GAVIN & DOHERTY GEOSOLUTIONS

Illaunbaun Wind Farm - Environmental Impact Assessment Report

Appendix A09-03: Ground Investigation Report

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EXECUTIVE SUMMARY

Gavin and Doherty Geosolutions Ltd. (GDG) were requested by JC Mont-Fort Holding SA to complete aplanning stage Ground Investigation Report (GIR) for the geotechnical design of Illaunbaun Wind Farm in Co. Clare, Ireland. The GIR includes the development of an engineering geological model of the study area and recommends characteristic geotechnical parameters for the preliminary geotechnical design of the civil infrastructure associated with the development. This assessment is based on the following:

1) A desk study of high-level open-source data from various online mapping databases,
2) Scheme-specific ground investigations consisting of
   a) Rotary Core Boreholes,
   b) Trial Pits, Hand Shear Vane Tests,
   c) Peat Probes,
   d) Russian Core Augers,
   e) Chemical and Environmental Laboratory Tests, and
   f) Geotechnical Soil and Rock Laboratory Tests.
3) Published and unpublished case histories.

It is highlighted that the geotechnical information detailed within this document is limited to the soil information made available at the time of writing. The latest information used in this report was taken from Clare N4 (Illaunbaun) Factual report, January 2025, prepared by Irish Drilling Ltd. Any additional information which may become available following the issue of this GIR should be reviewed by the relevant designer and incorporated into their design ground models which may result in variations from the geotechnical parameters recommended herein.

In general, the subsurface geology includes Peat, Shallow Cohesive Deposits, Glacial Till, Weathered Rock, and Sandstone bedrock. The anticipated depths and thickness of the underlying soil and rock stratigraphy have been summarised for the proposed development area. The groundwater levels recorded during the GI were also studied to determine the most probable groundwater level.

The results of in-situ tests (including Standard Penetration Tests, hand shear vanes, and Dynamic Probes), and geotechnical and geoenvironmental laboratory tests have been reviewed in this GIR. Recommended characteristic geotechnical parameters associated with each stratum have been presented based on the factual GI information received to date.

1 INTRODUCTION

1.1 DESCRIPTION OF THE PROJECT

GDG was requested by JC Mont-Fort Holding SA (JC Mont-Fort) to complete a Ground Investigation Report (GIR) for the planning stage design of the Illaunbaun Wind Farm. The Site is located in western County Clare, approximately 4.2 km northeast of Milltown Malbay and 5.2 km south of Lahinch. The site infrastructure outline layout is presented in Figure 1-1. The proposed development will include the following infrastructure:

- Construction of 6 wind turbines with a maximum overall blade tip height of 150 m.
- Construction of associated turbine foundations, crane pad hardstand and assembly areas.
- Construction of one permanent 38 kV electrical on-site substation with one control building with welfare facilities, all associated electrical switchgear, security fencing, underground cabling, drainage infrastructure, and all ancillary works.
- All associated internal underground electrical and communications cabling connecting the wind turbines to the on-site Substation.
- Upgrade of existing tracks, roads and provision of new site access roads to facilitate construction & operation of the wind farm.
- Two borrow pits.
- Three peat repository areas for peat & spoil management.
- Construction of one temporary construction compound.
- Development of internal site drainage.
- Permanent & Temporary tree felling to accommodate the construction & operation.
- Signages and
- All associated site development works.

1.2 GEOTECHNICAL CATEGORY

The scheme has been identified as Geotechnical Category 2 according to I.S. EN 1997-1:2005+A1:2013 in that it includes only conventional types of structure with no exceptional risk or difficult ground or loading conditions.

1.3 SCOPE OF THE REPORT

This GIR is prepared in accordance with I.S. EN 1997-1:2005+A1:2013 and the 2015 AGS Guide to Good Practice in Writing Ground Reports. This GIR provides a summary of a desk study, the factual ground investigation (GI) information, in-situ testing, geotechnical laboratory soil and rock testing, and a preliminary ground model for the Site. This report recommends characteristic geotechnical parameters for use in the planning stage design of the various infrastructure elements.

It is recommended that additional GI campaigns are completed in advance of the detailed design. This additional GI is to provide greater confidence in the characteristic geotechnical parameters and to provide location-specific ground models for the critical infrastructure (WTG locations, the substation, etc.).

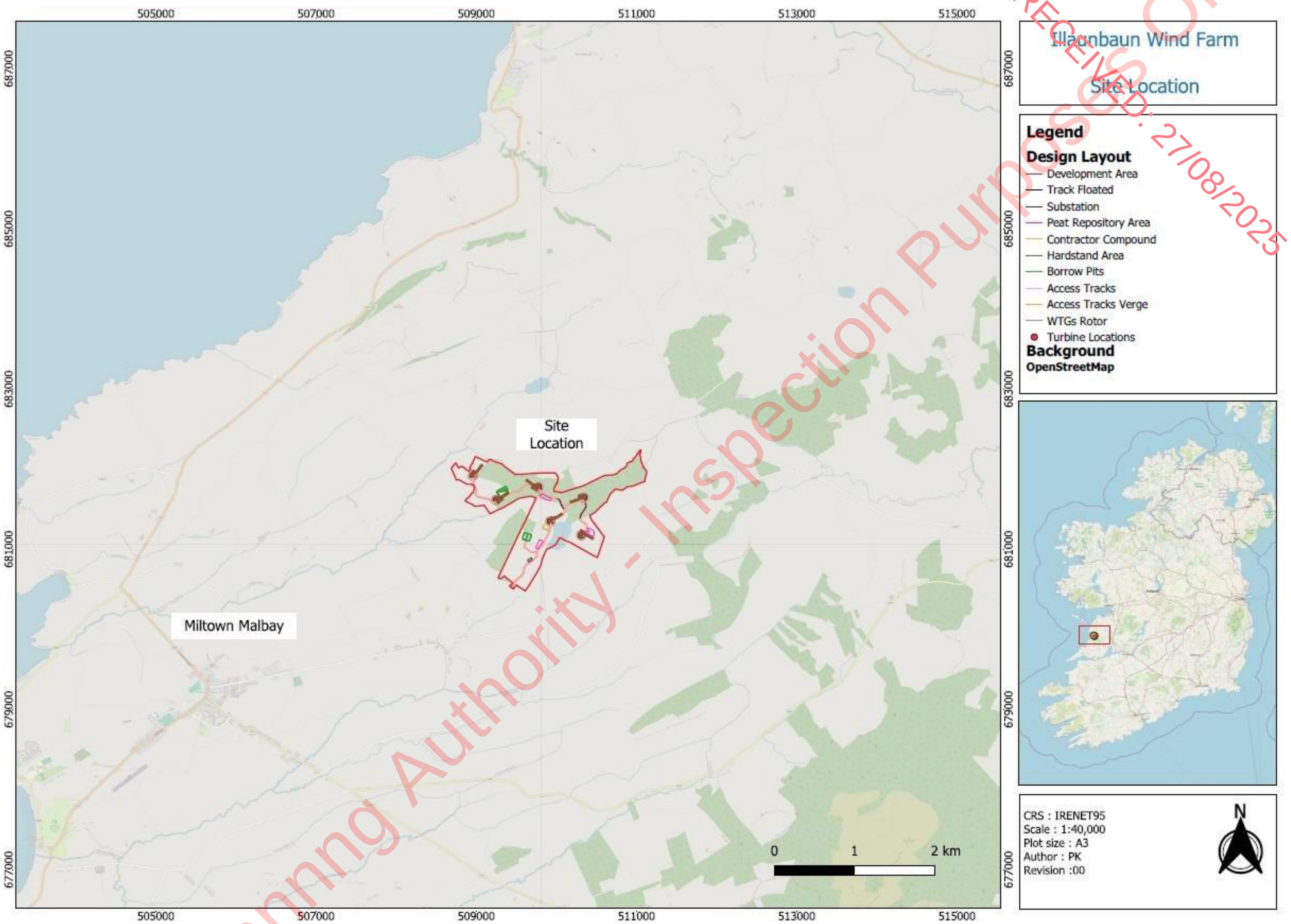

Figure 1-1: Illaunbaun Wind Farm Site location

2 DESK STUDY

2.1 SITE LOCATION AND DESCRIPTION

The proposed Illaunbaun Wind Farm is located approximately 2.9km west coast of the Co. Clare coastline. The town of Milltown Malbay is located 4.2km to the southwest and the town of Lahinch is located 5.2km to the north of the Site shown in Figure 2-1. The proposed planning boundary encompasses the townlands of Toreen, Slievenalicka, Illaunbaun, Lackamore and Drumbaun, and is approximately 150.48 hectares in size. The main site access is from the Slievenalicka regional road located to the southwest of the Site. Proposed access routes partially comprise existing forestry tracks. An initial site walkover conducted by GDG staff in April 2022 confirmed the presence of coniferous forestry and the associated tracks.

The Site is surrounded by private agricultural properties and Coillte-owned dense coniferous forests are located along the north and centre of the Site. Lough Keagh is the main surface water body and is centrally located within the proposed site. The surrounding agricultural land is comprised of peat and grass farmland with ditches and low walls between fields. The *Liscannor Flagstone Quarry* is located approximately 180m west of the proposed site.

2.2 TOPOGRAPHY

Site topography consists of low rolling hills with an average ground elevation of 179m above Ordnance Datum (m OD) with the lowest point at 129.4m OD in the western corner and the highest point at 195m OD on the hill of Knockabullaunduff. The access route from the northeast, through the Illaunbaun townland, is steep and rises from 95m OD to 185m OD. Access routes from the southwest join the Site at higher elevations, approximately 190m OD. A map showing contours, watercourses and other key features is shown in Figure 2-2.

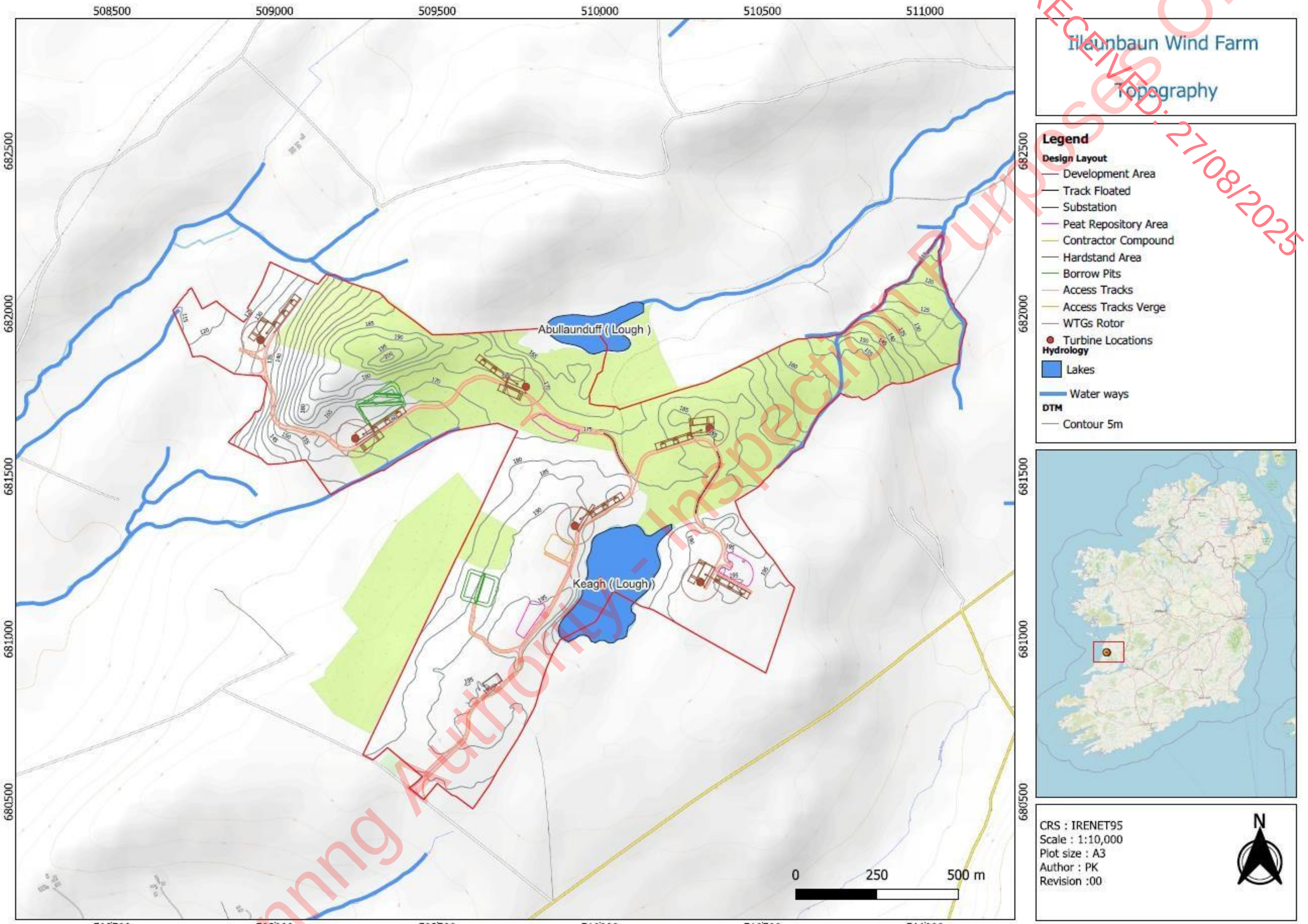

Figure 2-1: Site Topography

2.3 QUATERNARY GEOLOGY

According to the Geological Survey Ireland's (GSI, 2024) quaternary sediments mapping (1:50k) with an extract shown in Figure 2-3, the Proposed Development is predominantly underlain by a mosaic of blanket peat and bedrock outcrop or subcrop. The map indicates a combination of peat deposits interspersed between thin unsubstantial soils. Bedrock outcrop/subcrop is generally located in the upland areas and topographic highpoints within the north and west of the Site but is spatially extensive throughout. Tills derived from Namurian sandstones and shales are present at the boundaries of the Proposed Development area, especially in Drumbaun and Illaunnbaun townlands. Glacial till typically comprises a heterogeneous mix of sand, gravel, cobbles, and boulders, usually held in an over-consolidated clay matrix. This till classification indicates that the glacial tills are likely locally derived from the underlying Namurian age bedrock.

