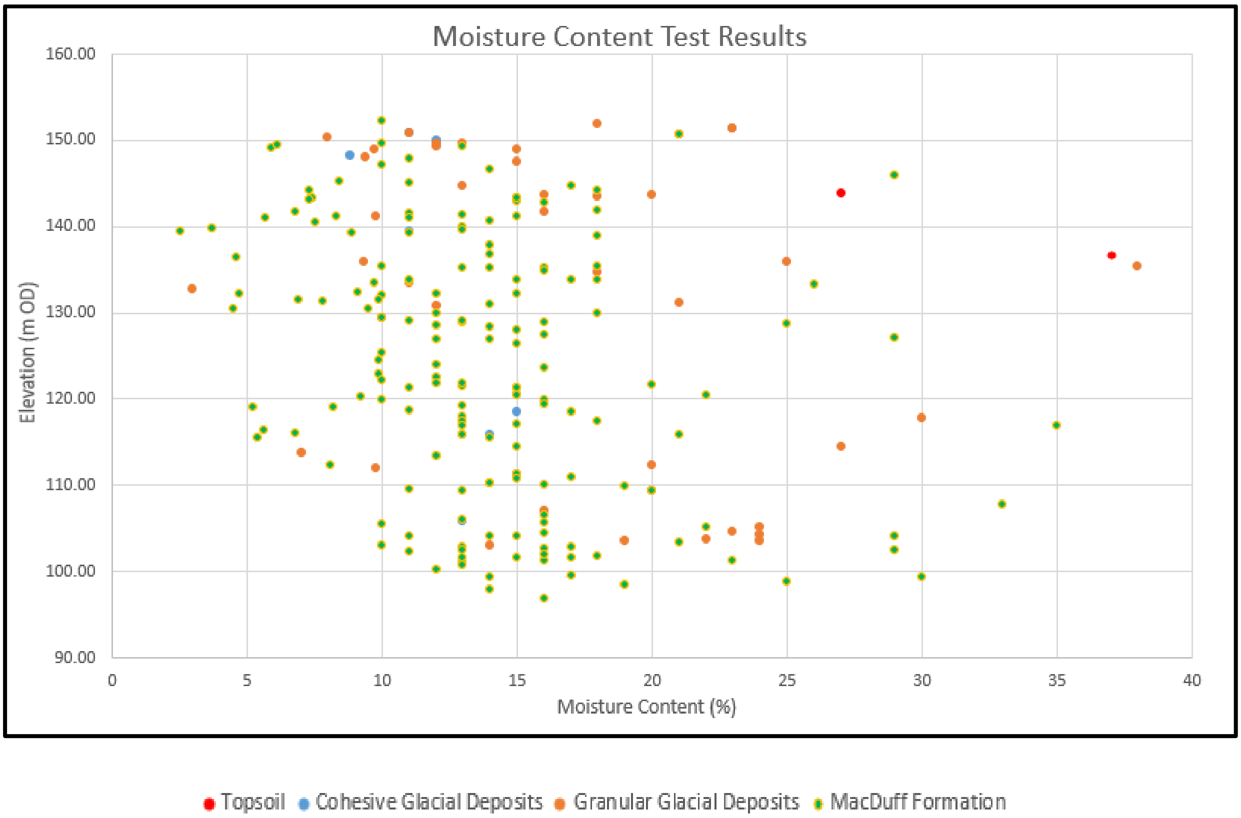

Figure 5. Plot of MC Test Results

5.3.3. Particle Size Distribution (PSD) Tests

Particle Size Distribution (PSD) analysis was undertaken on 174 No. samples, to determine the percentage (by weight) of soils passing openings ranging from 75 mm (coarse gravel) to 63µm (fine sand). 160 No. of the samples were also tested using sedimentation to differentiate the fine-grained portion of the material and determine the percentage of silt and clay respectively. The depth range of the data is from 0.3 to 7.0 m bgl across all exploratory holes that were subject to PSD testing. Samples tested were taken from Topsoil, Glacial Deposits and MacDuff Formation stratums and the results were highly varied, as can be seen from the plot for the Cohesive Glacial Deposits in Figure 6, Granular Glacial Deposits in Figure 7 and the plot for the MacDuff Formation in Figure 8 below.

Figure 6 and Figure 7 shows the Glacial Deposit samples to be a mix of cohesive and granular material which matches the engineering log descriptions while Figure 8 shows weathered MacDuff Formation generally to be of a granular nature, with a low percentage of cohesive material present. However, some samples of the weathered MacDuff Formation is described as being weathered to a Clay or Silt, which is supported by the grading curves in Figure 8 which indicate a greater percentage of cohesive material present in some locations. There was no notable trend as to the location where a higher percentage of cohesive material was encountered.

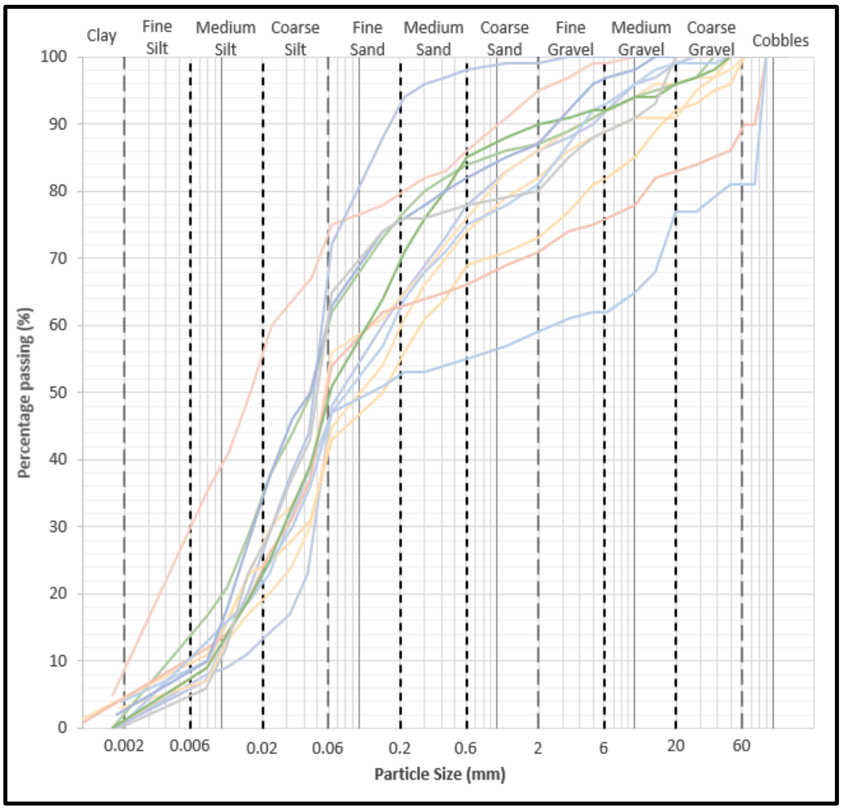

Figure 6. Plot of PSD Results – Cohesive Glacial Deposits

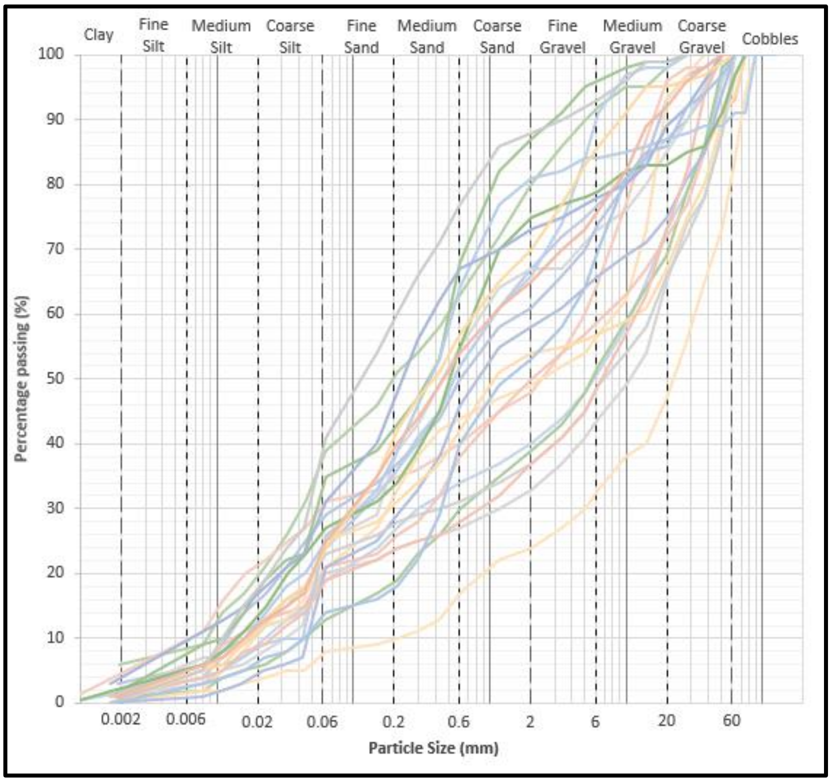

Figure 7. Plot of PSD Results – Granular Glacial Deposits

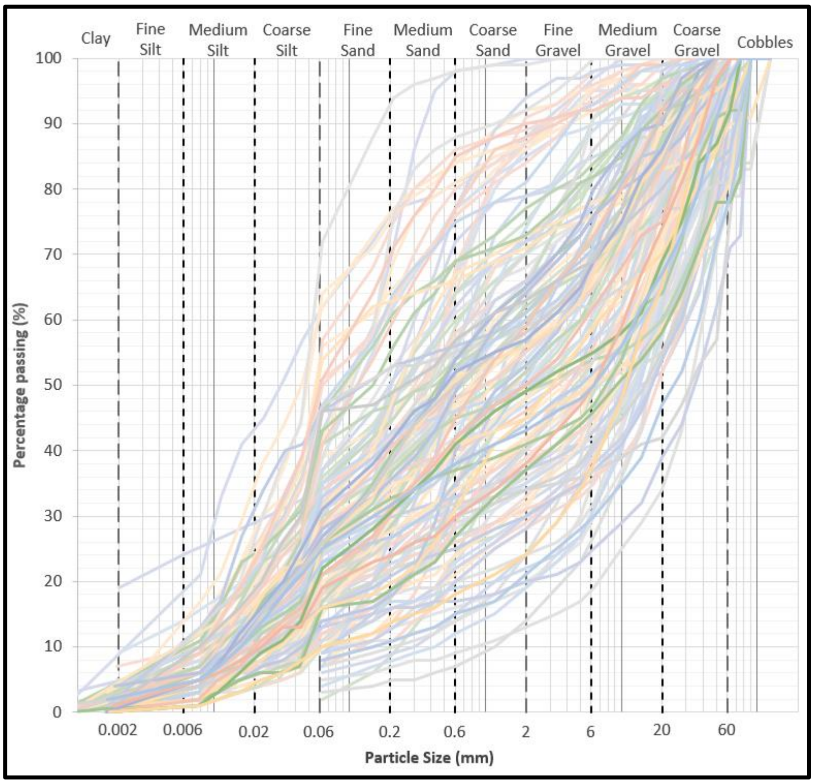

Figure 8. Plot of PSD Results – MacDuff Formation

5.3.4. 2.5 kg Compaction Tests

12 No. 2.5 kg compaction tests were scheduled; however 2 No. samples were recorded as being unsuitable for testing as a result of insufficient material to carry out the test. The compaction tests were undertaken using a 2.5 kg rammer with a combination of a CBR and 1L mould. Samples were taken from the Topsoil, Glacial Deposits and the MacDuff Formation. The purpose of these tests is to determine the dry density of the soil over a range of water contents. The plot of this data then allows for the derivation of the optimum water content at which the maximum dry density is achieved for this degree of compaction. The available test results are shown in Table 16 below.

Table 16. 2.5 kg Compaction Test Results

Exploratory Hole	Elevation of Sample (m OD)	Strata	Optimum Water Content (%)	Maximum Dry Density (Mg/m^3)
BH03	150.05	Cohesive Glacial Deposits	15.00	1.62
BH03	149.55	Cohesive Glacial Deposits	11.00	1.93
BH04	144.72	Cohesive Glacial Deposits	14.00	1.76
BH06	143.78	Topsoil	24.00	1.43
BH08	147.57	Cohesive Glacial Deposits	13.00	1.76
BH10	147.07	MacDuff Formation	10.00	2.06
BH11	145.85	Cohesive Glacial Deposits	16.00	1.69
BH13	132.41	Cohesive Glacial Deposits	16.00	1.74
BH16	139.89	Topsoil	20.00	1.49
TP09	147.97	Cohesive Glacial Deposits	8.80	1.97

5.3.5. 4.5 kg Compaction Tests

35 No. 4.5 kg compaction tests were scheduled; however 4 No. samples were recorded as being unsuitable for testing as a result of the material being too coarse (> 30% material retained on 20 mm sieve). The compaction tests were undertaken using a 4.5 kg rammer with a combination of a CBR and 1L mould. Samples were taken from the Topsoil and the MacDuff Formation. The purpose of these tests is to determine the dry density of the soil over a range of water contents. The plot of this

data then allows for the derivation of the optimum water content at which the maximum dry density is achieved for this degree of compaction. The available test results are shown in Table 17 below.

Table 17. 4.5 kg Compaction Test Results

Exploratory Hole	Elevation of Sample (m OD)	Strata	Optimum Water Content (%)	Maximum Dry Density (Mg/m^3)
BH08	147.07	MacDuff Formation	7.30	2.09
BH09	145.85	MacDuff Formation	11.00	1.87
BH15	133.19	MacDuff Formation	10.00	2.01
BH18	131.57	Topsoil	12.00	1.75
BH22	129.94	MacDuff Formation	7.30	2.08
BH23	122.23	MacDuff Formation	11.00	1.93
BH24	122.47	MacDuff Formation	11.00	1.84
BH25	115.43	MacDuff Formation	9.70	2.02
BH25	113.93	MacDuff Formation	10.00	2.02
BH26	126.69	MacDuff Formation	12.00	1.83
TP02	150.94	MacDuff Formation	8.50	2.10
TP04	147.21	MacDuff Formation	11.00	2.00
TP06	145.70	MacDuff Formation	9.10	2.08
TP06	145.20	MacDuff Formation	9.40	2.06
TP07	141.99	MacDuff Formation	7.80	2.11
TP08	144.58	MacDuff Formation	9.60	2.05
TP11	141.16	MacDuff Formation	7.80	2.09
TP12	141.63	MacDuff Formation	9.50	2.12
TP13	142.03	MacDuff Formation	7.60	2.13
TP14	140.60	MacDuff Formation	11.00	2.01
TP15	134.74	MacDuff Formation	12.00	2.02
TP16	135.39	MacDuff Formation	11.00	2.01

Exploratory Hole	Elevation of Sample (m OD)	Strata	Optimum Water Content (%)	Maximum Dry Density (Mg/m^3)
TP17	137.79	MacDuff Formation	9.90	2.00
TP18	134.34	MacDuff Formation	11.00	1.96
TP30	128.60	MacDuff Formation	11.00	1.94
TP30	126.60	MacDuff Formation	11.00	2.00
TP32	121.29	MacDuff Formation	11.00	1.97
TP32	119.29	MacDuff Formation	9.30	2.02
TP38	103.40	MacDuff Formation	9.10	2.02
TP42	115.87	MacDuff Formation	12.00	1.96
TP43	105.06	MacDuff Formation	12.00	1.92

A plot of the 2.5kg and 4.5kg compaction test results is presented in Figure 10 below.