2.4 GEOLOGY

Geological Survey Ireland (GSI, 2024) 1:100,000 scale bedrock mapping shows the Proposed Project and surrounding area to be underlain entirely by a single bedrock formation, the Central Clare Group (CCG). This formation, dating to the Carboniferous period, Namurian (331-319 Ma), is composed predominantly of grey to dark grey sequences of mudstone, siltstone, and sandstone. These sediments reflect a depositional environment influenced by fluvio-deltaic and basinal marine (turbiditic) processes, indicative of dynamic geological conditions during their formation.

This lithology is characterised by grey/dark grey cyclothemic sequences of mudstone, siltstone, and sandstone of fluvio-deltaic & basinal marine (turbiditic) origin. The basal mudstone is usually 7-18m thick and laminated. In general, the mudstones are overlain by laminated to massive grey siltstones followed by thick, laminated and cross-bedded sandstones. The *Liscannor Flagstone Quarry* to the west of the Site extracts dark grey sandstone flat slabs known as 'Flags' and is used for flooring and building stone.

As limestone bedrock does not occur within the site boundary, karst features are not considered to be a risk. The main bedrock unit and associated structural features within the Proposed Development boundary and surrounding area is shown in Figure 2-4.

The structural geology in the wider area is principally controlled by the dominant regional structural lineation in western Co. Clare, which exhibits signatures of historical compression in a northeast-southwest trending fold axis. Rocks are folded into relatively small folds with wavelengths of about 3 km. The intensity of folding decreases northwards (GSI, 2024). Some key structural features surrounding the Site include:

- A pair of WSW - ENE trending faults are mapped at Slievecallan townland, approximately 450m southeast of the Proposed Development.
- A further single fault, trending southeast-northwest, is located approximately 300m southwest of the Proposed Development, at Drumbaun townland.
- Bedrock strata in the area of the Proposed Development dips at right angles to the fold axes at angles of 10 °, with wider CCG bedrock exhibiting a range from 10 °-50° (GSI, 2024).

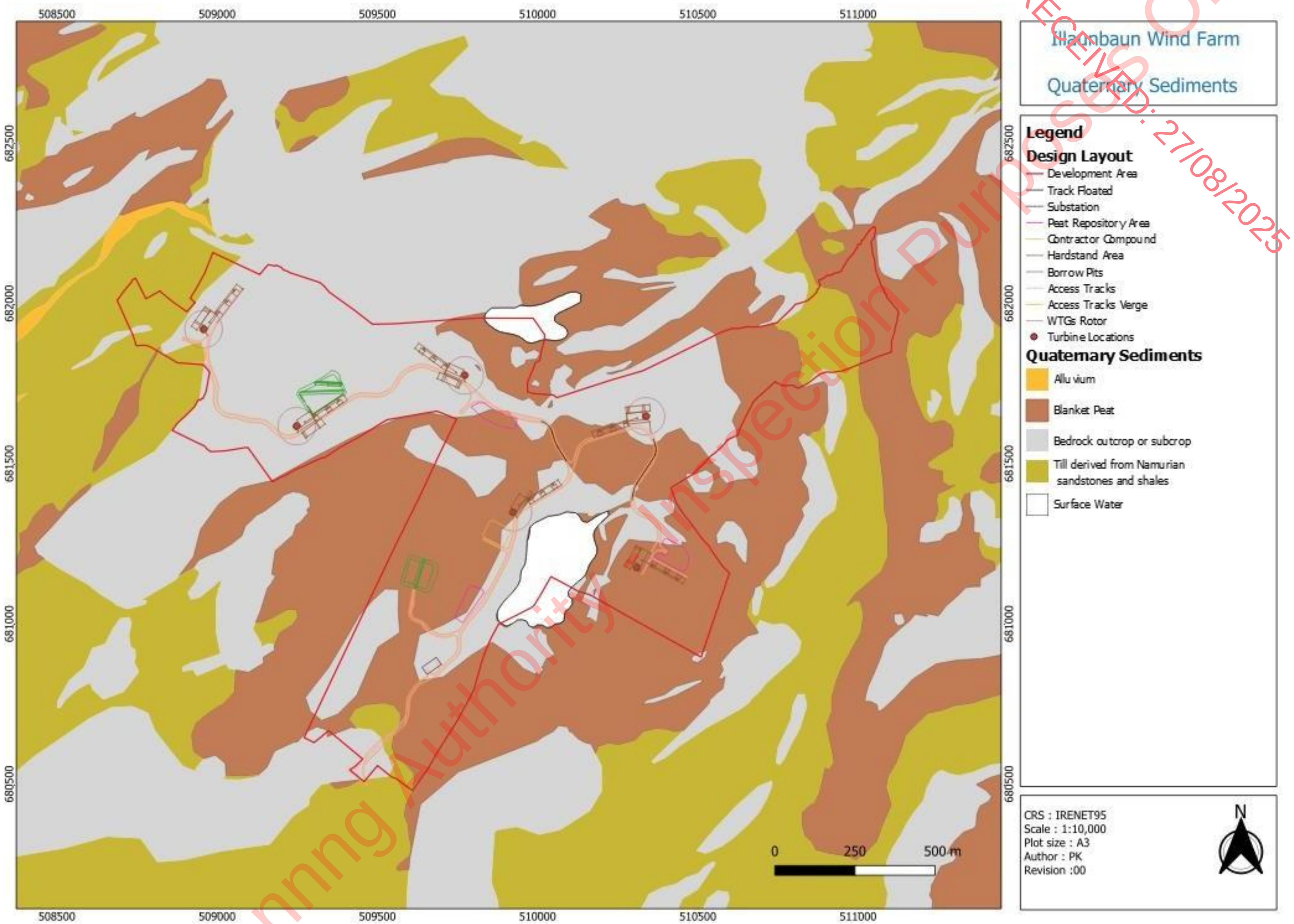

Figure 2-2: Quaternary Sediments

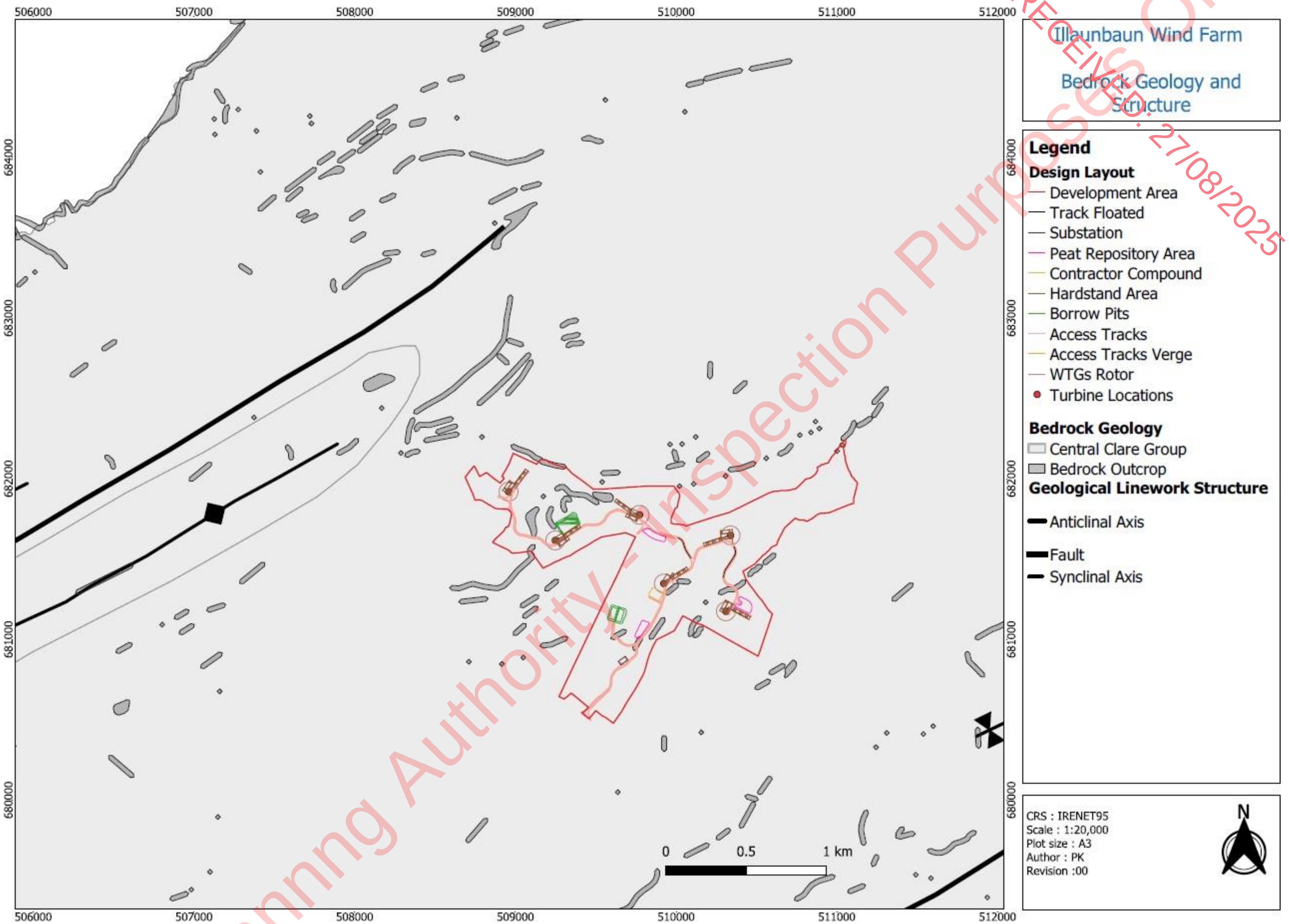

Figure 2-3: Bedrock and Structural Geology

2.5 HYDROGEOLOGY

2.5.1 GROUND WATER BODY

According to GSI's groundwater map viewer, the Proposed Development and associated access track routes are entirely underlain by the Milltown Malbay groundwater body (GWB), (ID: IE_SH_G_167). This GWB in relation to the Proposed Development boundary and surrounding area is shown in Figure 2-5.

The Milltown Malbay GWB covers much of western Co. Clare and comprises a total area of 766km^2. It is elongated north-south and elevations are predominantly below 100m OD. The GWB is bound to the west by the coastline, the north by karstified Dinantian Pure Bedded Limestones of the Slieve Elva GWB and to the east/south by surface water catchment divides associated with the Inagh River (GSI, 2024).

The GWB is dominated by rock units from the Namurian Undifferentiated group and large areas of Namurian sandstones are found in the northwest and southwest of the GWB. The GWB is comprised of low-permeability siliceous rocks with localised zones of enhanced permeability (GSI, 2003).

Transmissivities are generally in the range of 2-20m^3/d and aquifer storativity for all bedrock formations are likely to be low.

Aquifer units may be both confined and unconfined depending on local subsoil conditions. In general, groundwater flow will be concentrated in the upper part of the aquifers, approximately 10-15m below ground level (bgl). Static groundwater levels are often 0-8m bgl. The main discharges are to small streams crossing the aquifers. Local unconfined flow directions are oblique to the surface water channels and overall flow is westwards.

The Milltown Malbay GWB is characterised as having a 'PP' – poorly productive flow regime (GSI, 2000a). It is not designated as a Groundwater-Dependent Terrestrial Ecosystems (GWDTE).

Groundwater body status for 2015 – 2018 period is designated as 'Good' overall, passing both quantitative and chemical status requirements under the Water Framework Directive 3rd cycle assessment. Groundwater body risk status for the same assessment period is currently designated as 'Review'.

2.5.2 BEDROCK AQUIFER

The bedrock aquifer type within the Proposed Development boundary and surrounding area is shown in Figure 2-6. According to GSI's groundwater map viewer, bedrock directly underlying the Site is categorised as a Locally Important (LI) Aquifer Bedrock. This is defined as "Bedrock which is Moderately Productive only in Local Zones". This means groundwater flow occurs predominantly through fractures, fissures and joints, giving a low fissure permeability which tends to decrease with depth. Flow paths are thought to be between 30 – 300m in length and locally important aquifers are generally capable of yielding enough water to supply single domestic wells only (10-20m^3/d), (GSI, 2017). The bedrock aquifer has been categorised as a member of the 'Namurian Undifferentiated (NU)' Rock Unit Group (RUG).

The regional groundwater flow direction in the aquifer will be westwards towards the Atlantic Ocean (2000a). Localised groundwater flow paths within the Proposed Development will follow the orientation of surface water sub-catchments from topographic highs to lower elevation discharge points. Shallow groundwater in the south of the Site will flow in the direction of Lough Keagh

2.6 GROUNDWATER VULNERABILITY

Groundwater vulnerability in Ireland, as defined in the Water Framework Directive – Recharge and Groundwater Vulnerability, is a function of the thickness and permeability of the subsoil that overlies bedrock. These factors strongly influence the attenuation processes and the time it takes for contamination to be released into the subsurface.