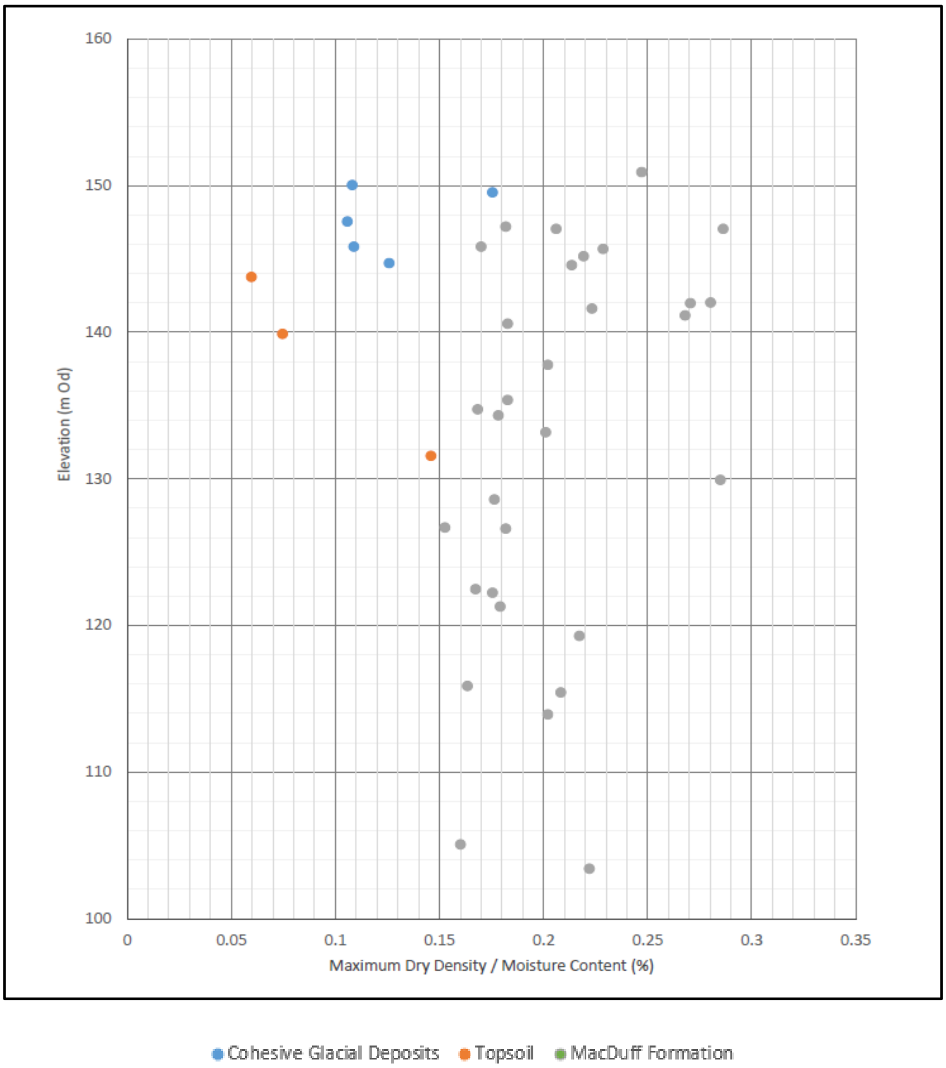

Figure 10 – Plot of Compaction Test Results

5.3.6. Single Point Moisture Condition Value (MCV) Tests

9 No. Single Point Moisture Condition Value (MCV) tests were conducted to obtain values of the minimum compactive effort required to achieve almost complete compaction of a specimen passing a 20 mm sieve. The MCV itself is an empirical figure which is determined by plotting the change in penetration of the compaction hammer between blows against the number of blows. The higher the MCV, the more blows required to achieve the same penetration. The available test results are shown in Table 18 below.

Table 18. Single Point MCV Test Results

Exploratory Hole	Elevation of Sample (m OD)	Specimen Description	Percentage Retained on 20mm Sieve (%)	Moisture Condition Value	Geological Unit
BH01	147.63	Brown Clayey SAND	0.00	9.90	MacDuff Formation
BH06	143.58	Sandy Gravelly CLAY	29.00	14.10	Cohesive Glacial Deposits
BH07	141.17	Sandy Gravelly CLAY	18.00	9.30	Cohesive Glacial Deposits
BH09	145.35	Brown Silty Gravelly SAND	2.00	7.90	MacDuff Formation
BH20	131.45	Brown Silty Sandy GRAVEL	58.00	10.70	MacDuff Formation
BH24	123.07	Brown Silty SAND	0.00	13.10	MacDuff Formation
BH25	117.53	Brown Clayey SILT	0.00	3.40	MacDuff Formation
BH25	116.53	Very Gravelly Clayey SAND	0.00	6.00	MacDuff Formation
BH25	112.43	Very Gravelly Clayey SAND	4.00	4.20	MacDuff Formation

5.3.7. Moisture Condition Value (MCV) Calibration Line Tests

4 No. Moisture Condition Value (MCV) calibration line tests were conducted to obtain a range of MCV values for a range of moisture contents. The results of these tests are summarised in Table 19.

Table 19. MCV Calibration Line Test Results

<table>
<tr><th>Exploratory Hole</th><th>Elevation of Sample (m OD)</th><th>Specimen Description</th><th>Percentage Retained on 20mm Sieve (%)</th><th>Moisture Content (%)</th><th>Moisture Condition Value</th><th>Geological Unit</th></tr>
<tr><td rowspan="4">TP02</td><td rowspan="4">150.94</td><td rowspan="4">Brown clayey sandy GRAVEL</td><td rowspan="4">29.00</td><td>9.00</td><td>13.40</td><td rowspan="4">MacDuff Formation</td></tr>
<tr><td>10.90</td><td>11.10</td></tr>
<tr><td>14.20</td><td>3.00</td></tr>
<tr><td>12.20</td><td>7.00</td></tr>
<tr><td rowspan="3">TP04</td><td rowspan="3">147.71</td><td rowspan="3">Brown clayey gravelly SAND</td><td rowspan="3">24.00</td><td>14.50</td><td>4.60</td><td rowspan="3">MacDuff Formation</td></tr>
<tr><td>12.20</td><td>6.40</td></tr>
<tr><td>10.00</td><td>11.40</td></tr>
</table>

Exploratory Hole	Elevation of Sample (m OD)	Specimen Description	Percentage Retained on 20mm Sieve (%)	Moisture Content (%)	Moisture Condition Value	Geological Unit
				8.20	13.10	
TP06	145.70	Brown clayey gravelly SAND	14.00	10.30	10.10	MacDuff Formation
				9.30	11.10	
				14.00	0.90	
				8.80	11.90	
TP09	148.47	Brown clayey gravelly SAND	33.00	9.70	7.30	Granular Glacial Deposits
				13.50	3.30	
				11.30	5.10	
				6.00	13.60	

A plot of the MCV test results is presented in Figure 11.

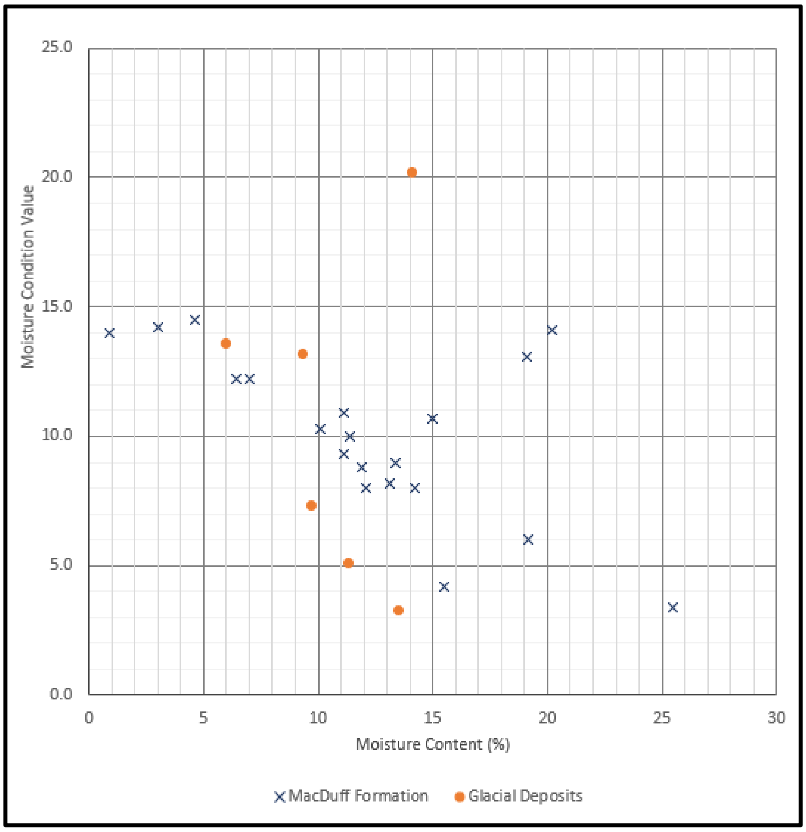

Figure 11 – MCV test results

5.3.8. Standard Shearbox Tests

34 No. shear box tests were carried out, however 3 No. tests were recorded to be unsuitable for testing due to insufficient material. 6 No. shear box tests were undertaken in samples recovered from the Glacial Deposits and 25 No. were undertaken in samples recovered from the MacDuff Formation. The full set of shear box testing results can be found in Table 20 below. Note that all samples were remoulded prior to conducting the test and the small shear box was used in every case.

Table 20. Shearbox Test Results

Exploratory Hole	Elevation of Sample (m OD)	Geological Unit	Specimen Description	Peak Cohesion Intercept c' (kPa)	Peak Angle of Friction φ' (°)
BH20	131.25	MacDuff Formation	Brown slightly silty slightly clayey fine to coarse CRUSHED ROCK.	3.0	43.5
BH22	129.74	MacDuff Formation	Brown slightly silty clayey fine to coarse CRUSHED ROCK.	14	41.0
BH25	117.53	MacDuff Formation	Brown gravelly sandy CLAY with sandstone fragments. Gravel is fine to medium.	12	35.0
BH25	116.53	MacDuff Formation	Brown clayey fine to coarse SAND and GRAVEL with pockets of silty clay.	13	37.5
BH25	113.93	MacDuff Formation	Yellowish brown gravelly very silty clayey fine to coarse SAND with sandstone fragments. Gravel is fine to coarse.	22	34.0
BH28	122.96	MacDuff Formation	Yellowish brown slightly silty clayey fine to coarse CRUSHED ROCK.	15	27.0
BH29	120.07	MacDuff Formation	Brown gravelly slightly silty very sandy CLAY. Gravel is fine to coarse.	7	41.0
BH31	119.45	MacDuff Formation	Brown clayey fine to coarse SAND and GRAVEL.	11	42.0
BH31	117.85	MacDuff Formation	Brown very gravelly fine to coarse SAND with silty clay pockets. Gravel is fine to coarse.	14	42.5
BH31	115.00	MacDuff Formation	Brown very gravelly fine to coarse SAND with silty clay pockets. Gravel is fine to coarse.	18	38.0

Exploratory Hole	Elevation of Sample (m OD)	Geological Unit	Specimen Description	Peak Cohesion Intercept c' (kPa)	Peak Angle of Friction φ' (°)
BH34	116.55	Cohesive Glacial Deposits	Brown very gravelly slightly silty very sandy CLAY with sandstone fragments. Gravel is fine to coarse.	13	34.5
BH36	110.37	Cohesive Glacial Deposits	Brown very gravelly very sandy CLAY with pockets of silt. Gravel is fine to coarse.	7.5	34.0
BH36	108.92	MacDuff Formation	Brown very gravelly very silty very sandy CLAY. Gravel is fine to coarse.	6.5	33.0
BH38	113.76	Granular Glacial Deposits	Brown slightly clayey slightly silty fine to coarse SAND and GRAVEL / CRUSHED ROCK.	6	34.5
BH39	116.97	MacDuff Formation	Brown very gravelly very sandy CLAY. Gravel is fine to coarse.	4	36.5
BH40	109.95	MacDuff Formation	Brown gravelly very sandy CLAY. Gravel is fine to coarse.	5.5	37.0
BH42	103.53	Cohesive Glacial Deposits	Brown slightly gravelly slightly clayey very sandy SILT. Gravel is fine to medium.	11	31.5
BH44	105.16	MacDuff Formation	Brown / grey very gravelly sandy CLAY. Gravel is fine to coarse.	4.5	38.0
BH46	103.01	Cohesive Glacial Deposits	Brown very gravelly very sandy CLAY. Gravel is fine to coarse.	12	32.5
TP02	150.94	MacDuff Formation	Brown clayey fine to coarse SAND and GRAVEL	17	39.0
TP04	147.21	MacDuff Formation	Brown very clayey fine to coarse SAND and GRAVEL	20	40.0
TP06	145.70	MacDuff Formation	Brown very clayey fine to coarse SAND and GRAVEL	20	42.0
TP07	141.99	MacDuff Formation	Brown very clayey fine to coarse SAND and GRAVEL	13	36.5
TP08	144.58	MacDuff Formation	Brown very clayey fine to coarse SAND and GRAVEL	17	38.0

Exploratory Hole	Elevation of Sample (m OD)	Geological Unit	Specimen Description	Peak Cohesion Intercept c' (kPa)	Peak Angle of Friction φ' (°)
TP44A	104.57	Cohesive Glacial Deposits	Brown very gravelly silty sandy CLAY. Gravel is fine to coarse.	5	34.0
TP45	102.28	MacDuff Formation	Brown very gravelly slightly silty sandy CLAY. Gravel is fine to coarse.	4	39.5
TP46	99.17	MacDuff Formation	Brown very gravelly very silty very sandy CLAY. Gravel is fine to coarse.	9.5	31.5
TP47	100.30	MacDuff Formation	Brown very gravelly very sandy CLAY. Gravel is fine to coarse.	10	37.5
TP47	98.30	MacDuff Formation	Brown slightly clayey fine to coarse SAND and GRAVEL / CRUSHED ROCK.	16	35.0
TP48	98.96	MacDuff Formation	Brown clayey fine to coarse SAND and GRAVEL / CRUSHED ROCK.	8	39.5
TP52	115.39	MacDuff Formation	Yellowish brown slightly clayey slightly silty fine to coarse SAND and GRAVEL / CRUSHED ROCK.	8	40.5

A plot of the shearbox test results is presented in Figure 12, Figure 13 and Figure 14. All figures show the line of best fit crossing the y-axis which indicates some residual cohesion. This residual cohesion will not be accounted for in the determination of geotechnical design parameters and in turn will be equal to zero.