Groundwater vulnerability classifications within the Proposed Development boundary and surrounding area are presented in Figure 2-7. The majority of the Proposed Development exhibits a mixture of 'E – Extreme groundwater vulnerability' and 'X – Extreme groundwater vulnerability with bedrock', where bedrock is at or near surface. The easternmost area of the Site borders a zone of 'High' vulnerability in Illaunnbaun townland.

Due to the localised variability on-site, pre-development vulnerability observed at individual WTGs and other infrastructure such as borrow pits, peat storage areas, site compounds and access roads will vary depending on location. Based on the site walkover, ecological surveys and likely shallow groundwater regime, sensitive Groundwater-Dependent Terrestrial Ecosystems (GWDTEs) are considered unlikely across this site.

2.6.1 SUBSOIL PERMEABILITY

Subsoil permeability across the Proposed Development is categorised mostly as 'N/A' due to thin superficial deposits, where depth to bedrock is less than 3m, including all WTG locations. Areas of 'Low' permeability, where superficial deposits are slightly thicker, surround the Site to the East, West, and South. One of the access tracks, in Illaunbaun townland, is underlain by 'Low'

permeability, shown in Figure 2-8. There are no superficial aquifers located within or adjacent to the Proposed Development boundary, although it is possible that localised perched groundwater is present at the base of peat deposits and within granular layers/ lenses within the glacial till matrix.

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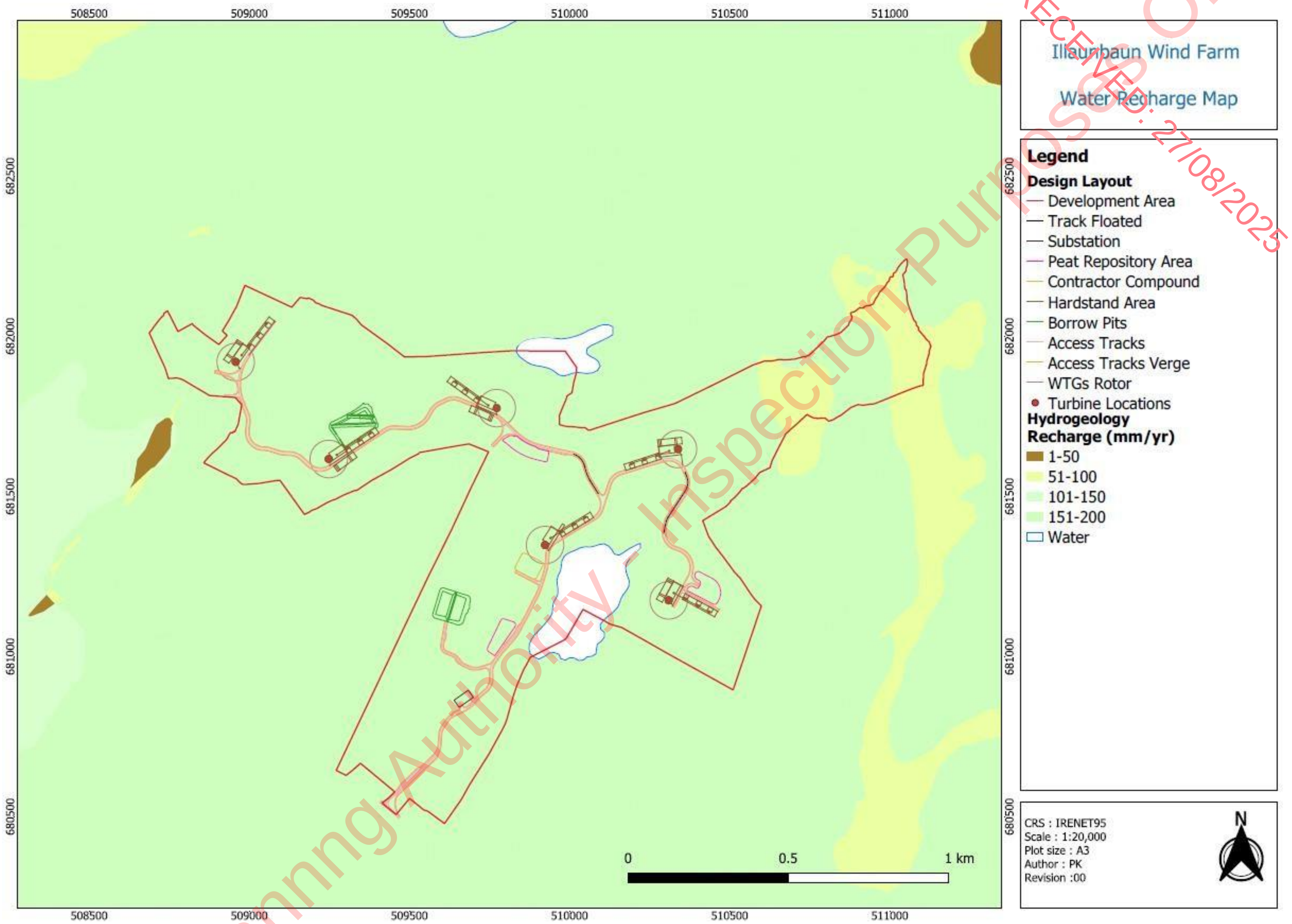