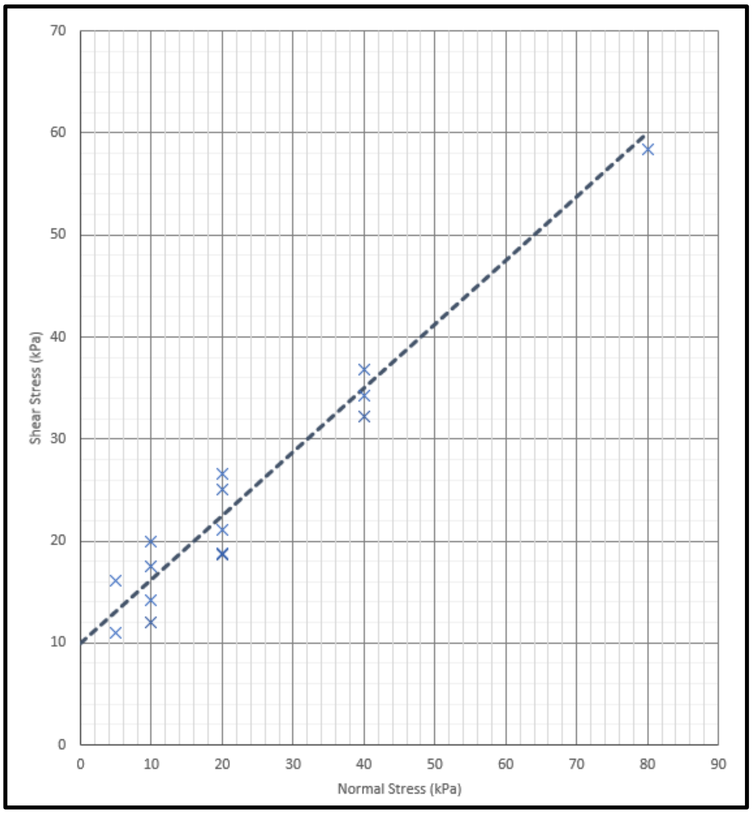

Figure 12 – Shearbox Test Results (Cohesive Glacial Deposits)

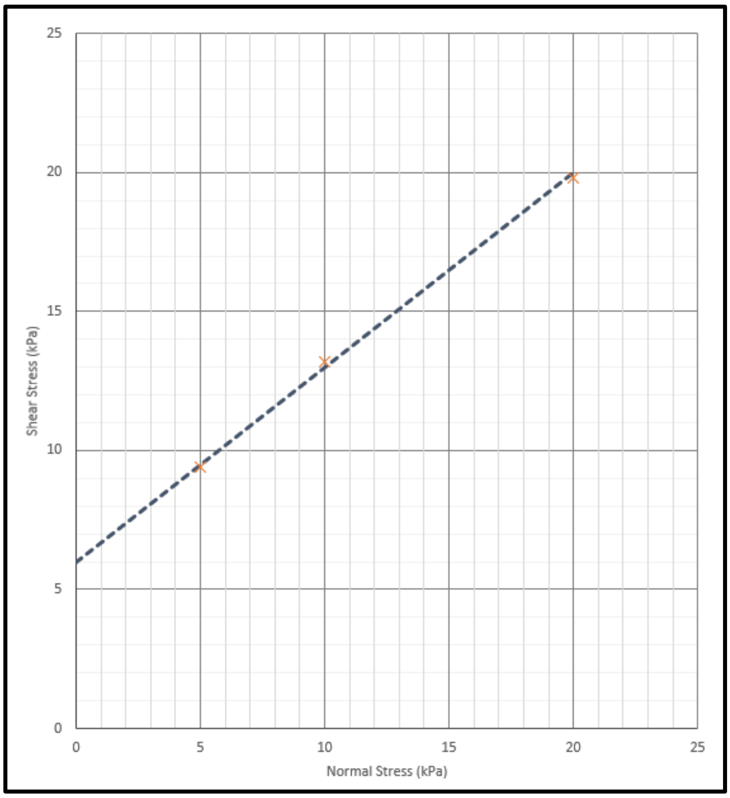

Figure 13 – Shearbox Test Results (Granular Glacial Deposits)

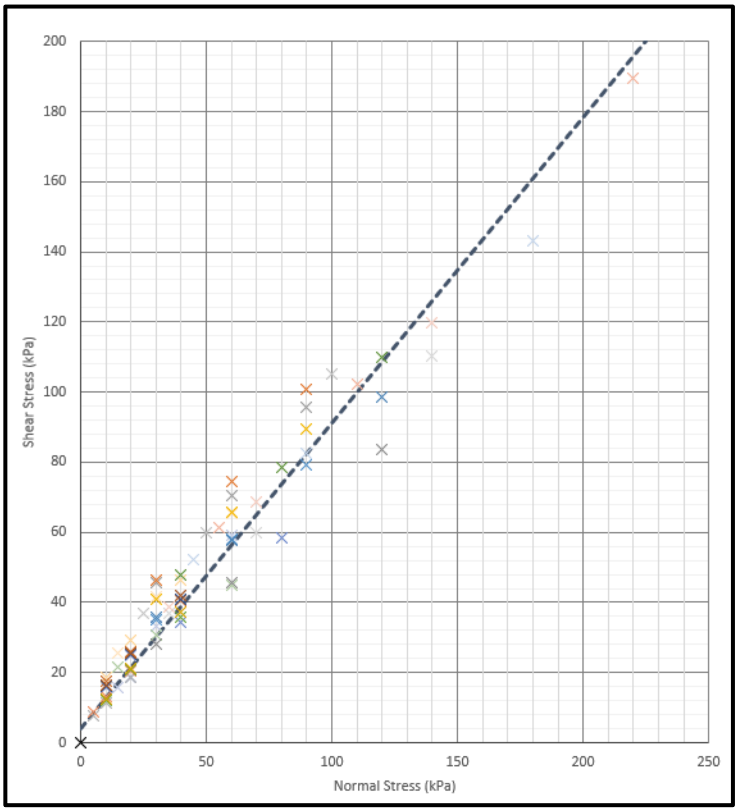

Figure 14 - Shearbox Test Results (MacDuff Formation)

5.3.9. Point Load Tests (PLT)

Point Load testing has been conducted on 17 No. rock samples. 3 No. types of PLT test were conducted across the Site (axial, diametral and irregular lump) which indicated an Is(50) value ranging between 0.0MPa and 9.1MPa, with an average value of approximately 1.5MPa. A plot of the PLT results is shown in Figure 15.

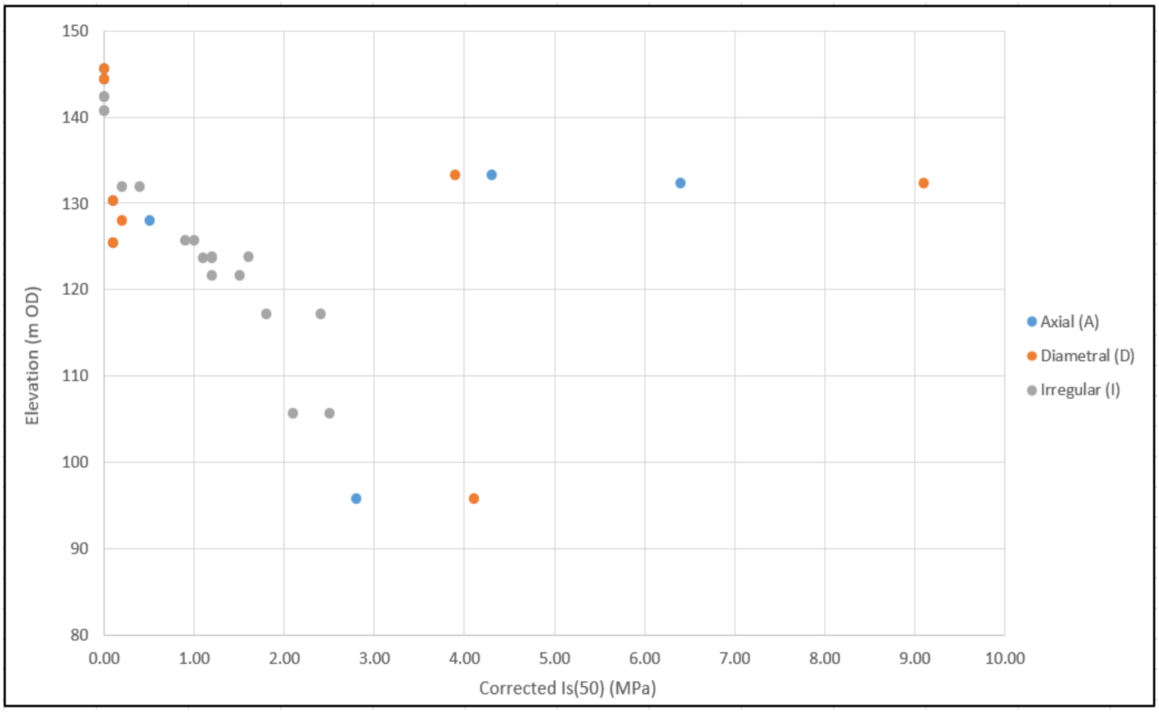

Figure 15. Plot of PLT Test Results

Typically, PLT results indicate the irregular lump tests as having the highest Is(50) values due to the rock being anticipated to have greater strength in the direction perpendicular to the bedding than any other direction. However, in this case both axial and diametral tests have the greater Is(50) values. The highest values come from BH03 where the rock is described as medium strong Conglomerate. This is the only confirmed location of Conglomerate present throughout the Site which may explain the greater Is(50) values.

5.3.10. Uniaxial Compressive Strength (UCS) Tests

UCS testing was undertaken on 1 No. rock samples from a sample of Psammite from BH08 at a depth of 7.7m bgl [140.32m OD] and indicated a UCS value of 3.86 MPa. This would classify the rock as weak, which matches the description provided on the field logs. Only one UCS test was conducted due to the non-intact nature of the rock samples providing short core lengths that are not suitable for UCS testing, hence only PLT's could be conducted.

5.3.11. Material Durability Testing

Both Los Angeles (LA) Coefficient and Magnesium Sulphate Soundness testing was undertaken on rock samples recovered from Site, to understand the durability characteristics of this material to be re-used during construction as an engineered fill.

3 No. LA coefficient results were returned from laboratory testing which recorded values in the range of 27 to 36.

6 No. Magnesium Sulphate Soundness results were returned from laboratory testing which recorded values in the range of 17% to 89%.

5.3.12. Chemical Testing

Chemical testing was conducted on 71 No. soil samples, to assess the aggressiveness of the chemical environment at the Site and assist in the determination of the design specification for buried concrete foundations.

Based on the results returned from these tests, the pH value of the soil was recorded to range between 5.3 and 8.6 and the water-soluble sulphate (SO4 2:1 extract) was recorded to range between <10mg/l (below the detection limit of laboratory testing) and 10mg/l.

5.3.13. Summary of Geotechnical Laboratory Test Results

Table 21 presents a summary of the results of the geotechnical laboratory analyses undertaken.

Table 21. Summary of Laboratory Test Results

Test Type	Test Method	Cohesive Glacial Deposits			Granular Glacial Deposits			MacDuff Formation		
		Range	Mean	Quantity	Range	Mean	Quantity	Range	Mean	Quantity
Classification	**Moisture Content (%)**	8.8 – 15.0	18.5	7.0	3.0 – 38.0	16.87	47.0	2.5 – 35.0	13.9	169.0
	Atterberg Limit									
	Liquid Limit	22.0 – 53.0	38.9	15.0	-	-	0.0	24.0 – 48.0	35.4	12.0
	Plastic Limit	16.0 – 34.0	23.2	15.0	-	-	0.0	17.9 – 29.0	23.2	12.0
	Plasticity Index	6.0 – 34.0	13.6	15.0	-	-	0.0	7.0 – 20.0	12.2	12.0
	Particle Size Distribution (PSD)									
	Cobbles (%)	10.1 – 19.7	15.67	3 of 16	3.1 – 15.7	7.47	6 of 19	0.0 – 28.7	3.8	49 of 143
	Gravel (%)	5.5 – 46.2	25.48	16 of 16	9.9 – 63.9	37.31	19 of 19	0.3 – 81.1	43.0	143 of 143
	Sand (%)	15.3 – 38	25.71	16 of 16	13.4 – 55.5	33.66	19 of 19	6.5 – 71.3	28.6	143 of 143
	Silt (%)	17.6 – 66.2	42.96	16 of 16	7.4 – 49.9	24.56	19 of 19	9.1 – 71.2	25.0	129 of 143
	Clay (%)	0.3 – 8.5	2.93	16 of 16	0.1 – 6.3	2.11	19 of 19	0.1 – 8.9	1.5	129 of 143
Compaction	**Moisture Condition Value (MCV)**	9.3 – 14.1	11.7	2.0	3.3 – 13.6	7.3	4.0	0.9 – 14.2	8.8	10.0
	4.5kg Compaction									
	Optimum Moisture Content (%)	8.8 – 16.0	13.4	7.0	-	-	0.0	7.3 – 12.0	9.7	30.0
	Maximum Dry Density (Mg/m^3)	1.62 – 1.97	1.78	7.0	-	-	0.0	1.8 – 2.1	2.0	30.0
	2.5kg Compaction									
	Optimum Moisture Content (%)	-	-	0.0	-	-	0.0	10.0	10.0	1.0
	Maximum Dry Density (Mg/m^3)	-	-	0.0	-	-	0.0	2.1	2.1	1.0
Soil Strength	**Standard Shearbox**									
	Peak Cohesion Intercept c’ (kPa)	5.0 – 13.0	9.7	5.0	6.0	6.0	1.0	3.0 – 20.0	11.5	26.0
	Peak Angle of Friction φ’ (°)	31.5 – 34.5	33.3	5.0	34.5	34.5	1.0	33.0 – 43.5	37.7	26.0
Rock Strength	**Point Load Tests Is(50)**	-	-	0.0	-	-	0.0	0.0 – 9.1	1.5	34.0
	Uniaxial Compressive Strength (UCS)	-	-	0.0	-	-	0.0	3.9	3.9	1.0

5.4. Derived Geotechnical Parameters

The results of both in-situ and laboratory testing data have been used to derive characteristic values of geotechnical design parameters that may be considered within the design of the proposed substation development.