Figure 2-4: Groundwater Recharge

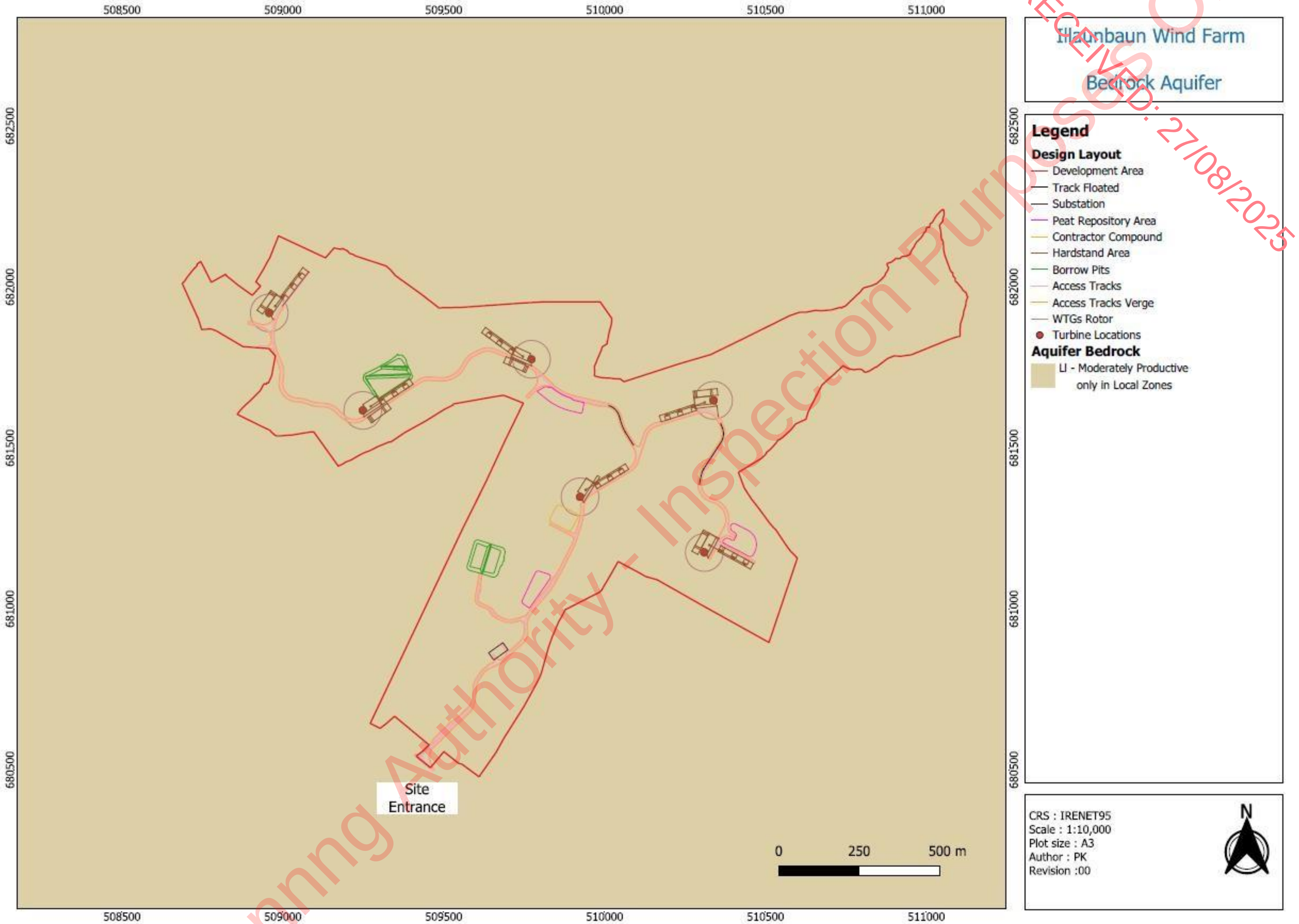

Figure 2-5: Bedrock Aquifer

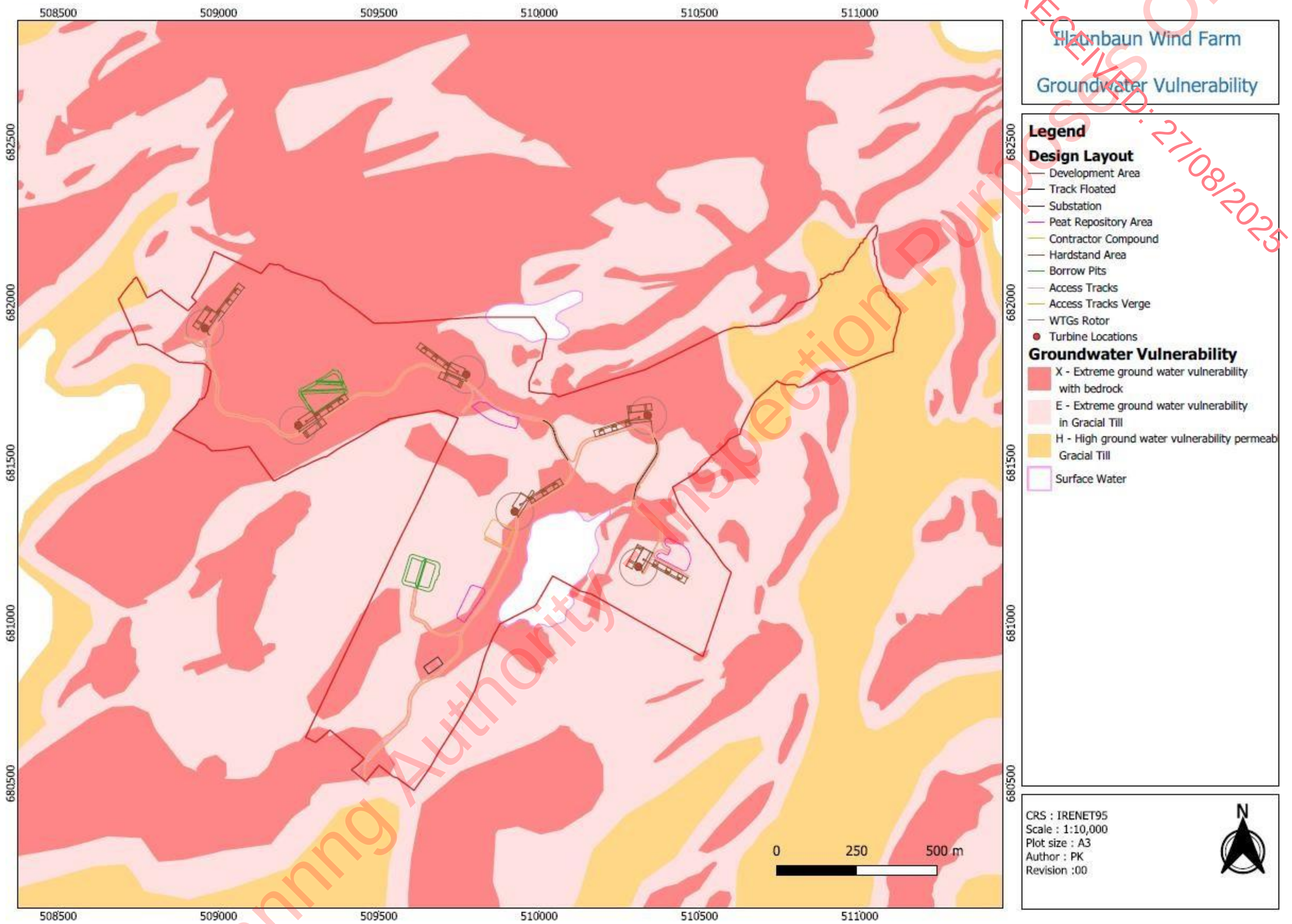

Figure 2-6: Ground Water Vulnerability

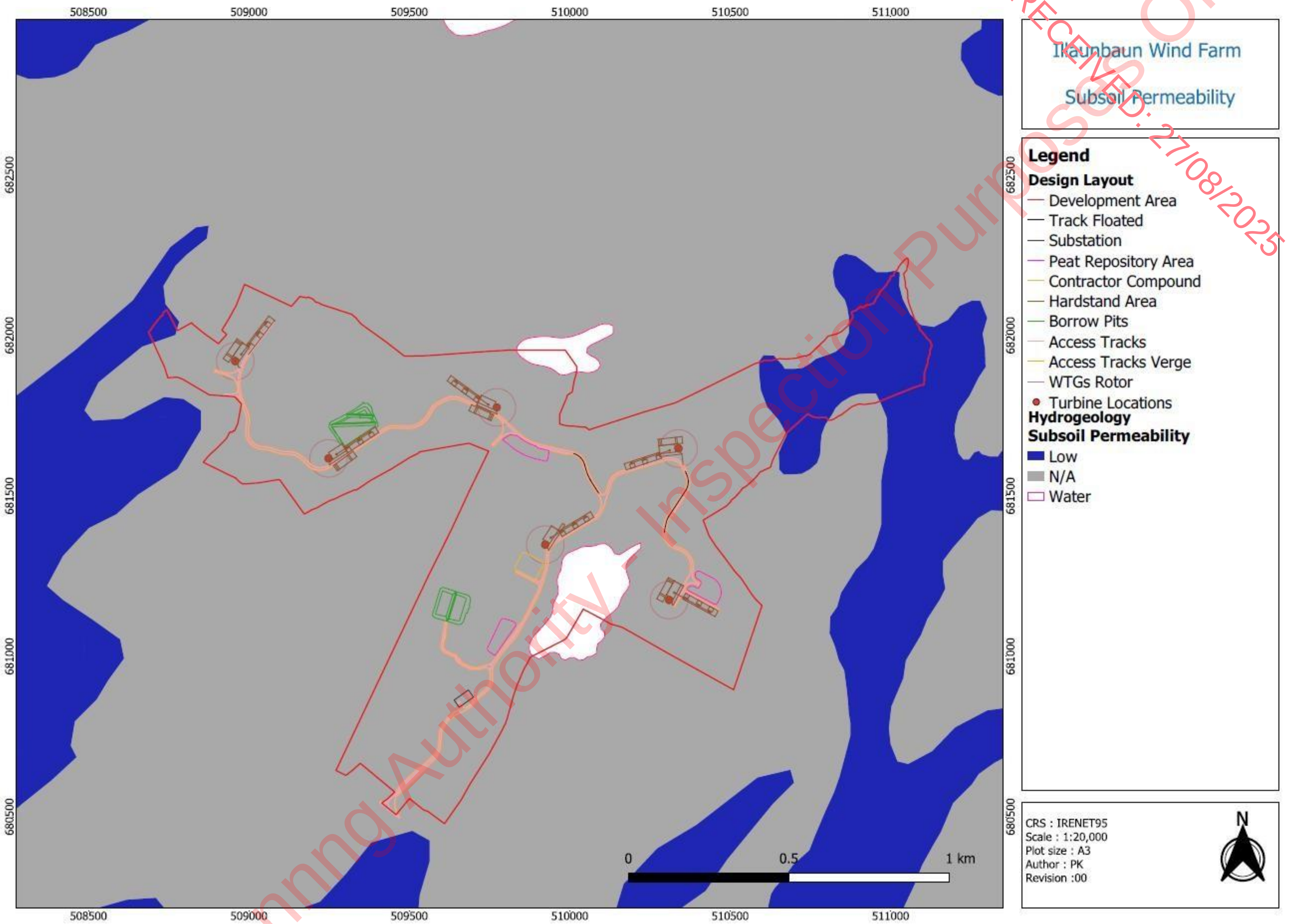

Figure 2-7: Subsoil Permeability

3 GROUND MODEL

3.1 GROUND INVESTIGATION

The geotechnical interpretation within this report is based on the factual ground investigation (GI) undertaken by Irish Drilling Ltd (IDL, 2025) in September 2024 and the corresponding Factual Report (revision 24_CE_108/02) issued in January 2025. The plan of the exploratory hole locations completed by the IDL is presented in Figure 3-1. The completed GI works comprised the following:

- Four Rotary Core Boreholes with Standard Penetration Tests attempted in the overburden materials;
- 17no. Machine-dug Trial Pits;
- 43no. Hand Shear Vane Tests;
- 88no. Peat Probes;
- 9no. Russian Core Augers;
- A suite of Geotechnical Soil and Rock Laboratory Tests; and
- A suite of Chemical and Environmental Laboratory Tests.

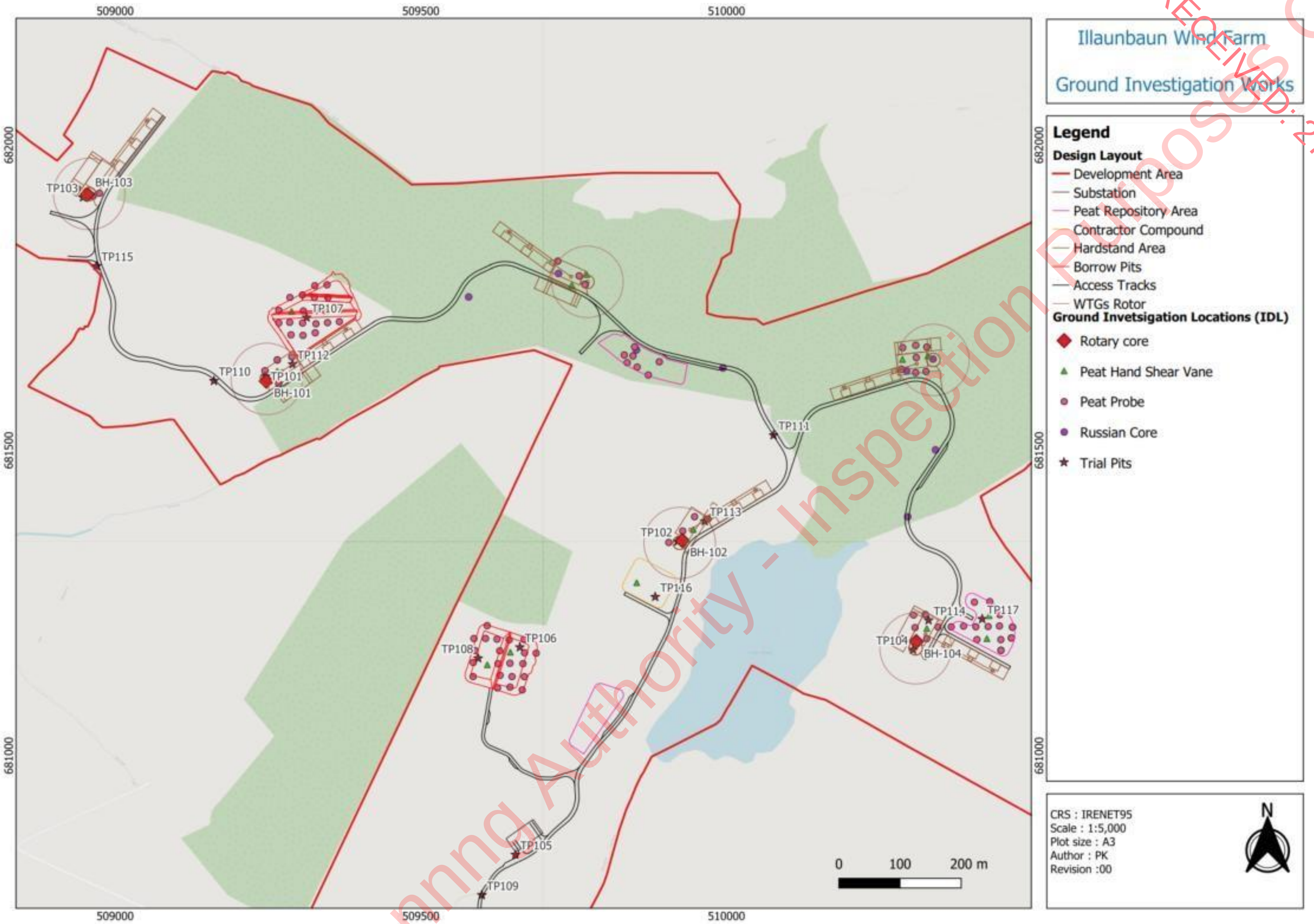

Figure 3-1: Ground investigation works completed by IDL (2024)

3.2 STRATIGRAPHIC MODEL

The stratigraphic model was developed from the geological descriptions presented in the rotary core borehole, trial pit and Russian Core Auger logs. In addition, the depth of Peat was further extrapolated at key areas around the site using peat probes. 88no. peat probes were completed using Hand-held Peat Probing Equipment. Access was not available for a further four peat probe locations (PP401, 403, 405, 406 inclusively). Peat probing involves the pushing of steel rods with a cone tip by hand into the subsoil with the aid of a 'T' bar at the top. As penetration is achieved additional rods are threaded onto the top and the process is repeated until target depth and/or refusal is encountered. The peat probes were carried out to depths ranging from 0.10m to 2.10m below ground level. Refer to the IDL Factual Report (IDL, 2024) for the full set of Peat Probe depths.

The strata recorded across the Site based on the 2024 GI factual report include:

- **Peat** – Spongy/firm black pseudo-fibrous/fibrous PEAT, often with grass, rushes, heather, briars and/or trees at surface level. The peat was designated as:
    - H2, H3 and H4 indicating the degree of humification is almost entirely undecomposed to slightly decomposed peat;
    - B1, B2 and B3 indicating a dry to moderate water content;
    - F2 and F3 indicating a moderate to high fine fibre content;
    - R1 and R2 indicating it is low to moderate coarse fibre content;
    - Typically, W0 indicating nil wood content, with the exception of TP116 which indicated a W3 high wood content;
    - TV1 indicating a low tensile strength in the vertical direction;
    - TH0 and TH1 indicating a zero to low tensile strength in the horizontal direction; and
    - A0 and A1 indicating the smell of the peat suggest no to slight fermentation under anaerobic conditions.
- **Shallow Cohesive Deposits** – Comprised of cohesive materials of various descriptions including:
    - Very soft greyish green SILT;
    - Soft blue slightly sandy slightly gravelly clayey SILT;
    - Soft to firm creamish light brown slightly sandy organic SILT;
    - Firm wet grey gravelly silty CLAY;
    - Firm blue CLAY with rootlets; and
    - Firm to stiff creamish brown organic CLAY with rootlets.
- **Glacial Till** – Comprised of interbedded cohesive, granular and mixed materials with no definitive trend with depth which is typical of Irish Glacial Till. As the depth to rock is relatively shallow across the Site (less than 8m below existing ground level), the Glacial Till is recommended to be

treated as the most onerous of a cohesive, granular or mixed material. The various descriptions of the interpreted Glacial Till included:

- Firm bluish grey CLAY with occasional cobbles and occasional boulders;
- Firm bluish grey slightly gravelly slightly sandy silty CLAY;
- Firm greyish brown gravelly silty CLAY with occasional cobbles and occasional boulders;
- Firm grey slightly sandy gravelly SILT with occasional cobbles and occasional boulders and large boulders;
- Stiff bluish grey slightly sandy slightly gravelly clayey SILT with occasional cobbles and occasional boulders;
- Stiff grey slightly sandy gravelly SILT;
- Stiff bluish grey SILT;
- Stiff pinkish brown/brownish grey slightly gravelly SILT;
- Stiff greenish brown slightly gravelly CLAY;
- Stiff blue and light bluish grey slightly sandy slightly gravelly silty CLAY;
- Very stiff dark grey slightly sandy gravelly SILT with rare cobbles;
- Very stiff blue gravelly CLAY. Gravel is angular to subangular fine to coarse of shale;
- Bluish grey sandy clayey angular to subangular and flat shale GRAVEL with rare cobbles;
- Bluish grey very sandy silty shale GRAVEL with medium cobble content; and
- Sand is fine to coarse. Gravel is subrounded to subangular fine to medium of assorted brown, dark grey and grey fine-grained sandstone and occasionally of grey siltstone. Cobbles are angular to subrounded and flat of grey sandstone and mudstone. Boulders are angular to subrounded and flat of grey sandstone and mudstone.

- **Weathered Rock** – Very strong locally strong thinly laminated greenish grey silty fine-grained sandstone and occasionally weak shale recovered as angular fine to coarse gravel and cobble-sized clasts with some grey gravelly silt. Gravel is fine to medium of greenish-grey and grey sandstone.
- **Intact Sandstone bedrock** – Very strong locally medium-strong to extremely strong thinly interlaminated greenish grey, grey and dark grey silty fine-grained SANDSTONE.

The stratigraphic trends encountered by the IDL GI works across the Site are summarised in Table 3-1. The geotechnical cross-sections of the exploratory holes at the substation, the two proposed borrow pit locations and each WTG location are shown in Figure 3-2 and Figure 3-7. These cross-sections demonstrate the GDG interpretation of the Site stratigraphy based on the factual GI information. It is noted that the GI completed at the proposed locations of WTG2 and WTG5 were Russian Core Augers and Hand Shear Vane tests only and as such geotechnical cross-sections have not been produced for these areas.

Table 3-1: Summary of the stratigraphic trends encountered across the Site

Geology	Depth to top (mbgl)		Level at top* (mOD)		Thickness (m)	
	Min	Max	Min	Max	min	max
Peat	0.00	0.00	128.78	195.07	0.20**	1.50
Shallow Cohesive Deposits	0.20	1.50	156.78	193.07	0.05**	2.70
Glacial Till	0.37	4.20	127.84	156.28	0.90**	6.20
Weathered Rock	0.80	2.20	156.28	189.94	0.20**	2.10
Sandstone	0.70	7.70	121.64	192.04	Unproven	

*Note: Levels vary significantly due to the existing topography of the Site.

**Note: Not all strata were encountered in every exploratory hole. However, the minimum thickness where each stratum was encountered has been included for context.

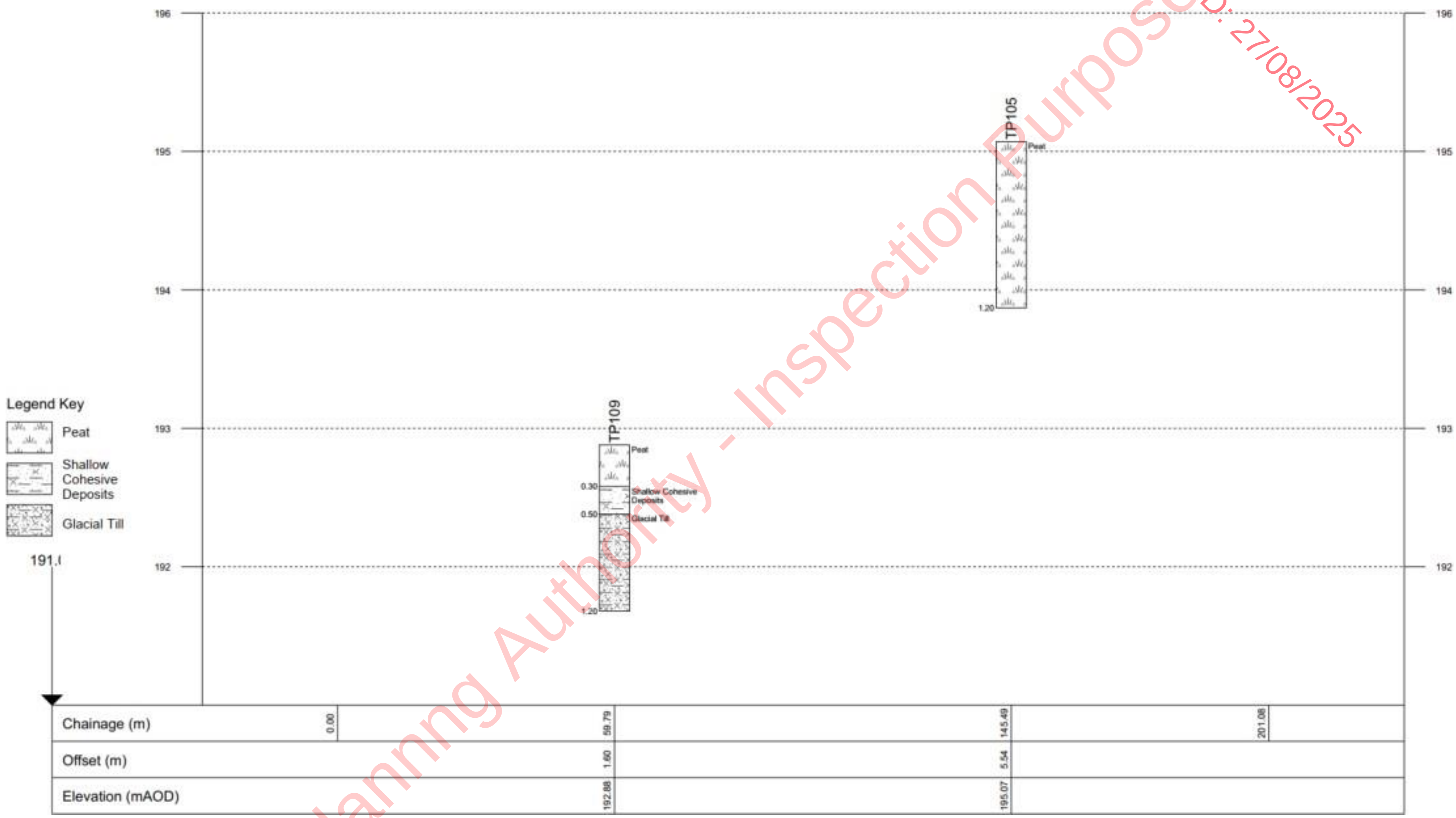

Figure 3-2: Substation geotechnical cross-section

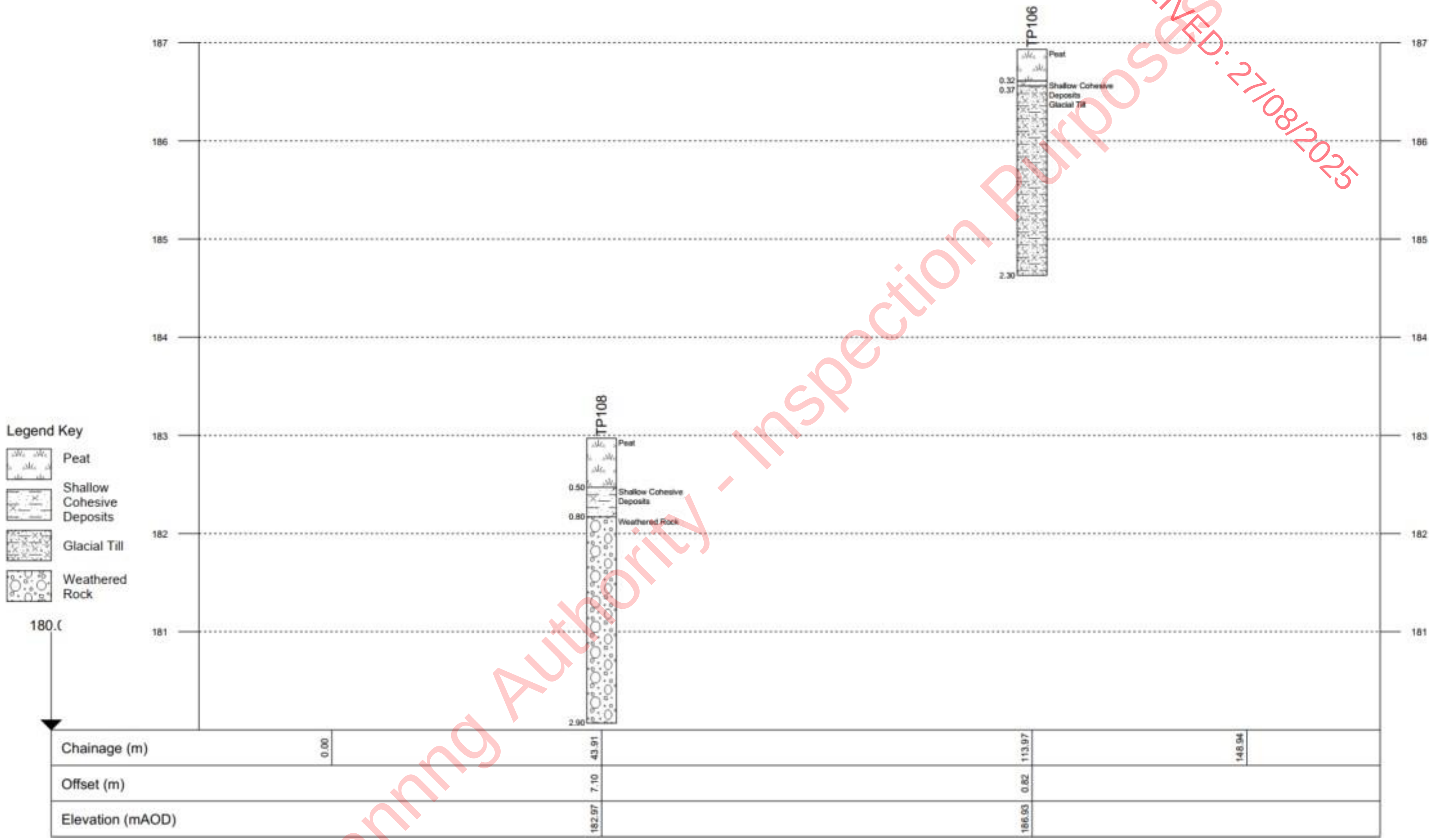

Figure 3-3: Borrow Pit 1 geotechnical cross-section

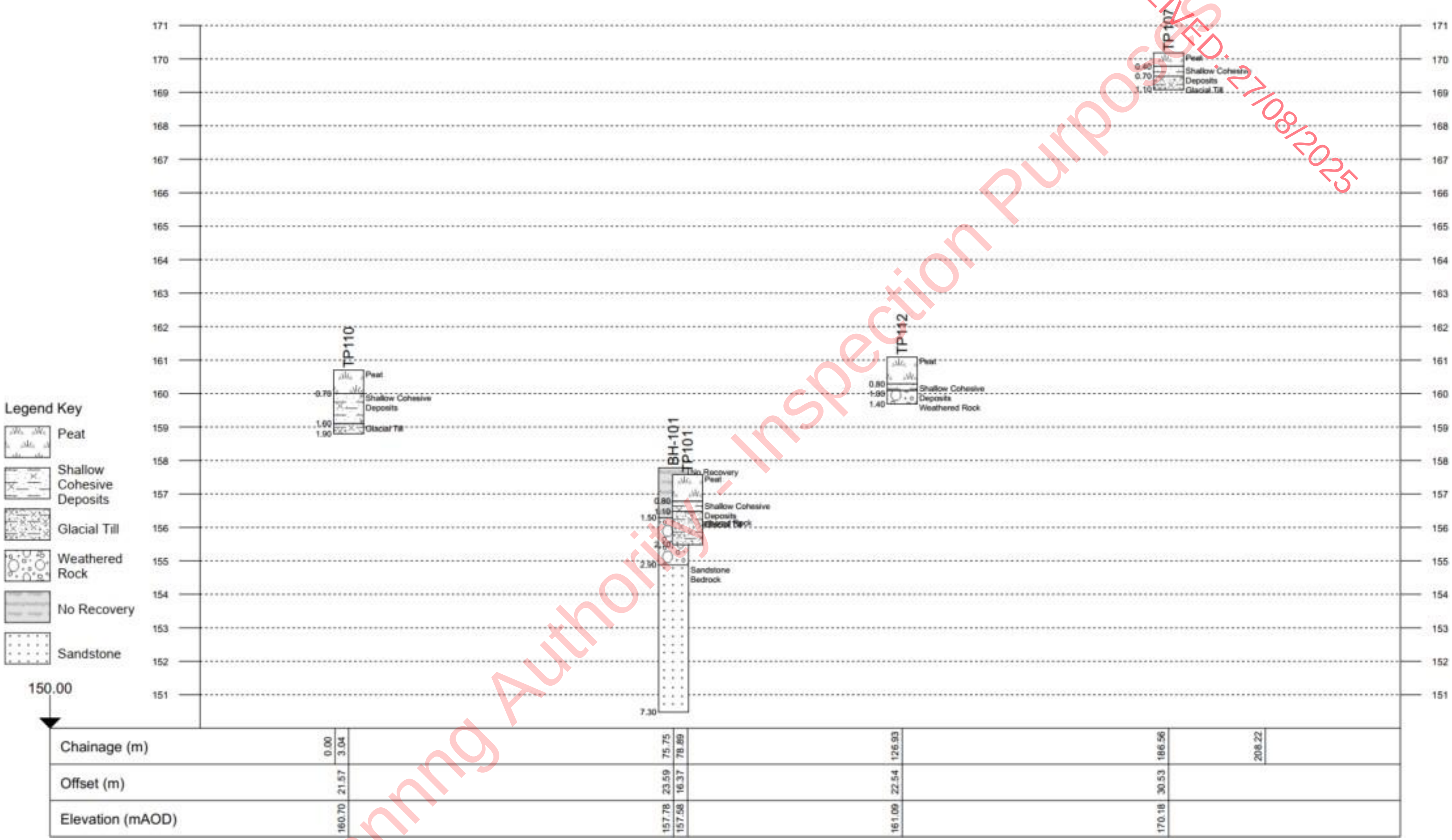

Figure 3-4: WTG1 and Borrow Pit 2 geotechnical cross-section

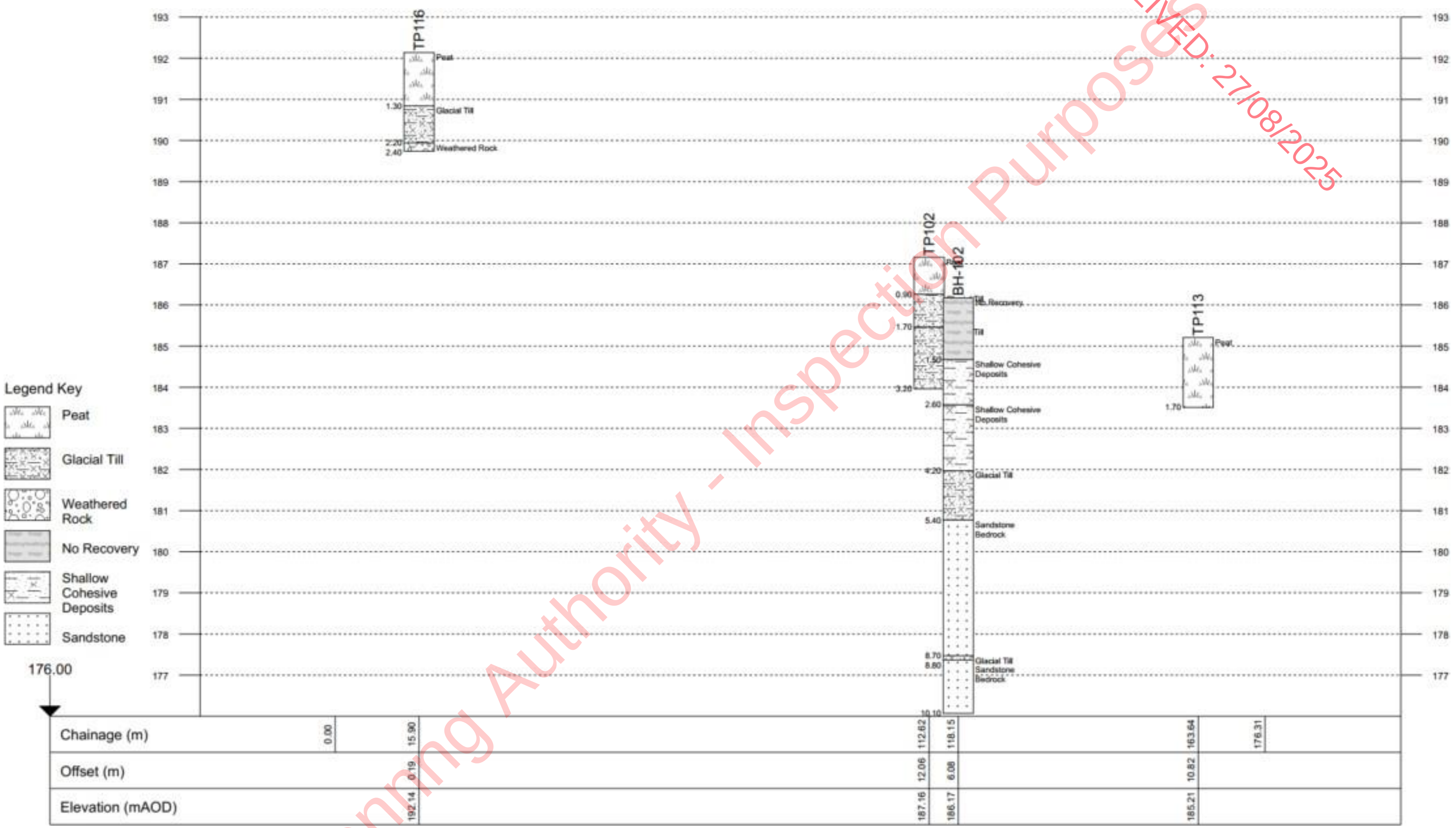

Figure 3-5: WTG3 geotechnical cross-section

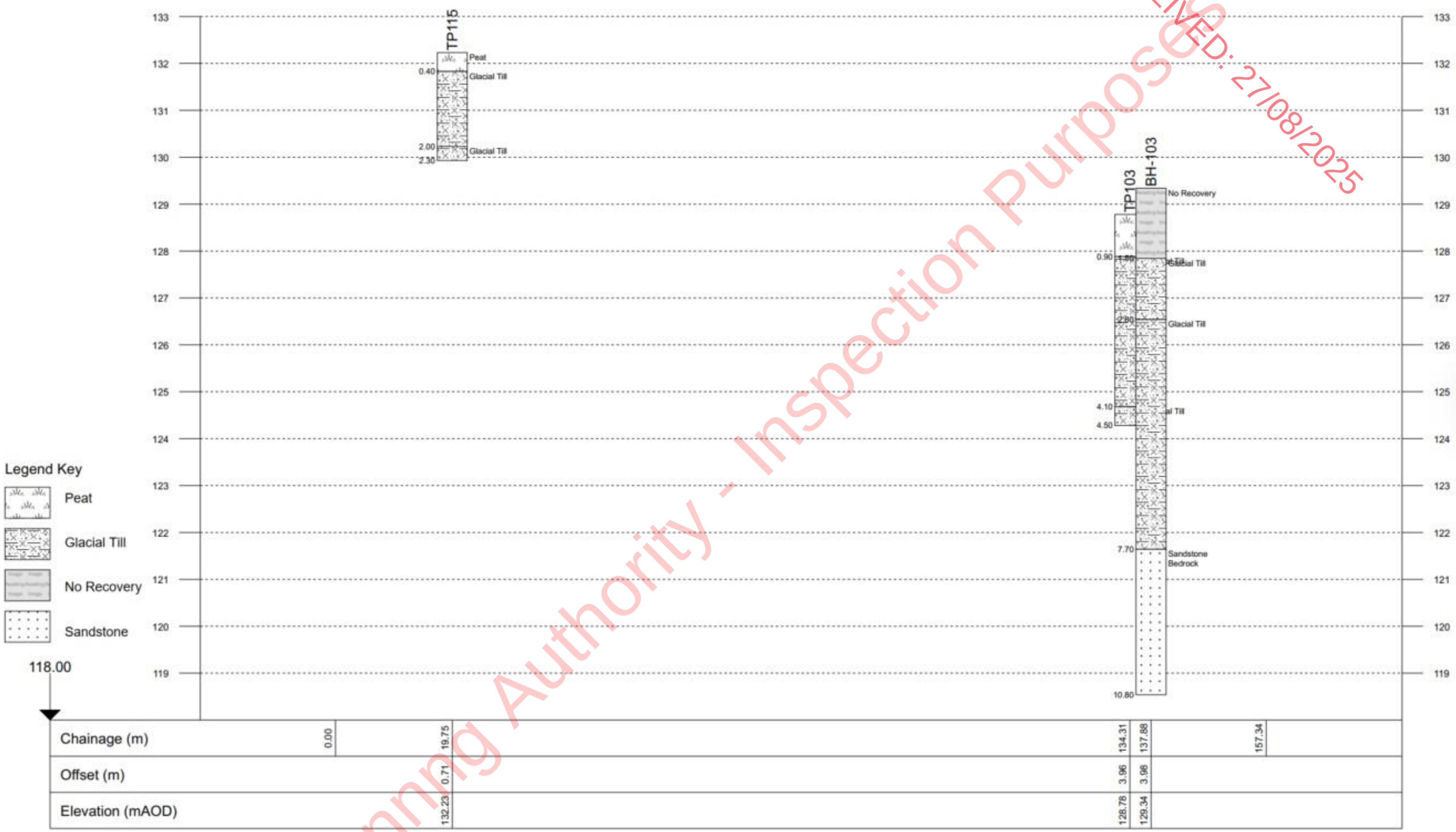

Figure 3-6: WTG4 geotechnical cross-section

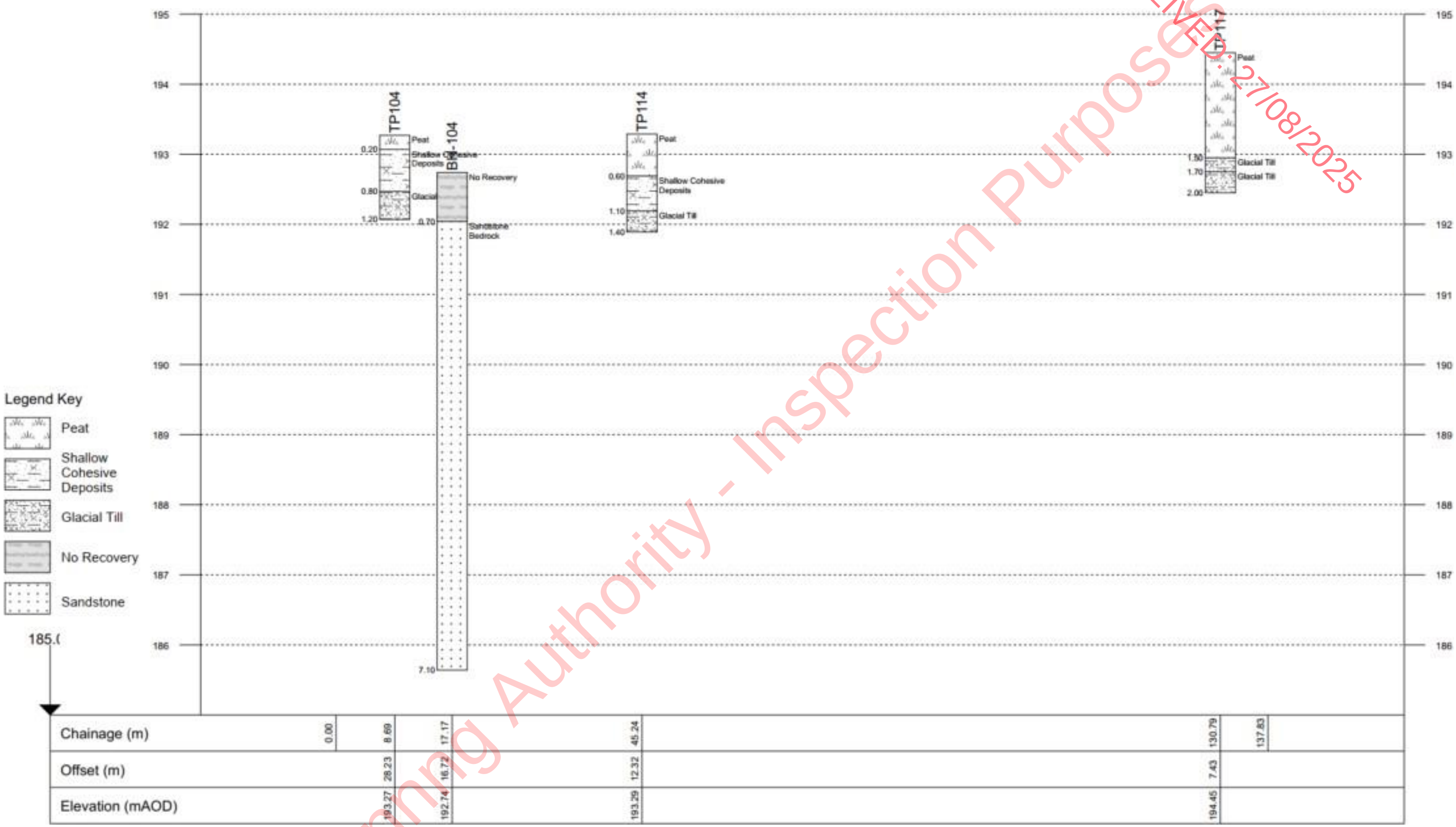

Figure 3-7: WTG6 geotechnical cross-section

3.3 GROUNDWATER CONDITIONS

The groundwater conditions across the TP's indicate varying levels of water seepage and ingress. Trial pit locations; TP101, TP102, TP104, TP115, and TP117 encountered water seepage between 1.2m to 3.2m bgl. In addition, TP105, TT106, and TP113 exhibited surface water ingress. The remaining trial pits did not yield any notable groundwater observations during the excavation processes. Four water monitoring standpipes were installed within the rotary hole locations during backfilling. Water levels were measured on 12/12/24, exhibiting water levels of 0.05m, 0.91m, 0.00m and 1.41m bgl for BH101, BH102, BH103 and BH104, respectively.

It should be noted at the time of this report only one set of readings has been collected. Water levels can vary due to diurnal or seasonal variations, weather conditions, and/or other environmental effects and may at times differ from those recorded during the investigation. Therefore, a conservative groundwater