5.4.1. Topsoil

It is recommended that all Topsoil is removed from the area of proposed development and not considered as engineering materials, or founding strata, during the construction works. As such, engineering properties have not been assigned to this strata layer.

5.4.2. Cohesive Glacial Deposits (Till)

Unit weight

Where given, the Cohesive Glacial Deposits are generally described as soft to firm sandy gravelly Clay with some localised pockets of soft to stiff sandy gravelly Silt. Hence, using typical values of natural density for Glacial Deposits (Till), a characteristic design bulk unit weight of 20-21 kN/m^3 can be assigned (Correlations of Soil Properties, Carter & Bentley).

Undrained Shear Strength Parameters

For the Cohesive Glacial Deposits present throughout the Site, an estimation of the undrained shear strength (C_u) has been developed through a correlation of SPT N_{60} values and undrained shear strength values developed by Stroud (1989):

$$c_u = f_1 N_{60}$$

The values of the multiplication factor, f_1, from Stroud (1989) are taken from Figure 16.

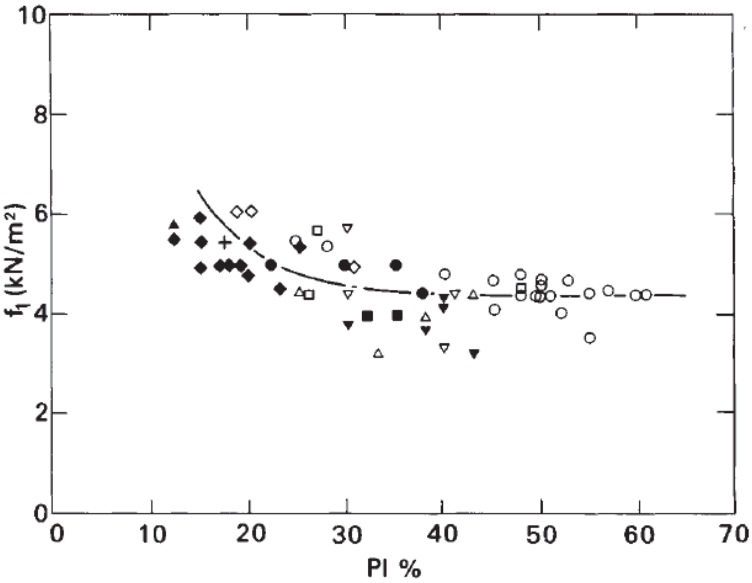

Figure 16. Multiplication Factor f_1 used in Undrained Shear Strength determination. Adapted from Stroud (1989)

Adopting a characteristic plasticity index range of 6.0 – 34.0 for the Cohesive Glacial Deposits throughout the Site, gives a multiplication factor range of 4.5 – 6.5. Using a characteristic SPT N_{60} value of 35 provides an anticipated undrained shear strength range of 155 – 225 kPa.

Drained Shear Strength Parameters

An estimation of the effective angle of shearing resistance for the Cohesive Glacial Deposits on Site is estimated using a relationship established by Sorensen and Okkels (2013) as shown in Figure 17.

$$\boxed{\phi' = 44 - 14\ \log(\mathrm{PI})}$$

Figure 17. Estimation of Effective Angle of Shearing Resistance for Cohesive Soils. Adapted from Sorensen and Okkels (2013)

Adopting a plasticity index range of 6.0 – 34.0 as before, provides a range of effective angle of shearing resistance of φ =22.5 - 33.1 °. Furthermore, the drained shear strength of Clay in effective stress terms should be considered frictionless, so the effective cohesion, c' = 0.

The shear box test data may also be used to derive drained strength parameters, although it should be noted that the peak angle of shearing resistance and peak drained cohesion are provided by this analysis method, rather than characteristic values. For the Cohesive Glacial Deposits, 5 No. samples were tested in the shear box and provided a peak angle of friction in the region of φ = 31.5 – 34.5 ° and a peak effective cohesion, c', in the region of 5.0 – 13.0 kPa.

Stiffness – Young's Modulus

The Young's Modulus (E) can be estimated from the results of SPT N_{60} values for the Cohesive Glacial Deposits, using the correlation presented in Stroud (1989):

$$E = 0.9N_{60}$$

Adopting a characteristic SPT N_{60} value of 35 – 40 for the cohesive soils throughout the Site would provide an anticipated Young's Modulus range of 31 - 36 MPa.

Permeability

The coefficient of permeability of the soil, k, can also be correlated from the laboratory test data, using the results of PSD testing. The correlation developed by Kozeny-Carman Hazen (1927) allows the use of grain size data D10 (corresponding to the sieve size at which 10% of material passes) to estimate the permeability of the soil. The plot of permeability values for the Cohesive Glacial Deposits is presented in Figure 18.

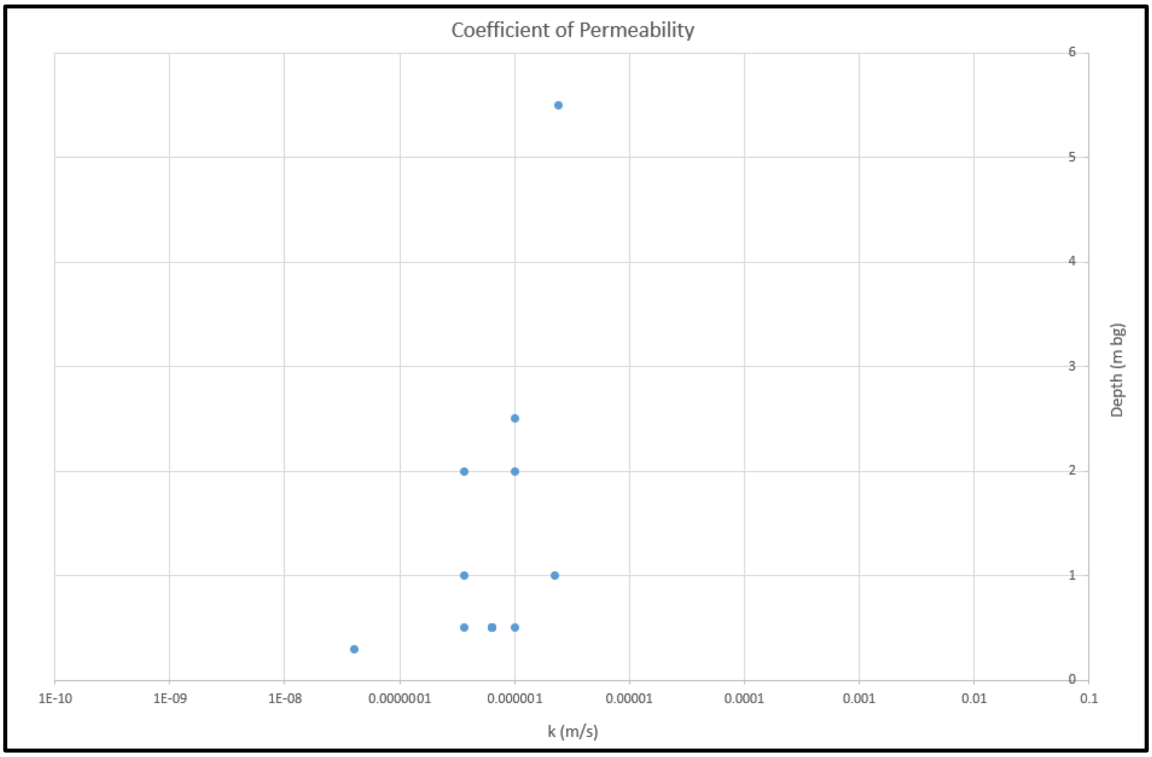

Figure 18. Plot of k values for Cohesive Glacial Deposits

Figure 18 shows a range of soil permeability values of approximately 1x10^{-8} to 1x10^{-6} ms^{-1}. This would indicate the Cohesive Glacial Deposits to be of a low to very low permeability, meaning natural drainage properties of these soils may be poor.

These values generally align with typical values anticipated for the material encountered at the Site based on the descriptions provided within the exploratory hole logs, as shown in Figure 19.

Table 2.1 Coefficient of permeability (m/s) (BS 8004: 1986)

1 to 10^{-1}	10^{-1} to 10^{-4}	10^{-4} to 10^{-7}	10^{-7} to 10^{-10}
Clean gravels	Clean sands and sand–gravel mixtures	Very fine sands, silts and clay-silt laminate	Unfissured clays and clay-silts (>20% clay)
	Desiccated and fissured clays	Desiccated and fissured clays	

Figure 19. Typical Coefficient of soil permeability values, extract from Craigs Soil Mechanics 7th Edition

5.4.3. Granular Glacial Deposits (Till)

Unit weight

Where given, the Granular Glacial Deposits are generally described as fine to coarse sandy Gravel with some localised pockets of medium dense silty gravelly Sand. Hence, using typical values of natural density for this soil type, a characteristic design bulk unit weight of 18-20 kN/m^3 can be assigned (Correlations of Soil Properties, Carter & Bentley).

Shear Strength Parameters

For the Granular Glacial Deposits at the Site, an estimation of the internal angle of friction has been developed through correlation developed by Peck et al, using the chart presented in Figure 20. Observing the scatter of SPT N_{60} values for the Granular Glacial Deposits, a characteristic SPT value of between N_{60} = 35 - 45 would be considered appropriate for design purposes, and therefore the anticipated non-peak angle of shearing resistance for the soil is estimated to be in the region of φ = 37 - 40°.

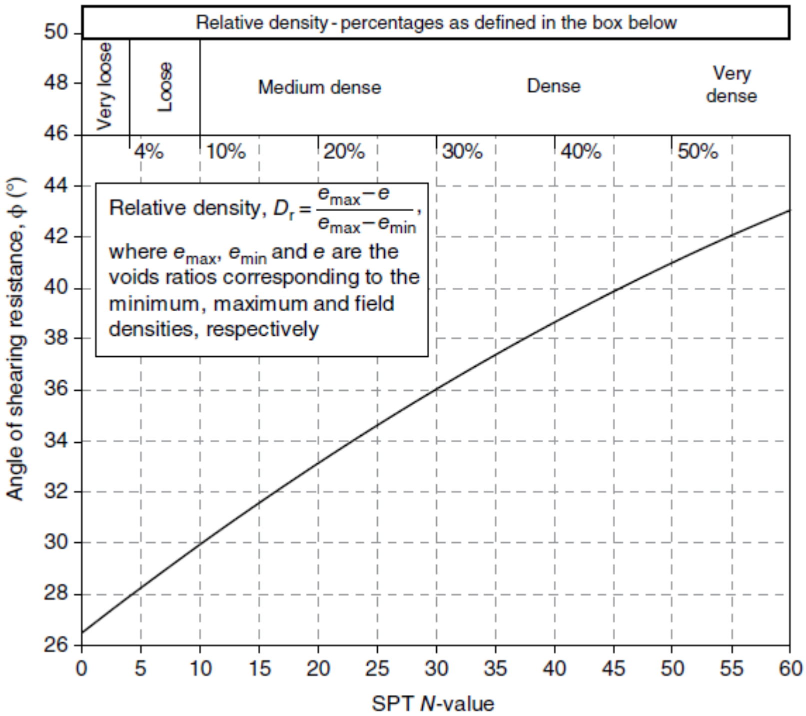

Figure 20. Estimation of the Friction Angle of Granular Soils from SPT results. Adapted from Peck et al. (1974)

The shear box test data may also be used to derive drained strength parameters, although it should be noted that the peak angle of shearing resistance and peak drained cohesion are provided by this analysis method, rather than characteristic values. However, for Granular Glacial Deposits only 1 No. sample was tested in the shearbox which produced a peak friction angle of 34.5° and a peak cohesion of 6kPa.

Stiffness – Young's Modulus

The drained Young's Modulus (E') can be estimated from the results of SPT N_{60} values for the Granular Glacial Deposits, using correlations presented in CIRIA Report C143 (Clayton. C.R.I., 1995), valid for both over-consolidated and normally consolidated sands and gravels.

It is considered likely that the Granular Glacial Deposits at the Site should be treated as a normally consolidated material, and hence in reference to Figure 20, a value of E'/N of 1 has been assumed.

Adopting a characteristic SPT N_{60} value of between 35 – 45 blows, this allows for the derivation of a characteristic Young's Modulus value of E' = 35 – 45 MPa.

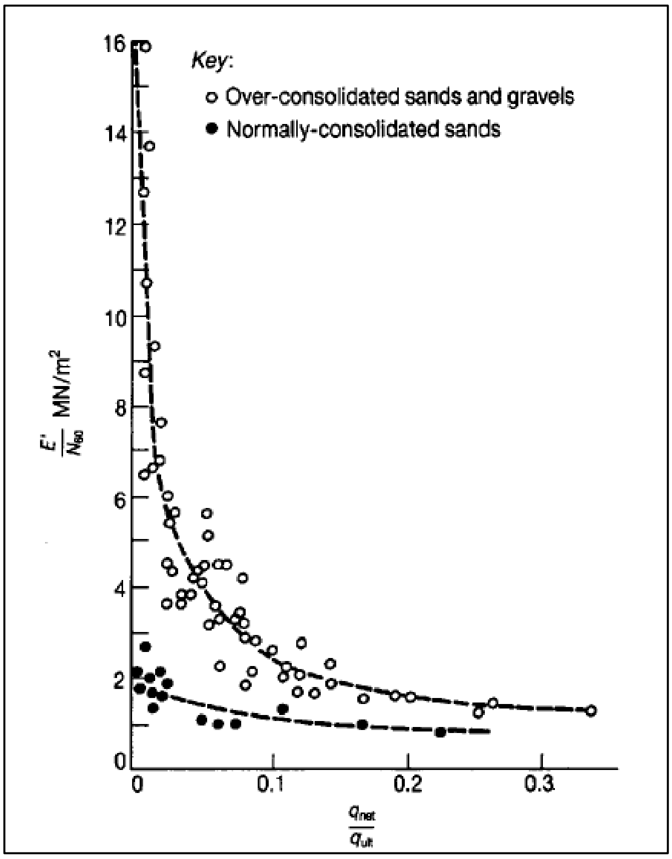

Figure 21. Relationship between Stiffness, Penetration Resistance and Degree of Loading for Sand (after Stroud, 1989)

Permeability

The coefficient of permeability, k, is derived from the correlation developed by Kozeny-Carman Hazen (1927) as before in Section 5.4.2. The plot of k values for the Granular Glacial Deposits are shown in Figure 22.