level is recommended for design to mitigate against possible increases in porewater pressures or reductions in design resistances. As a minimum, the design groundwater levels should coincide with the upper-bound groundwater profile recorded near the proposed design element. For design purposes, a conservative groundwater level may be assumed to be at existing ground level, i.e. 0m bgl.

4 IN-SITU TESTS

4.1 STANDARD PENETRATION TEST

Standard penetration testing (SPT) was carried out in boreholes BH102 and BH103 at approximately 1.5m intervals to the full depth of overburden within the boreholes. The energy ratio for the SPT hammer used for the GI was not provided in the factual GI report. Therefore, the SPT N values discussed in this GIR are uncorrected and were treated with due caution. A plot of the uncorrected SPT N profile from the boreholes is shown in Figure 4-1 and the data are summarised in Table 4-1. It should be noted that several refusals were recorded during the SPT testing in the Glacial Tills likely due to the presence of obstructions, such as cobbles and boulders, which is typical of Irish Glacial Tills. To illustrate the refusals on the SPT plot, they have been plotted as an SPT N of 50 alongside the penetration achieved during the test.

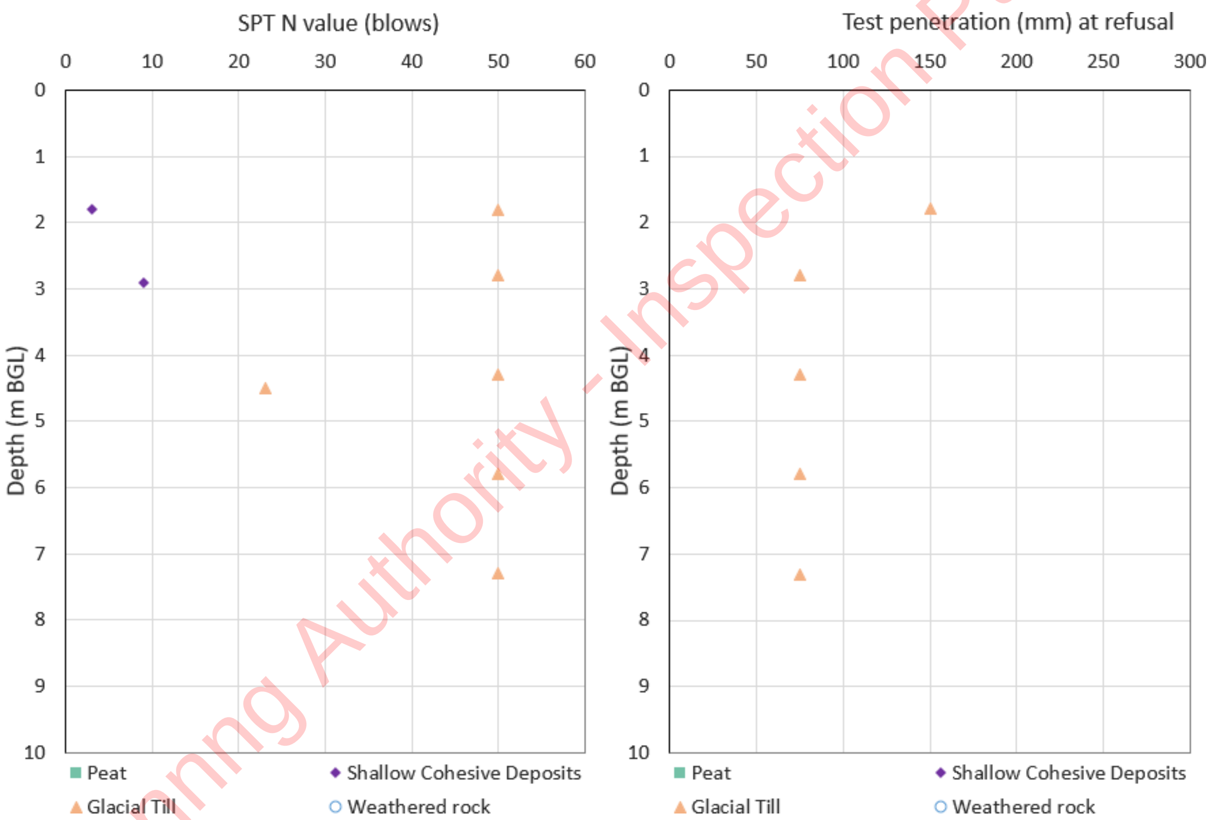

Figure 4-1: Standard Penetration SPT-N value

Table 4-1: Summary of SPT results

Stratum	Test Count	Min	Average	Max	No. of refusals
Peat	0	--	--	--	--
Shallow Cohesive Deposits	2	3	6	9	0
Glacial Till	6	23	--	Refusal	5

Sandstone	0	--	--	--	--

4.2 HAND SHEAR VANE TEST

IDL attempted to complete 43no. hand shear vane tests across the site. However, four locations (HSV1, HSV2, HSV10 and HSV11) were inaccessible during the GI campaign, while the shear vane refused penetration at shallow depths (less than 0.1m) at six other locations (HSV3, HSV6, HSV7, HSV8, HSV15 and HSV19). The refusals were likely due to the presence of large roots, cobbles and boulders in the near subsurface. The successful hand shear vane tests were completed in the Peat, Shallow Cohesive Deposits, Glacial Till and Weathered Rock with summaries of hand shear vane test results given in Table 4-2, Table 4-3 and Table 4-4. The strata anticipated at each test location and depth were based on the visual observations made in nearby trial pits and Russian Core Augers.

18no. hand shear vane tests were carried out in the Peat layer with the peak undrained shear strength (c_u) measured to be between 4 kPa to 40 kPa at depths ranging between 0.2m bgl and 1.3m bgl. Seven tests were carried out in the Shallow Cohesive Deposit strata with the peak c_u measured to be between 35 kPa to 62 kPa at depths ranging between 0.1m bgl and 1.1m bgl. Seven tests were completed in the Glacial Till strata with the peak c_u measured to be between 32 kPa to 180 kPa at depths ranging between 0.5m bgl and 2.3m bgl. One test was completed in TP08 at a depth of 2.0m bgl (or 180.97m OD) within a stratum interpreted to be Weathered Rock with a peak c_u value measured at 40 kPa. However, this test was likely completed in a small pocket/seam of cohesive material within the Weathered Rock mass and was, therefore, not deemed representative of the stratum.

Table 4-2: Summary of Hand Shear Vane test results in Peat

Hand Shear Vane or Location ID	Depth(m)	Test Level(m OD)	Peak undrained shear strength (kPa)	Residual undrained shear strength (kPa)
HSV12	0.5	185.5	15	1
HSV13	0.3	186.4	6	0
HSV14	0.2	192.5	38	2
HSV16	0.6	193.2	15	2
HSV17	0.7	194.3	40	3
HSV18	0.6	194.5	14	2
TP101	0.8	156.78	10	--
TP102	0.4	186.76	10	--
TP105	1.2	193.87	10	--
TP110	0.5	160.20	20	--
TP111	1.0	177.33	30	--
TP111	2.0	176.33	20	--
TP111	3.0	175.33	15	--

TP112	0.5	160.59	30	--
TP113	1.0	184.21	5	--
TP113	1.3	183.71	4	--
TP116	1.0	191.14	22	--
TP117	1.0	193.45	5	--
		Min	4	0
		Max	40	3
		Average	17	1.5

Table 4-3: Summary of Hand Shear Vane test results in Shallow Cohesive Deposits

Hand Shear Vane or Location ID	Depth(m)	Test Levelm OD	Peak undrained shear strength(kPa)	Residual undrained shear strength(kPa)
TP101	1.1	156.48	40	--
TP107	0.4	169.78	40	--
TP110	0.8	159.90	60	--
TP112	0.8	160.29	45	--
TP114	0.6	192.69	62	--
HSV5	0.2	169.2	50	10
HSV9	0.5	174.9	35	7
		Min	35	7
		Max	62	10
		Average	47.5	8.5

Table 4-4: Summary of Hand Shear Vane test results in Glacial Till

Hand Shear Vane or Location ID	Depth(m)	Test Levelm OD	Peak undrained shear strength (kPa)	Residual undrained shear strength (kPa)
TP103	1.5	126.28	62	--
TP104	1.2	192.07	73	--
TP106	2.3	184.63	42	--
TP115	0.5	131.73	180	--
TP115	2.0	130.23	177	--
HSV4	1.0	169.5	32	2
HSV4	1.4	169.5	32	2
		Min	32	2
		Max	180	2
		Average	85.5	2

5 LABORATORY TEST

A range of geotechnical tests were conducted on selected samples collected from boreholes and trial pits to assist in classifying soils and inform the proposed characteristic geotechnical parameters. The results and interpretation of these tests are presented in the following subsections. The geotechnical laboratory testing carried out by IDL included the following:

- Particle Size Distribution (PSD) analysis, including Hydrometer testing;
- Atterberg limit testing;
- Moisture content determination;
- Point Load tests;
- Uniaxial Compressive Strength (UCS);
- Soil chemistry including organic content, pH, Sulphate (Water Gravimetric), Chloride content, andAcid soluble sulphate content testing.

5.1 PARTICLE SIZE DISTRIBUTION AND HYDROMETER

Particle size distribution (PSD) classification testing was completed by IDL on nine of the samples. Samples were collected within the Glacial Till. PSD samples locations, recovery depths and strata are shown in Table 5-1 while the grading curves and constituent percentages obtained from the PSD results are illustrated in Figure 5-1 and Figure 5-2.

Only one of the samples subjected to PSD testing was recovered from the Shallow Cohesive Deposit strata. The PSD indicated the material is likely cohesive with a high fines content of 58% (49% silt-sized particles and 9% clay-sized particles).

Six samples recovered from the Glacial Till strata were subjected to PSD testing. The results of these tests suggested a wide spread of grading curves suggesting the material is comprised of cohesive, granular or mixed materials.

Two samples subjected to PSD testing were recovered from the Weathered Rock strata. The PSD indicated the material is largely comprised of course granular particle sizes. The percentage by mass of gravel and cobble within the samples was greater than 85% and the maximum fines content was 6%.

Table 5-1: PSD samples

Location	Surface Elevation (m OD)	Sample Depth (m)	Sample Elevation (m OD)	Stratum
TP101	157.58	0.8	156.78	Shallow Cohesive Deposits
TP102	187.16	3.0	184.16	Glacial Till
TP103	128.78	1.5	127.28	Glacial Till

TP104	193.27	0.8	192.47	Glacial Till
TP106	186.93	1.0	185.93	Glacial Till
TP107	170.18	0.7	169.48	Glacial Till
TP108	182.97	2.0	180.97	Weathered Rock
TP112	161.09	1.0	160.09	Weathered Rock
TP115	132.23	0.5	131.73	Glacial Till

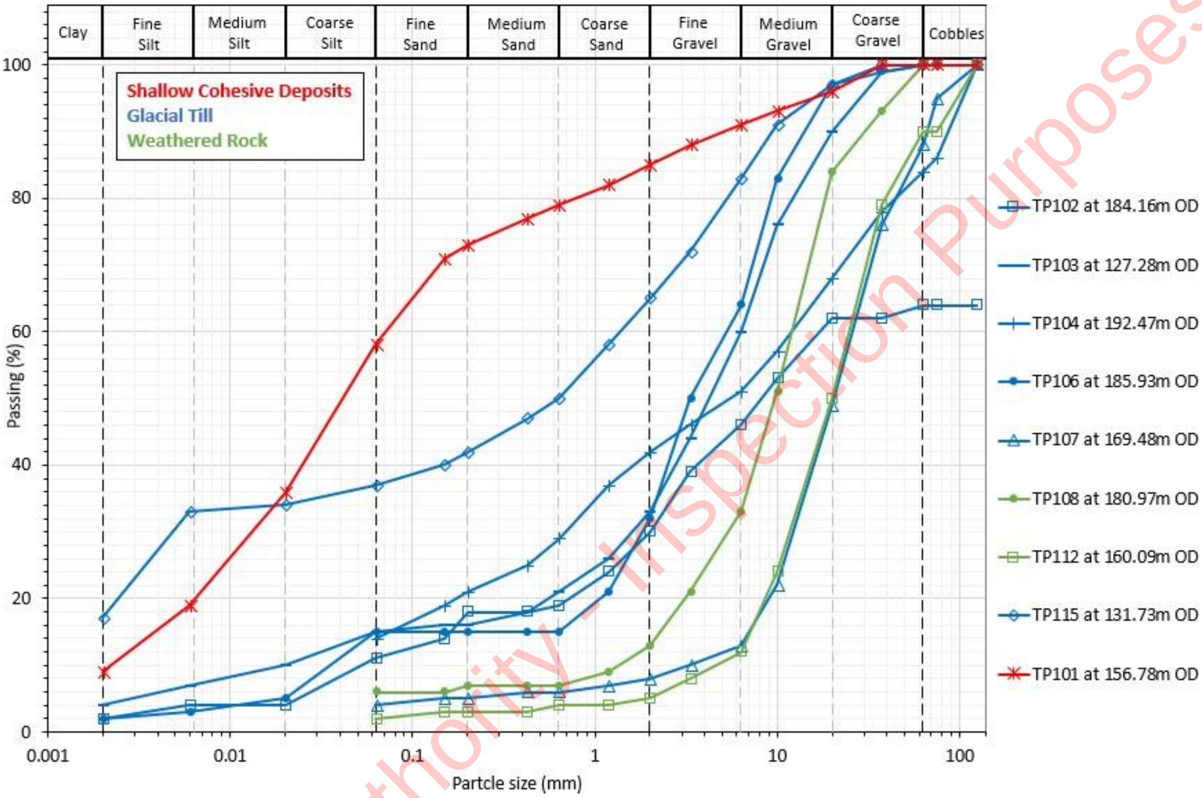

Figure 5-1: PSD results of Cohesive and Granular Glacial Till

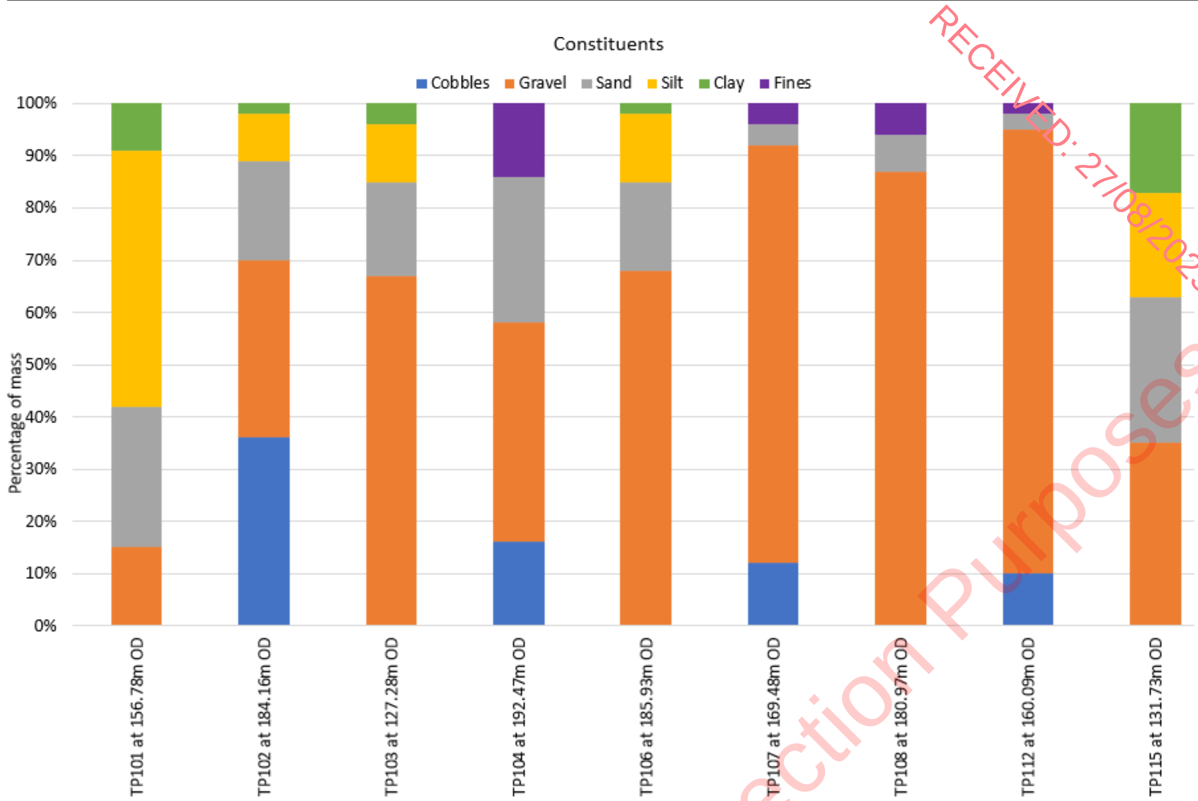

Figure 5-2: PSD fraction distribution

5.2 MOISTURE CONTENT AND ATTERBERG LIMIT TESTS

Moisture content (MC) testing was completed on 13no. samples recovered from the Site and 12no. were subjected to Atterberg Limit testing. The sole sample to be excluded from the Atterberg Limit testing was that recovered from the Glacial Till stratum encountered within TP104 as this material was a very sandy silty GRAVEL and was likely unsuitable for Atterberg Limit testing. The Atterberg limit testing was completed to determine the Liquid Limit (LL) and Plastic Limit (PL) values for the cohesive material of each sample having removed all particles greater than 425µm in diameter. The Plasticity Index (PI) was then calculated as the difference between the LL and PL of each sample.