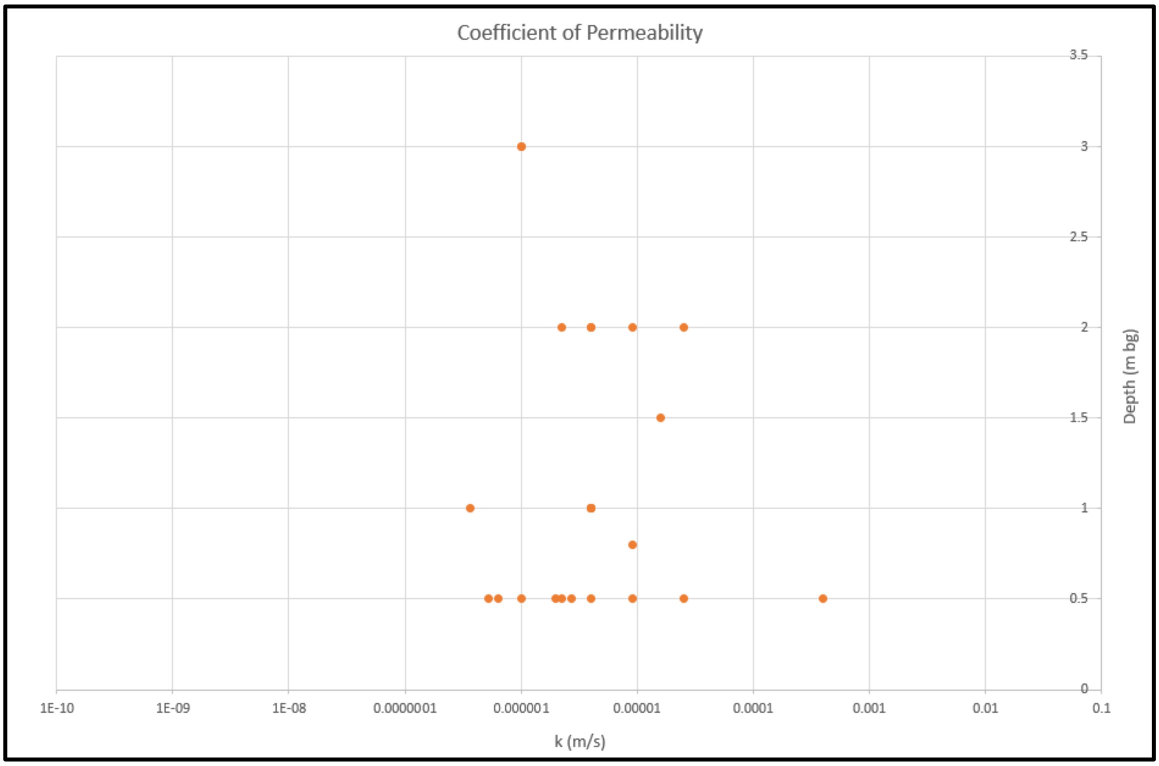

Figure 22. Plot of k values for Granular Glacial Deposits

Figure 22 shows a range of soil permeability values of approximately $1x10^{-7}$ to $1x10^{-4}$ ms^{-1}. This would indicate the Granular Glacial Deposits to be of a low permeability meaning natural drainage properties of these soils may be poor.

While these samples were classified as a granular material, a permeability value of $1x10^{-7}$ ms^{-1} is a value more typically aligned with a cohesive material. A review of the PSD charts for these samples do show a granular grading but with a high percentage of fines (approx. 30%) which will affect the permeability value and, in this case, reduce the value.

5.4.4. Weathered and Non-Intact Rock

The overall ground model for the Site, described in Section 4.3, shows a layer of either Weathered or Non-Intact Rock underlying the Glacial Deposits. Weathered Rock should be considered as a Granular material for design purposes and the following section will explain the derivation of geotechnical characteristic parameters for this material.

Unit Weight

Where given, the weathered rock is described as either Psammite, Pelite or Semi-Pelite recovered as a medium dense to dense Gravel. Using typical natural density values the weathered and non-intact rock, a characteristic bulk density value can be given as 19 – 22 kN/m^3 (Correlations of Soil Properties, Carter & Bentley).

Shear Strength Parameters

For the Weathered and Non-Intact Rock at the Site, an estimation of the internal angle of friction has been developed through correlation developed by Peck et al, using the chart presented previously in

Figure 18. Observing the scatter of SPT N_{60} values for the Weathered and Non-Intact Rock a characteristic SPT value of between N_{60} = 45 - 55 would be considered appropriate for design purposes, and therefore the anticipated angle of shearing resistance for the Weathered and Non-Intact Rock is estimated to be in the region of φ = 40 - 42 °.

Stiffness – Young's Modulus

The drained Young's Modulus (E') can be estimated from the results of SPT N_{60} values for the Weathered and Non-Intact Rock, using correlations presented in CIRIA Report C143 (Clayton. C.R.I., 1995), valid for both over-consolidated and normally consolidated sands and gravels.

It is considered likely that the Weathered and Non-Intact Rock at the Site should be treated as a normally consolidated material, and hence in reference to Figure 13, a value of E'/N of 1 has been assumed. Adopting a characteristic SPT N_{60} value of between 45 – 55 blows, this allows for the derivation of a characteristic Young's Modulus value of E' = 45 – 55 MPa.

Permeability

The coefficient of permeability, k, is derived from the correlation developed by Kozeny-Carman Hazen (1927) as before in Section 5.4.2 and Section 5.4.3. The plot of k values is shown below in Figure 23.

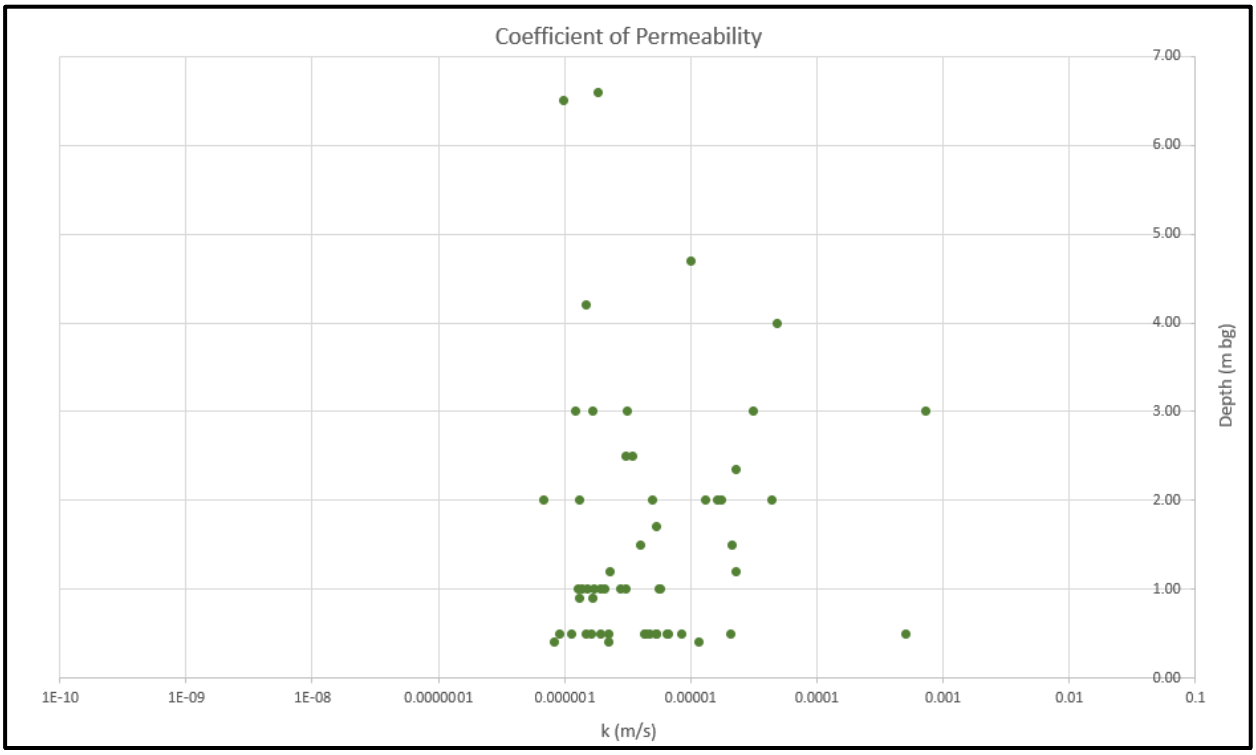

Figure 23. Plot of k values for Weathered Rock

Figure 23 shows the range of k values for weathered rock is $1x10^{-4} – 1x10^{-7}$ ms^{-1}. This would indicate the Weathered Rock to be of a medium to low permeability meaning natural drainage properties of these soils may be poor.

While these samples were all classified as a granular material, a permeability value of $1x10^{-7}$ ms^{-1} is a value more typically aligned with a cohesive material. A review of the PSD charts for these samples do show a granular grading but with a high percentage of fines (approx. 30%) which will affect the permeability value and, in this case, reduce the value.

5.4.5. Pelite

Unit weight

No direct testing to determine the unit weight of the Pelite bedrock was undertaken during the GI works. A value of 23 kN/m^3 has been considered adequate for design purposes at the time of writing, as this value aligns with anticipated values[8] for this material.

Rock Strength (UCS)

PLTs were undertaken to infer a characteristic rock strength value of the Pelite present on Site. To produce a comparable plot of data, the correlation of UCS = 20 x Is(50) has been used to correlate the PLT test results with UCS data. No Site-specific correlation factor could be produced due to the lack of UCS testing so an estimate of 20 was used[9].

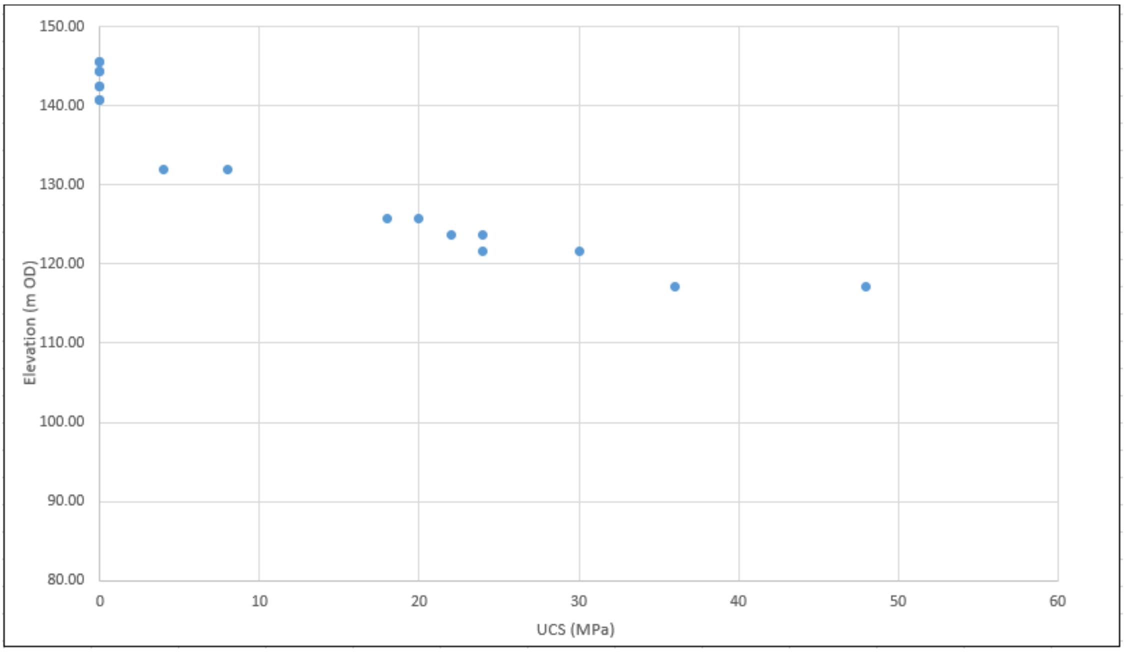

Figure 24. Plot of PLTs converted to Equivalent UCS

It can be seen from Figure 24 that the UCS of the Pelite increases with depth. An average UCS value of 13MPa is taken for the Pelite, when considering correlated values.

[8] Barnes, G,; Soil Mechanics: Principles and Practice, 4th Edition, Macmillan Education, England, 2016
[9] Ameratunga, J. Sivakugan, N. Das, B. M. (2016). Correlations of soil and rock properties in geotechnical engineering. Springer. India. pp207-223.

Rock Mass Stiffness

The Young's Modulus of the rock mass may be required to inform foundation design, and this can be determined based on properties and characteristics of the bedrock obtained during the GI works. The Rock Mass Stiffness can be calculated as a function of the:

- Rock Strength, q_{uc} (UCS) determined form laboratory testing;
- Modulus Ratio M_r, determined based on the rock type after Hock and Deiderichs (2006),
- Mass Factor, J, determined based on the results of the Rock Quality Designation (RQD) value presented on rock core logs, typically >90%, using guidance provided within Table 53.15 of the ICE Manual of Geotechnical Engineering.

Adopting a characteristic rock strength of 10MPa, a modulus ratio of 250 and a mass factor of 0.2, the stiffness modulus of the rock mass can be calculated to be 500MPa.

As this method is only suitable for rock where RQD > 90%, the stiffness modulus is only suitable for intact rock and can not be used for non-intact or weathered rock.

5.4.6. Psammite

Unit Weight

1 No. UCS test was conducted on a Psammite sample which indicated a bulk density of 24.4kN/m^3. A value of 23kN/m^3 has been considered adequate for design purposes at the time of writing, as this value aligns with anticipated values[8] for this material.

Rock Strength (UCS)

A combination of PLTs and UCS tests are used to classify the strength of Psammite. A factor of 20 is used as before to convert the Is(50) value into UCS a value.

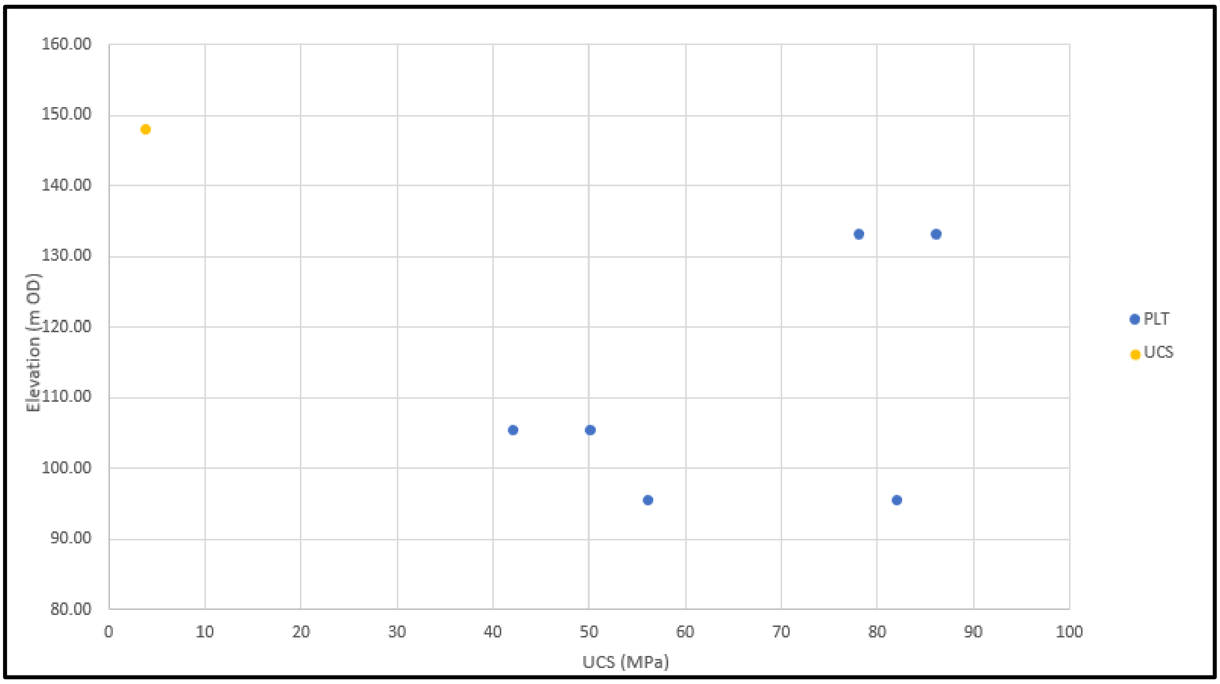

Figure 25. Plot of UCS and PLTs converted to Equivalent UCS

It can be seen from Figure 25 that no particular trend of the UCS of the Psammite is shown with depth from the information available. An average UCS value of 57 MPa is taken for the Psammite, when considering both direct testing and correlated values.

Rock Mass Stiffness

The Young's Modulus of the rock mass may be required to inform foundation design, and this can be determined based on properties and characteristics of the bedrock obtained during the GI works. The Rock Mass Stiffness can be calculated as a function of the:

- Rock Strength, q_{uc} (UCS) determined form laboratory testing;
- Modulus Ratio M_r, determined based on the rock type after Hock and Deiderichs (2006),
- Mass Factor, J, determined based on the results of the Rock Quality Designation (RQD) value presented on rock core logs, typically >90%, using guidance provided within Table 53.15 of the ICE Manual of Geotechnical Engineering.

Adopting a characteristic rock strength of 50MPa, a modulus ratio of 250 and a mass factor of 0.2, the stiffness modulus of the rock mass can be calculated to be 2500MPa.

As this method is only suitable for rock where RQD > 90%, the stiffness modulus is only suitable for intact rock and can not be used for non-intact or weathered rock.

5.4.7. Semi-Pelite

Unit Weight

No UCS testing of samples in Semi-Pelite were conducted due to the non-intact nature of the cores providing unsuitable samples for UCS testing. Hence, no bulk density was unable to be obtained through laboratory testing. A value of 23 kN/m^3 has been considered adequate for design purposes at the time of writing, as this value aligns with anticipated values for this material[8].

Rock Strength (UCS)

PLT tests were undertaken to classify the strength of Semi-Pelite. A factor of 20 is used as before to convert the Is(50) value into UCS a value.

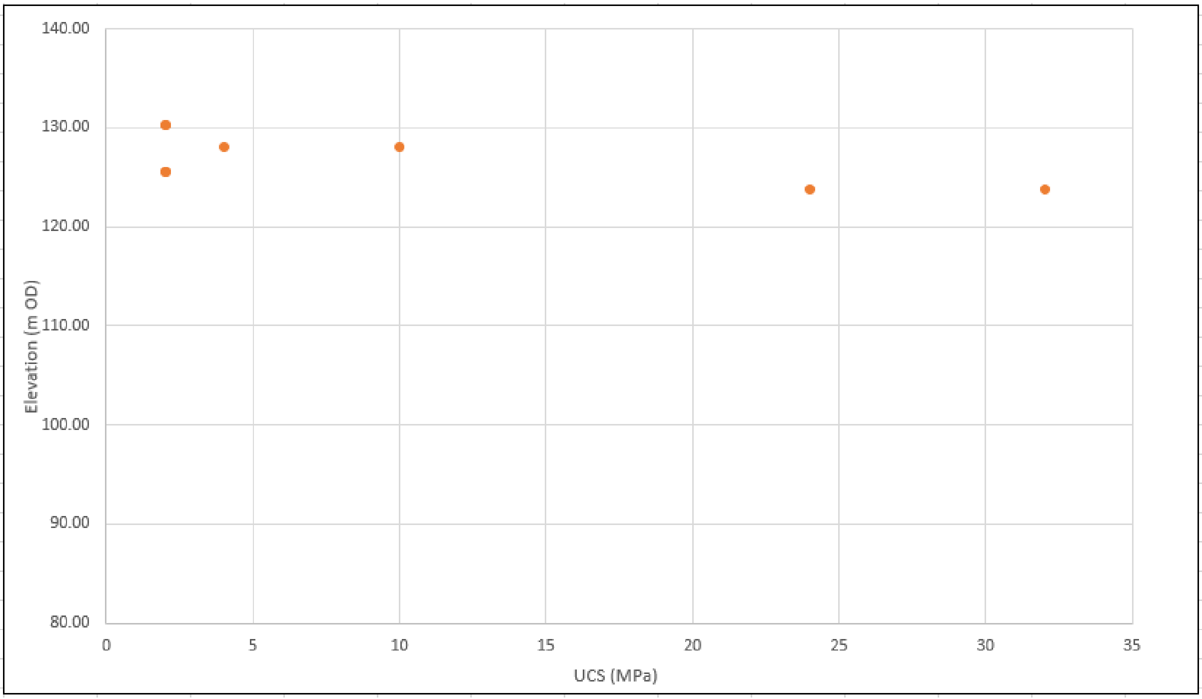

Figure 26. Plot of PLTs converted to Equivalent UCS

It can be seen from Figure 26 that the UCS of the Semi-Pelite is shown to increase with depth. An average UCS value of 10 MPa is taken for the Semi-Pelite, when considering correlated values.

Rock Mass Stiffness

The Young's Modulus of the rock mass may be required to inform foundation design, and this can be determined based on properties and characteristics of the bedrock obtained during the GI works. The Rock Mass Stiffness can be calculated as a function of the:

- Rock Strength, q_{uc} (UCS) determined form laboratory testing;
- Modulus Ratio M_r, determined based on the rock type after Hock and Deiderichs (2006),
- Mass Factor, J, determined based on the results of the Rock Quality Designation (RQD) value presented on rock core logs, typically >90%, using guidance provided within Table 53.15 of the ICE Manual of Geotechnical Engineering.

Adopting a characteristic rock strength of 10MPa, a modulus ratio of 250 and a mass factor of 0.2, the stiffness modulus of the rock mass can be calculated to be 500MPa.

As this method is only suitable for rock where RQD > 90%, the stiffness modulus is only suitable for intact rock and cannot be used for non-intact or weathered rock.

5.4.8. Summary of Geotechnical Parameters

A summary of derived geotechnical parameters is presented in below.

Table 22. Summary of Derived Characteristic Geotechnical Parameters

Parameter	Strata					
	Cohesive Glacial Deposits	Granular Glacial Deposits	Weathered and Non - Intact Rock	Pelite	Psammite	Semi-Pelite
Bulk unit weight, γ (kN/m^3)	20.0 – 21.0	18.0 – 20.0	19.0 – 22.0	23.0	24.0	23.0
Friction angle, φ (°)	22.5 – 31.5	37.0 – 40.0	40.0 – 42.0	N/A	N/A	N/A
Drained cohesion, c' (kPa)	0.0	0.0	0.0	N/A	N/A	N/A
Undrained Shear Strength (kPa)	155.0 – 180.0	N/A	N/A	N/A	N/A	N/A
Drained Young's Modulus, E' (MPa)	31.0 – 36.0	35.0 – 45.0	45.0 – 55.0	500	2500	500
Permeability, k (ms^{-1})	$1x10^{-8}$ – $1x10^{-6}$	$1x10^{-7}$ – $1x10^{-4}$	$1x10^{-7}$ – $1x10^{-4}$	Impermeable	Impermeable	Impermeable
USC (MPa)	N/A	N/A	N/A	10.0	50.0	10.0

5.5. Material Re-usability Assessment

The following section has been developed to understand the potential of the on-site materials to be re-used as engineered fill during the proposed construction works. The assessment has been undertaken by reviewing the existing properties of the on-site soil and rock, against the requirements of construction materials outlined within the MCHW SHW Series 600.

5.5.1. Material Grading

The particle size distribution (PSD) data obtained from samples taken within the superficial material across Site have been used to assess the materials potential for re-use by comparing the grading envelopes against those required of typical engineering fill materials, including Class 6F2 (Capping), Class 1 (General Granular Fill) and Class 2 (General Cohesive Fill).

The results of the PSDs from the Granular Glacial Deposits and Weathered and Non-Intact Rock from the MacDuff Formation are plotted against the grading envelopes for Class 6F2 and Class 1 Fills outlined within Table 6/2 of the SHW – 'Grading Requirements for Acceptable Earthworks Materials' are presented in Figure 27.

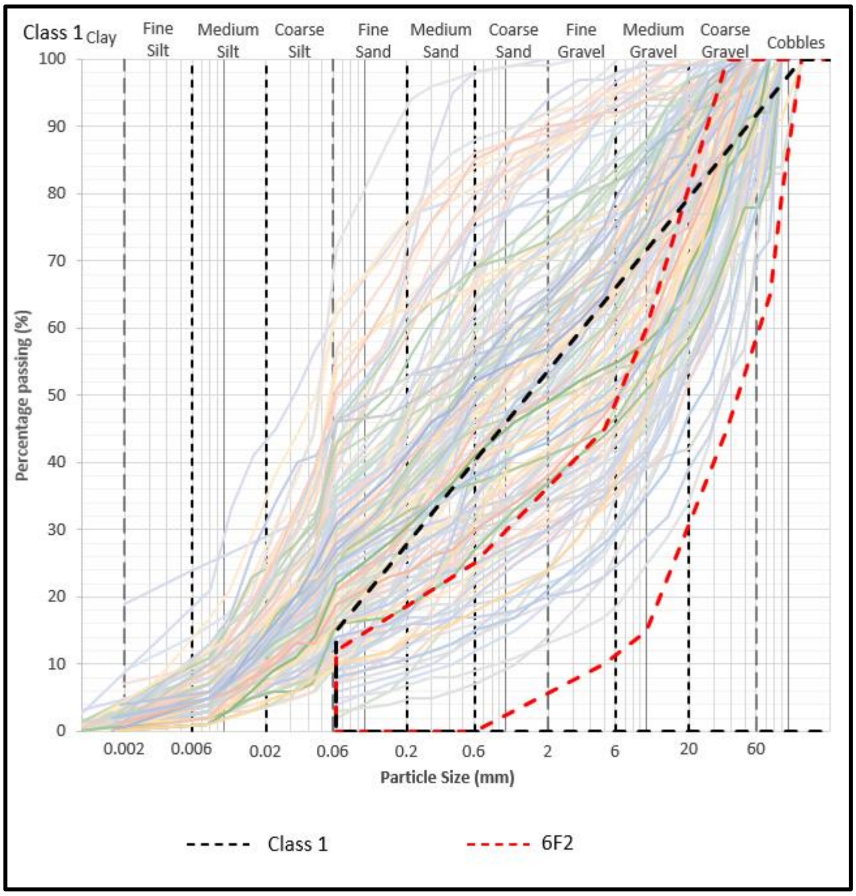

Figure 27. Plot of PSD Results compared to typical Granular Engineering Fills

The data from Figure 27 indicates that the majority of the PSD curves for the granular superficial material lie out with the envelope for the in-situ material to be immediately classified as either Class 1 or Class 6F2 engineering fill, due to high fines content. As such, earthworks processing would be required in order to develop most of the on-site soils into an engineering fill for use during construction, which would require the introduction of larger particles. On Site sourcing of these larger particles may be possible through excavation and reprocessing of the Site bedrock, which could be sourced from areas of deep cut within the west of the site.

The results of the PSDs from the Cohesive Glacial Deposits are plotted against the grading envelopes for Class 2 Fills outlined within Table 6/2 of the SHW – 'Grading Requirements for Acceptable Earthworks Materials' are presented in Figure 28.

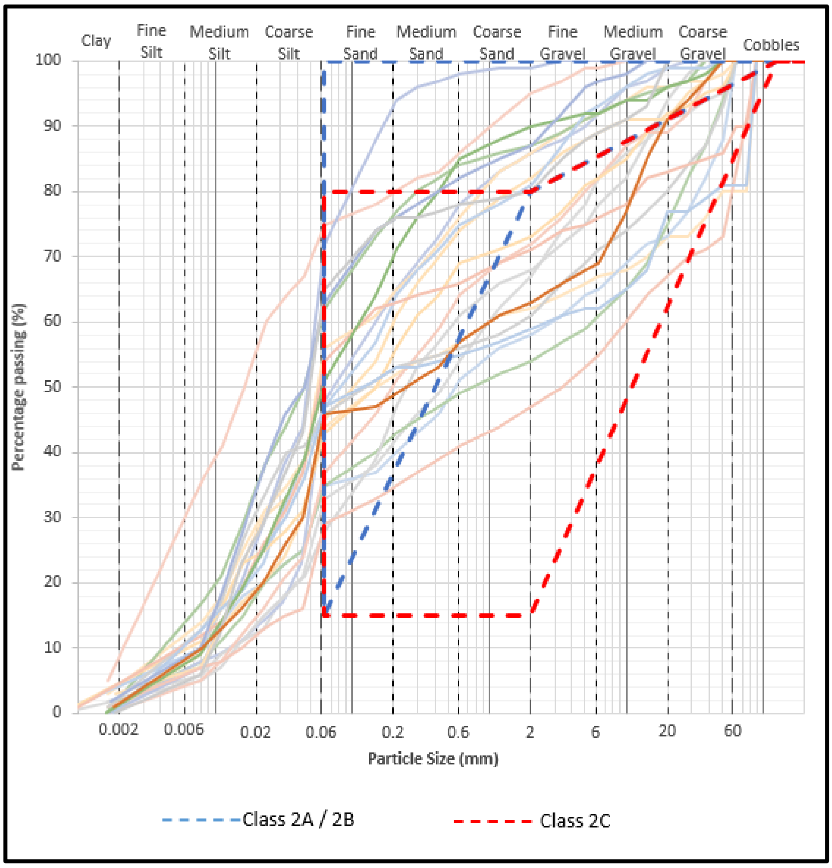

Figure 28. Plot of PSD Results compared to typical Cohesive Engineering Fills

The data from Figure 28 shows that the majority of the PSD curves for the Cohesive Glacial Deposits fall within the envelope for the in-situ material to be immediately classified as Class 2 general fill.

5.5.2. Material Durability

Further criteria required for a material to be considered for re-use as an engineering fill are the durability characteristics. Material durability testing comprising both LA Coefficient testing and Magnesium Sulphate Soundness testing were undertaken as part of the GI. The results of the LA Coefficient tests have been plotted against the limits for a Class 6F2, Class 1C, Type 1 and a Concrete Aggregate in Figure 29, and the results of the Magnesium Sulphate Soundness testing against the limits for a Type 1 Aggregate in Figure 30.

The data indicates that the durability characteristics of the rock at the Site fall within the acceptable limits for reusing this material as a Class 6F2, Class 1C and a Concrete Aggregates. However, only Pelite was tested and no LA Coefficient testing was conducted on Semi-Pelite or Psammite samples. Magnesium Sukphate Soundness tests were conducted on all three rock types encountered throughout the Site, and the test results show many of the Magnesium Sulphate Soundness test results fall above the upper bound limit for consideration of this material to be reused as a Type 1 fill.

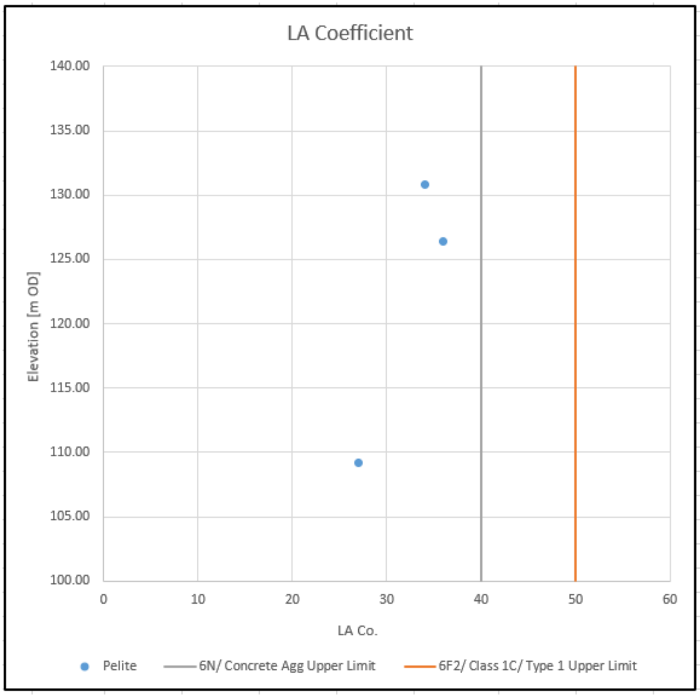

Figure 29. LA Coefficient Test Results

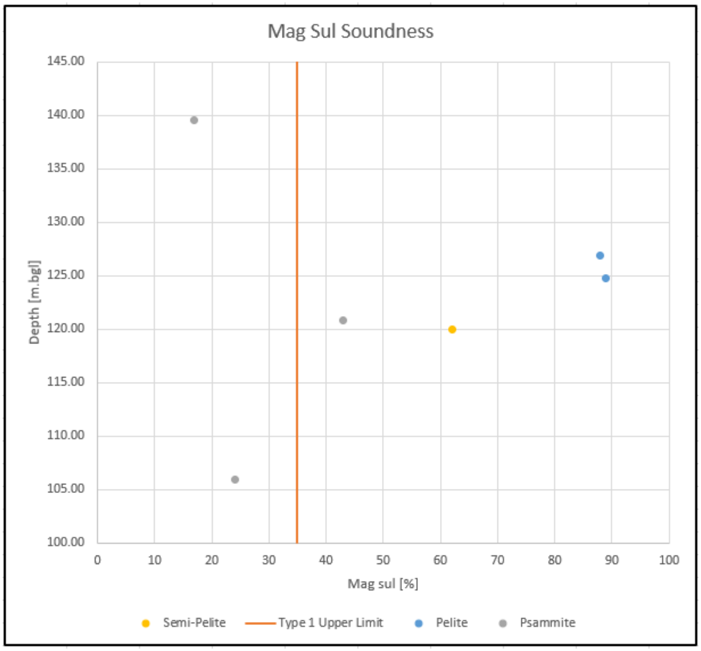

Figure 30. Aggregate Soundness Test Results

5.5.3. Rock Excavatability

Pending confirmation of the proposed finished level of the substation platform at detailed design, Site levelling works may involve considerable excavation through rock, based on the preliminary information provided so far. As such, an assessment of rock excavatability has been undertaken based on the samples obtained from the rotary core drilling into bedrock.

Figure 31 below presents analysis of the rock strength data against the rock fracture index to provide an indication of the likely required methods of excavation required, based on a maximum excavatability chart developed by Pettifer and Fookes[10].

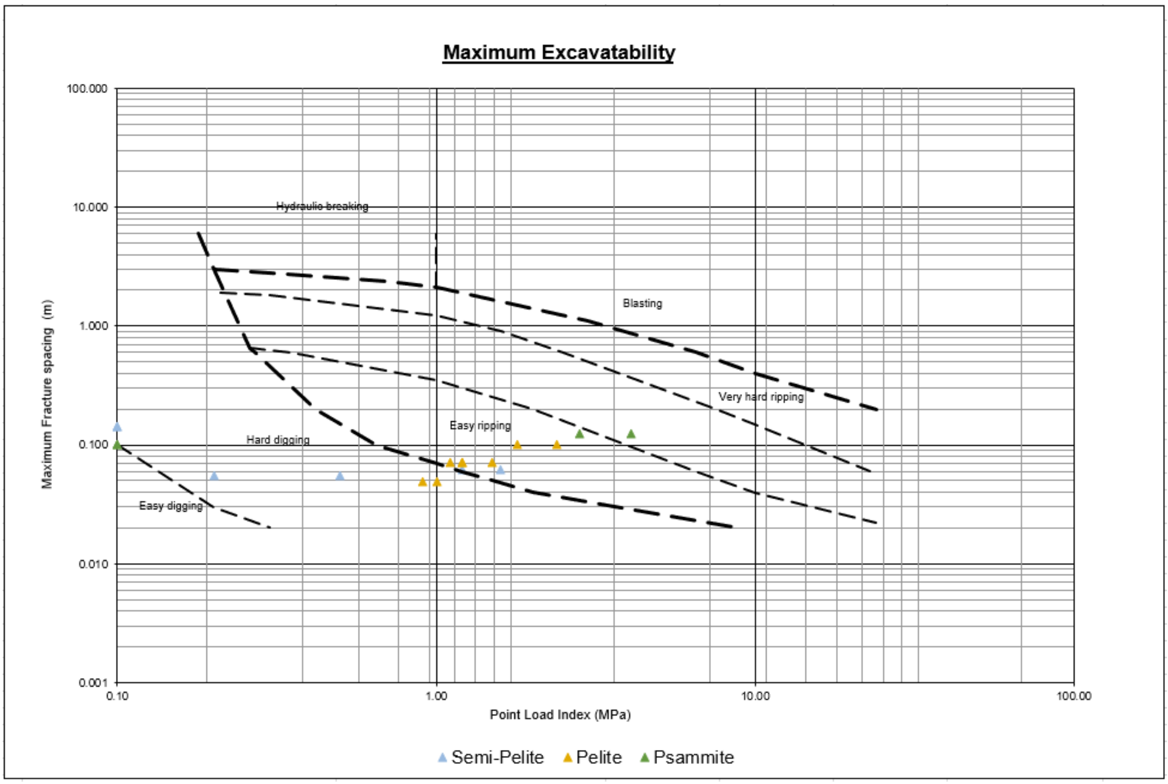

Figure 31. Excavatability Chart

Most of the plotted data can be observed to fall within the areas of the chart that would indicate a requirement for Hard digging and easy ripping in order to excavate the bedrock at the Site. Blasting is not anticipated to be required for the bedrock at the Site.

5.6. Buried Concrete

Soil and rock samples recovered during the GI were analysed for pH levels and the concentrations of water-soluble sulphate (SO_4) to assess the aggressiveness of the Site soils and assist in the determination of the design specification for buried concrete foundations.

An assessment of this has been undertaken as part of this GIR in accordance with guidance outlined within BRE Special Digest 1 – Concrete in Aggressive Ground. Based on the results of the chemical laboratory testing, the characteristic values adopted within the assessment are as follows:

- Characteristic pH level (soils and rock) of 5.51;
- Characteristic pH level (groundwater) of 6.20;
- Characteristic Water Soluble SO_4 level (soils and rock) of 10.0mg/l;
- Characteristic Water Soluble SO_4 level (groundwater) of 21.5mg/l.

[10] Pettifer and Fookes (1994) A revision of the graphical method for assessing the excitability of rock. Q J Eng Geol 27:145-164

Based on the above test results and BRE SD1 guidance Table C1, the recommended Design Sulphate Class for the Site is DS-1 and the corresponding ACEC classification is AC-1. It is recommended all buried concrete should be designed in accordance with the minimum requirements of this BRE classification.

6. Geoenvironmental Assessment

6.1. Introduction

This section provides a geoenvironmental assessment of the groundwater conditions, based upon the findings of the ground investigation undertaken between 21/08/2023 and 20/10/23 and a subsequent programme of gas / groundwater monitoring commencing in November 2023 and is currently ongoing at the time of reporting. The overall aims of this geoenvironmental assessment are to:

- Provide an evaluation of potentials risks the water environment from the presence of groundwater contaminants;
- Provide a preliminary assessment of ground gas risks;
- Provide a preliminary assessment of the potential for groundwater at the Site to be used for potable water supply.

The available information has been used to develop a conceptual site model for the site based upon available, however it should be noted that a Tier 1 Contamination Risk Assessment has not been undertaken for the site.

This assessment has been undertaken in general accordance with:

- Land Contamination and Development, published by Environmental Protection Scotland
- BS10175:2011+A2:2017 Investigation of Potentially Contaminated Sites – Code of Practice

6.2. Summary of Contamination Laboratory Analysis

No environmental contaminant testing was conducted on soil samples across the Site, only groundwater samples were tested.

Groundwater samples were collected from all 10 No. standpipes on 05/12/23, then variably scheduled for a suite of analysis comprising:

- 10 No. test samples for pH, Soluble Sulphate (SO_4), Cyanide (total), Phenols (screen), Boron, Arsenic, Cadmium, Chromium (total), Copper, Lead, Mercury, Nickel, Zinc, Total TPH and speciated PAH (USEPA 16).

6.3. Risks to the Water Environment

6.3.1. Overview and Selection Criteria

A Tier 2 Generic Quantitative Risk Assessment for the groundwater at the Site has been undertaken based on the results of the ground investigation, to assess potential contamination risks to the water environment and provide a preliminary assessment of the potential for shallow groundwater within the superficial deposits at the Site to be used as a potable supply. The risk assessment has been undertaken in general accordance with Land Contamination and Development and comprises the comparison of groundwater laboratory analysis derived from the ground investigation with published criteria.

The principal water environment receptor to groundwater at the Site is the Burn of Greens, which lies approximately 250m to the east of the Site. The geological stratum underlying the Site is classified by SEPA as a low productivity aquifer.

On the basis of the above the following criteria are considered to be appropriate for the comparison of the groundwater laboratory analysis results:

- Environmental Quality Standards for discharge to surface waters (WAT-SG-53) to assess potential risks to the water environment;
- Water quality standards prescribed by Scottish Water for the evaluation of potable water supply potential.

6.3.2. Summary of Groundwater Sampling

One round of groundwater sampling was undertaken following completion of the ground investigation, comprising:

- Sampling of BH01, BH08, BH10, BH18, BH24, BH28, BH31, BH36, BH42 and BH46 on 05/12/23.

6.3.3. Tier 2 Generic Quantitative Risk Assessment – Groundwater

Inorganic Contaminants

A comparison of groundwater contaminant concentrations with the appropriate screening criteria is provided in Table 23 below.

Table 23. – Summary of Inorganic Groundwater Analysis and Comparison with Screening Criteria

Contaminant	No. of Samples	Concentration Range (ug/l)	Screening Criteria (µg/l)		Is Screening Criteria Exceeded? (No. of Exceedances)
			EQS	WQS^	
Arsenic	10.0	<0.15 – 0.47	50*	-	No
Boron	10.0	<10.0 – 22.0	2,000*	-	No
Cadmium	10.0	<0.02 – 0.22	0.09*	-	Yes (2)
Chromium (Total)	10.0	<0.2 – 2.5	4.7*	-	No
Copper	10.0	0.9 – 10.0	1***	2,000	Yes (9)
Lead	10.0	<0.2 – 1.1	1.2***	10	No
Mercury	10.0	<0.05 – 0.13	0.07**	-	Yes (1)
Nickel	10.0	0.7 – 42.0	4***	-	Yes (5)
Zinc	10.0	9.7 – 35.0	10.9***	-	Yes (9)

Contaminant	No. of Samples	Concentration Range (ug/l)	Screening Criteria (µg/l)		Is Screening Criteria Exceeded? (No. of Exceedances)
			EQS	WQS^	
Total Cyanide	10.0	<10	1*	-	Yes#
pH (pH units)	10.0	6.1 – 7.3	-	6.5 – 9.5	Yes (3)

* EQS for freshwater – Annual Average
** EQS for freshwater – Maximum Allowable Concentration
*** EQS for freshwater – Annual Average, no account taken of bioavailability of contaminant
^ Scottish Water water quality standards https://www.scottishwater.co.uk/-/media/ScottishWater/Document-Hub/Factsheets-and-Leaflets/Factsheets/100620SWFactSheet22020V6web.pdf
# Limit of detection higher than screening criteria

The table above indicates that the EQS and / or WQS were exceeded for a number of contaminants, a summary of which is provided in Table 24 below.

Table 24. – Summary of Elevated Inorganic Groundwater Contaminants

Contaminant	Location of Exceedances	Screening Criteria (µg/l)	Elevated Concentrations Identified (µg/l)
Cadmium	BH10, BH36	0.09 (EQS)	0.21 – 0.22
Copper	BH08, BH10, BH18, BH24, BH28, BH31, BH36, BH42, BH46	1 (EQS)	1.7 – 10.0
Mercury	BH08	0.07 (EQS)	0.13
Nickel	BH08, BH10, BH31, BH36, BH42	4 (EQS)	4.7 – 42.0
Zinc	BH01, BH08, BH10, BH18, BH24, BH28, BH31, BH36, BH46	10.9 (EQS)	12.0 – 35.0
pH (pH units)	BH01, BH10, BH42	6.5 – 9.5 (WQS)	6.1 – 6.3

Table 22 above indicates that there are exceedances of the EQS for Cadmium, Copper, Mercury, Nickel and Zinc, and also exceedances of the WQS for pH. A review of the exploratory hole logs does not show a reason for these exceedances and a review of the Site information provides no previous Site use that could cause groundwater contamination. Furthermore, there are no pockets of Made Ground present throughout the Site that could explain the exceedances. Subsequently, it is considered possible that the contaminant concentrations identified are reflective of natural concentrations within groundwater rather than being suggestive that activities at or adjacent to the site have led to contamination of the groundwater.

The elevated concentrations of some contaminants with respect to the water quality standards suggests that groundwater quality may not be suitable for potable supply, however it is recommended that further specialist advice is obtained in this regard.

Organic Contaminants

The results of the organic groundwater laboratory analysis showed that all concentrations of organic contaminants (Total Phenols, PAHs and Petroleum Hydrocarbons) were below the laboratory detection limits so no organic contamination issues are expected to be present on Site.

6.4. Ground Gas

There are no known anthropogenic sources of ground gas at the Site e.g. landfills, disused mines and thick layers of Made Ground. Peat is only present within one exploratory hole (BH18).

Following the ground investigation a total of four rounds of ground gas monitoring have been undertaken at the time of reporting. A further two rounds of monitoring is yet to commence. The results in this report are based on the "New Deer 2 Ground Investigation Report" by BAM Ritchies dated 15 February 2024, Final version 00. A summary of the current monitoring data available at the time of reporting is provided in Table 23 below

Table 25. – Summary of Ground Gas Monitoring Data

Monitoring Well	Depth of Response Zone (mbgl)	Geological Unit	Groundwater Levels (mbgl)	Peak CH_4 (%)		Peak CO_2 (%)		Minimum O_2 (%)		Peak H_2S (ppm)		Peak CO (ppm)		Peak Flow (l/hr)	
				Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.	Min.	Max.
BH01	1.0 – 5.0	Cohesive Glacial Deposits and MacDuff Formation	3.80 – 3.98	0.0	0.0	3.9	77.0	9.3	20.9	0.0	1.0	0.0	0.0	0.0	0.4
BH02	0.5 – 3.5	Granular Glacial Deposits and MacDuff Formation	Dry	0.0	0.0	0.4	4.9	10.5	21.7	0.0	1.0	0.0	0.0	0.0	0.4
BH05	1.0 – 8.0	Granular Glacial Deposits and MacDuff Formation	8.20 – 8.26	0.0	0.0	6.3	7.1	12.6	15.1	0.0	1.0	0.0	0.0	0.0	0.4
BH07	0.5 – 2.5	Cohesive Glacial Deposits and MacDuff Formation	Dry	0.0	0.0	4.8	6.1	12.9	18.3	0.0	0.0	0.0	0.0	0.0	1.6
BH08	5.0 – 7.5	MacDuff Formation	Dry	0.0	0.0	4.8	6.0	16.0	17.1	0.0	1.0	0.0	0.0	0.0	0.4
BH10	4.3 – 7.3	MacDuff Formation	2.21 – 2.44	0.0	0.0	2.3	4.4	0.0	21.3	0.0	0.0	0.0	0.0	0.0	1.7
BH16	1.2 – 2.7	Cohesive Glacial Deposits and MacDuff Formation	Dry	0.0	0.0	2.8	3.8	16.0	17.5	0.0	1.0	0.0	0.0	0.0	0.4
BH18	1.0 – 2.5	Cohesive Glacial Deposits	1.84	0.0	0.0	2.9	6.0	15.7	19.1	0.0	1.0	0.0	0.0	0.0	0.4
BH20	0.5 – 2.3	Granular Glacial Deposits and MacDuff Formation	2.17 – 2.27	0.0	0.0	1.4	1.7	0.0	20.6	0.0	0.0	0.0	0.0	0.4	0.5
BH24	4.0 – 8.0	MacDuff Formation	6.35 – 6.81	0.0	0.0	1.6	2.5	19.6	20.7	0.0	0.0	0.0	0.0	0.0	0.5
BH28	1.0 – 5.0	Granular Glacial Deposits and MacDuff Formation	Dry	0.0	0.0	0.2	2.8	8.6	22.0	0.0	0.0	0.0	0.0	0.0	0.4
BH31	1.0 – 4.0	Granular Glacial Deposits	Dry	0.0	0.0	0.2	3.4	17.3	21.8	0.0	0.0	0.0	0.0	0.0	0.6
BH36	0.5 – 2.3	Granular Glacial Deposits and MacDuff Formation	0.45 – 0.49	0.0	0.0	3.2	5.6	11.0	21.0	0.0	1.0	0.0	0.0	0.0	5.2
BH39	0.5 – 3.5	MacDuff Formation	Dry	0.0	0.0	3.3	3.3	0.0	21.7	0.0	0.0	0.0	0.0	0.0	0.0
BH42	1.2 – 5.0	MacDuff Formation	N/A	0.0	0.0	0.2	3.8	18.5	21.9	0.0	0.0	0.0	0.0	0.3	0.5
BH46	0.5 – 2.0	Cohesive and Granular Glacial Deposits.	N/A	0.0	0.0	0.5	2.3	18.0	21.4	0.0	1.0	0.0	0.0	0.0	0.4

The table above indicates that predominately the measured groundwater level within each monitoring well was below the top of the corresponding response zone i.e. it was not flooded, therefore the gas monitoring results are likely to be indicative of the soil pore gas concentrations. BH36 is the exception to this, where the groundwater level is above the top of the response zone which means the well was flooded. In line with UK guidance (principally CIRIA C665), it is therefore not considered appropriate to use the available ground gas monitoring data for BH36 to undertake a ground gas assessment as the data does not provide a true indication of soil gas concentration and therefore could be misleading with respect to appraising ground gas risks. However, monitoring points show CO_2 present within some monitoring points which will require a ground gas risk assessment to be undertaken.

6.5. Recommendations and Conclusions

Based upon the findings of the risk assessments outlined within this report, it is recommended that:

- Whilst there is no record of contamination present throughout the Site, geo-environmental testing should be conducted on soil samples from the Site to confirm the levels of contamination during construction are within acceptable levels.
- The elevated concentrations of some contaminants with respect to the water quality standards suggests that groundwater quality may not be suitable for potable supply, however it is recommended that further specialist advice is obtained in this regard.

There is also potential for previously unidentified contamination to be present at the Site and it is further recommended that the contractor has a procedure in place to manage any such occurrences of previously unidentified contamination e.g., a Remediation Strategy or Construction Environmental Management Plan, that has been approved by the Local Authority in association with any planning conditions that may be specified for the proposed development.

7. Engineering Discussion and Recommendations

The substation platform is proposed to be situated on an area of sloping ground, with levels falling from west to east from approximately +155mOD to +115m OD across the proposed platform development area (i.e. an elevation difference of up to approximately 40 m). It is therefore anticipated a cut and fill earthworks exercise will be required to provide a level platform for the substation.

Earthworks design, detailed foundation design, and slope stability assessment is out with the scope of this report; however, a preliminary engineering discussion is provided below.

7.1. Foundations

Based on the ground conditions encountered at the Site, then it is considered likely that shallow (gravity bearing) foundations will be acceptable in supporting the proposed structures at the Site, to meet both bearing capacity and settlement criteria due to the shallow rockhead encountered during the Ground Investigation.

Within the western edge of the platform, where it is anticipated that the existing ground level will be cut down to bedrock level, then the bedrock at the Site should be considered to act as a competent bearing stratum. Within the eastern edge of the proposed development, where it is anticipated that upfilling of existing ground levels will be required to achieve the finished platform level, then the Glacial Deposits are also considered likely to act as a competent sub-grade for the substation platform to be constructed on.

It is recommended that Topsoil is not considered as an acceptable sub-formation material for both foundations and for the proposed substation platform and should be removed from the platform area.

Chemical aggressive testing conducted on soil samples throughout the Site are used in accordance with BRE SD1 to provide the suitable concrete classification to be used for the foundations. The test results provide a recommended Design Sulphate Class for the Site of DS-1 and the corresponding ACEC classification of AC-1.

7.2. Access Tracks

Dynamic Cone Penetration (DCP) tests were conducted along the proposed access track and the results are shown in Section 5.2.3. The testing is predominately conducted on the Topsoil present at the existing ground level and weathered rock from the MacDuff Formation. The DCP test results provide a range of CBR values of 2.9 – 11.0 %, which correlates to a range of design subgrade surface modulus of 35 - 82 MPa, as outlined in CD 225 - Design for new Pavement Foundations[11].

This range of values would suggest a founded access track would be suitable and there are no sections of the track that will require to be of 'floated' construction. The subgrade surface modulus can then be reviewed against design charts presented within CD 225 to give an indicative track thickness for the founded pavement. Based on achieving a Class 2 Foundation constructed using

[11] CD 225 – Design for new pavement foundations, Highways England (April 2020)

Class 6 capping material, the track thicknesses may be in range of 250mm to 550mm, however this should be confirmed during detailed design and based on a review of anticipated road loading.

This thickness could be reduced through the use of Type 1 sub-base material; however, as this material may not be readily available at the site this may not be an economic approach. Similarly, consideration of the use of geogrid may allow for a reduction in the volume of Class 6 material used; however, specialist suppliers should be consulted during the design of these elements.

7.3. Slope Stability and Retaining Structures

The current development drawings indicate the finished Site level to sit at approximately +129.0mOD. These proposals result in the requirement for an extensive cut to be undertaken along the western boundary of the Site, offset with significant volumes of fill within the eastern area of the Site. As a result, it is considered likely that there will be a requirement for the design and construction of engineered slopes or the installation of retaining structures, particularly in any areas of spatial constraints where the formation of traditional slopes may not be permitted.

Where a retaining structure is required, then it is anticipated that a gravity based retaining structure (e.g. a reinforced concrete retaining wall, pre-cast concrete modular retaining wall, or gabion basket retaining wall) will be the most practical form of retention due to the presence of shallow bedrock. It is anticipated that the bedrock within this area will act as a competent bearing stratum for a gravity based retaining structure within the west of the Site. The construction or installation of 'top down' cantilevered retaining wall systems (e.g. sheet pile retaining structure or secant / contiguous piled wall retaining structure) are considered to be an unfeasible or uneconomical solution when considering the ground conditions at the Site (i.e. shallow bedrock that may hinder conventional driving techniques).

Within the east of the Site, due to the proposed platform being at a greater level than current ground level, there will be a resulting requirement for the design and construction of an engineered slope. Based on the use of a well-compacted granular engineering fill, these embankment slopes could be constructed to gradients of 1V:2.5H, or potentially steeper if a reinforced earth solution is adopted. It is recommended that all Topsoil, and soft / loose deposits are removed from beneath the platform area prior to construction and benches formed prior to placement of engineered fill, to support the stability of the embankment slopes at this location.

Where cut-slopes are proposed within natural materials, then it is anticipated that these could be cut at gradients of up to 1V:2.5H within the Cohesive Glacial Deposits, 1V:2H within the Granular Glacial Deposits, up to 1V:1.5H within weathered or non-intact rock, and up to 1V:1H within intact rock. The stability of all cut slopes should be assessed at detailed design stage taking cognisance of any adjacent loading or vehicle trafficking. Due to the poor natural drainage characteristics of the Site (discussed in Section 7.4) it is recommended that any engineered slopes are supported with adequate drainage.

7.4. Earthworks and Material Re-use

An assessment of the material's re-use characteristics has been undertaken for the Site, which has indicated that following re-processing of the superficial, through the introduction of larger particles, then this material may be considered for re-use during construction; however, in its current state the material predominately does not comply with recognised construction material outlined within the MCHW SHW. With most of the material throughout the Site being granular in nature, then reprocessing of this material would lend itself more readily to being developed into a Class 1 or Class 6 material.

An assessment of the characteristics of Site bedrock would indicate that the durability characteristics of this material, if crushed to the appropriate grading, may be re-used during the construction works as a Class 1 or Class 6 material.

Existing proposals for the substation platform indicate a finished level of +129mOD. This finished level in relation to existing Site levels would suggest that a significant volume of fill is required relative to the volume of material that will be cut, or 'won', from the Site. Furthermore, as the most upper layers of the ground were recorded to predominantly contain Topsoil, the reusability of this material is considered limited. Should a more balanced cut and fill volume be desirable, then it is recommended that lowering of the platform level (or over-excavation of the substation platform) is considered in order to recover a higher volume of higher quality material, won at depth within the bedrock. This approach should be considered against the challenges foreseen with rock excavation, and it is recommended that a detailed earthworks assessment is undertaken in order to determine the most practical and economical course of action.

Whilst no contaminated land is expected to be present within the Site, no soil samples were tested for contaminants. It is recommended that further geo-environmental testing is conducted during the construction works phase to confirm the contamination levels throughout the Site.

7.5. Site Drainage

Drainage of the Site will be required to be considered in both the permanent works and the temporary works of the proposed development.

With regard to temporary works, the GI has revealed that groundwater exists at the Site at shallow depths, which was observed through groundwater strikes recorded within the exploratory holes and the long-term monitoring undertaken using standpipe installations within boreholes. As a result, the control and management of groundwater should be a key consideration in the temporary works phase of the project. Shallow groundwater may result in the requirement for pumping or removal of groundwater from excavations, and also may result in excavation instability during construction.

With regard to permanent drainage at the Site, based on the results of the in-situ soakaway testing undertaken during the GI works, it is envisaged that the adoption of a natural drainage system, or the use Sustainable Urban Drainage Systems (SUDS) may be challenging, and would require careful consideration.

In addition to the findings of the in-situ testing, assessment of the permeability of the Glacial Deposits at the Site through correlation with the results of PSD testing indicated a permeability range of $1x10^{-6}$ to $1x10^{-8}$ ms^{-1} for the granular material and of $1x10^{-7}$ to $1x10^{-9}$ ms^{-1} for the cohesive material. Permeability values of these magnitude may be considered 'poor', hence, it is considered probable that the adoption of a 'hard' drainage system (i.e. installation of a series of pipes, culverts and manholes) may be required to facilitate the Site drainage requirements; however, this should be confirmed through detailed design.

Appendix A – GI Factual Report

Appendix B – Ground Models