The Atterberg limits and the percentage by mass of the samples passing the 425µm sieve have been plotted against depth in Figure 5-3 and Figure 5-4, with a summary of the minimum, average and maximum values for the Atterberg limits test results shown in Table 5-2. In addition, the PI values have been plotted against LL as shown on the plasticity chart in Figure 5-5.

The PL and LL values for the Shallow Cohesive Deposits were relatively consistent across the four samples. Three of the MC values measured on these samples were typically near the PL, suggesting the material is relatively low saturation and likely to be of medium to high strength. One outlying sample produced a much higher moisture content of 58.6% although this sample was recovered from a depth near the interface with the overlying peat. This sample was likely subject to wetting due to groundwater infiltration from the overlying Peat which should be considered when assigning soil parameters to the upper depth of this stratum. The relevant data points on the plasticity chart show that the Shallow Cohesive Deposit samples are generally of intermediate to high plasticity suggesting the material has medium compressibility.

The PL and LL values for the Glacial Till were relatively consistent across the eight samples. However, one outlier suggested a LL (91%) and a PL (48%) much higher than the other samples and even higher than the Shallow Cohesive Deposits. The sample was recovered from the centre of the Glacial Till stratum in TP102, although it was also noted that only 55% of the sample passed the 425µm sieve. As such, this sample was deemed to be unrepresentative of the Glacial Till mass. The MC values measured on the Glacial Till samples were typically lower than the PL for the material, suggesting the material is relatively low saturation and likely to be of high to very high strength. The relevant data points on the plasticity chart show that the Glacial Till samples are generally of low to intermediate plasticity suggesting the material has low compressibility.

Table 5-2: Summary of Liquid limit, Plastic limit and Plasticity Index test results

Stratum	Moisture Content(%)			Liquid Limit (%)			Plastic Limit(%)			Plasticity index(%)		
	Min	Ave	Max	Min	Ave	Max	Min	Ave	Max	Min	Ave	Max
Shallow Cohesive Deposits	24.8	38.3	58.6	44	49	57	29	33	37	15	17	20
Glacial Till	10.8	17.4	31.7	24	35*	91	17	22*	48	7	14*	43

*** Note**: The average Atterberg Limit values for the Glacial Till did not consider the outlying results measured using the sample recovered from TP102 at 2.0m bgl.

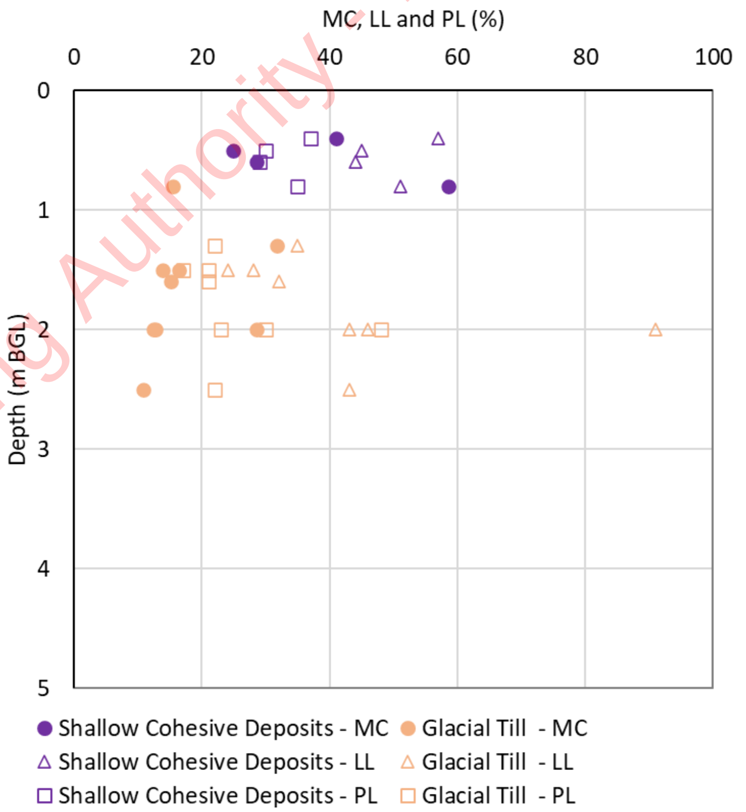

Figure 5-3: Atterberg Limits results

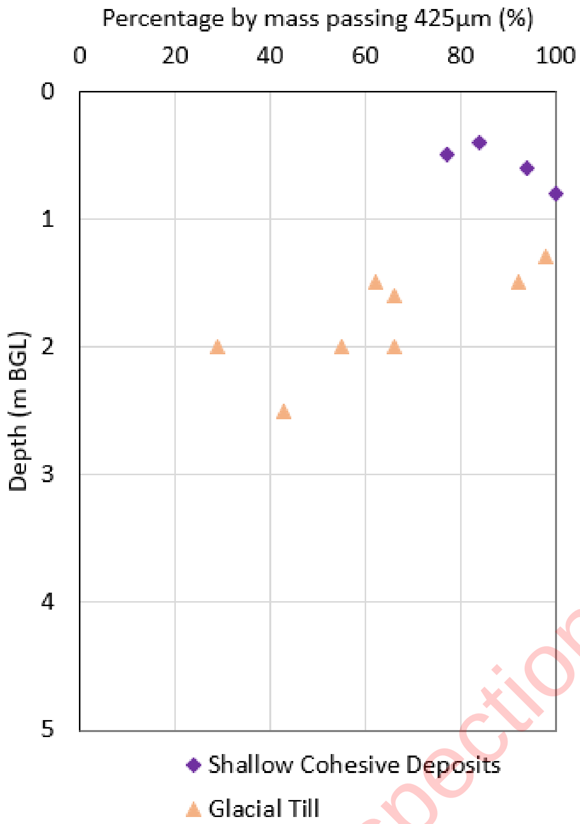

Figure 5-4: Percentage by mass passing the 425µm sieve for the Atterberg Limit test

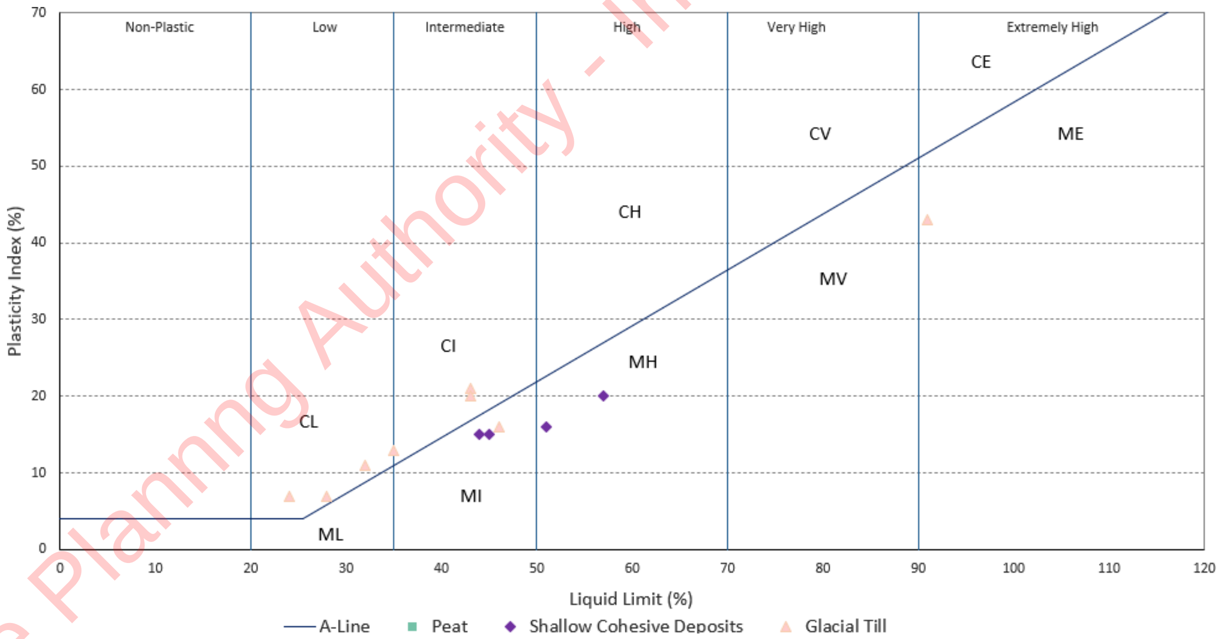

Figure 5-5: Plasticity chart following Atterberg Limit testing

5.3 CHEMICAL TESTING

Seven samples were collected from six locations ranging from recovery depths of 0.4m to 2.0m BGL. The pH, water-soluble Sulphate, acid-soluble Sulphates and organic matter were assessed by Envirolab on behalf of IDL, with the results summarised in Table 5-3. The relevant designer should consider the results of the pH, water-soluble Sulphate and Acid Soluble Sulphate tests when selecting the materials grades required to achieve the necessary design life durability.

Three peat samples demonstrated high organic contents with percentages ranging between 47.5% and 71.3 %, which is typical of peat material, as it typically contains decomposed plant and animal materials, as well as microbial activity in accordance with BS 5930:2015. Two samples of the Shallow Cohesive Deposits demonstrated organic contents of 5.3% and 17.9%. According to the IDL Factual report, Marly SILT/CLAY is described as having dark organic content within the silty material. It was noted that the two samples tested were located near the interface with Peat and as such the organic content may have been related to plant material present before the development of the overlying peat layer. Alternatively, the samples may have been contaminated by Peat material during recovery.

Table 5-3: Chemical testing results

Location	Sample depth (m)	Sample elevation(m OD)	Sample Description	pH	Water soluble Sulphate (mg/l)	Acid soluble sulphate (mg/kg)	Organic Matter (% w/w)
TP102	1.0	186.16	Glacial Till	5.21	30	510	-
TP103	0.9	127.88	Glacial Till	-	30	340	-
TP107	0.4	169.78	Shallow Cohesive Deposits	-	-	-	17.9
TP111	1.0	177.33	Peat	-	-	-	63.3
TP111	2.0	176.33	Peat	5.30	80	4,000	71.4
TP112	0.8	160.29	Shallow Cohesive Deposits	-	-	-	5.3
TP113	1.0	184.21	Peat	-	-	-	47.5

5.4 ROCK TESTING

5.4.1 UNIXACIAL COMPRESSIVE STRENGTH

Three Uniaxial Compressive Strength (UCS) were completed on selected rock cores to assess the bedrock underlying the Site. The UCS tests are summarised in Table 5-4 and illustrated in Figure 5-6. UCS values range from 29.3 to 211 MPa with an average of 130.4 MPa, suggesting the rock is moderately strong to very strong.

Table 5-4: Uniaxial Compressive Strength

Location ID	Sample depth(m)	Sample elevation (mOD)	Rock Type	Sample elevation (mOD)	Uniaxial Compressive Strength (MPa)
BH-102	5.6	180.57	Sandstone	78.32	211
BH-103	7.3	122.04	Sandstone	76.62	29.3
BH-104	2.6	190.14	Sandstone	105.07	151

5.4.2 POINT LOAD STRENGTH INDEX

Point Load Strength Index (PLSI) testing was undertaken on selected rock cores to assess the bedrock underlying the Site. Core was obtained from all four of the rotary core boreholes. The equivalent UCS was estimated using the following equation:

$UCS = k \times I_{s(50)}$

where factor k = 20 and $I_{s(50)}$ the PLSI value for a core diameter of 50 mm.

The results of the PLSI testing are summarised in Table 5-5 and plotted in Figure 5-6 against the depth at which the sample was obtained. $I_{s(50)}$ values typically ranged from 2.6 MPa to 10.2 MPa, with an average of 4.5 MPa, and a low outlying value of 0.2 MPa. The typical rock samples can thus be characterised to be from a strong to very strong rock, with the low outlier being characterised as weak.

Table 5-5: PLSI Test Results

Location	Sample depth (m)	Sample elevation (mOD)	Rock Type	$I_{s(50)}$ (MPa)	Equivalent UCS (MPa)
BH-101	2.7	155.08	Sandstone	3.7	74
BH-101	4.3	153.48	Sandstone	3.1	62
BH-101	5.7	152.08	Sandstone	6.8	136
BH-102	5.6	180.57	Sandstone	5.2	104
BH-102	7.1	179.07	Sandstone	10.2	204
BH-102	8.6	177.57	Sandstone	5.6	112
BH-103	7.3	122.04	Sandstone	2.6	52
BH-103	8.8	120.54	Sandstone	2.8	56
BH-103	10.3	119.04	Sandstone	3.5	70
BH-104	1.1	191.64	Sandstone	4.9	98
BH-104	2.6	190.14	Sandstone	0.2	4
BH-104	4.1	188.64	Sandstone	4.7	94

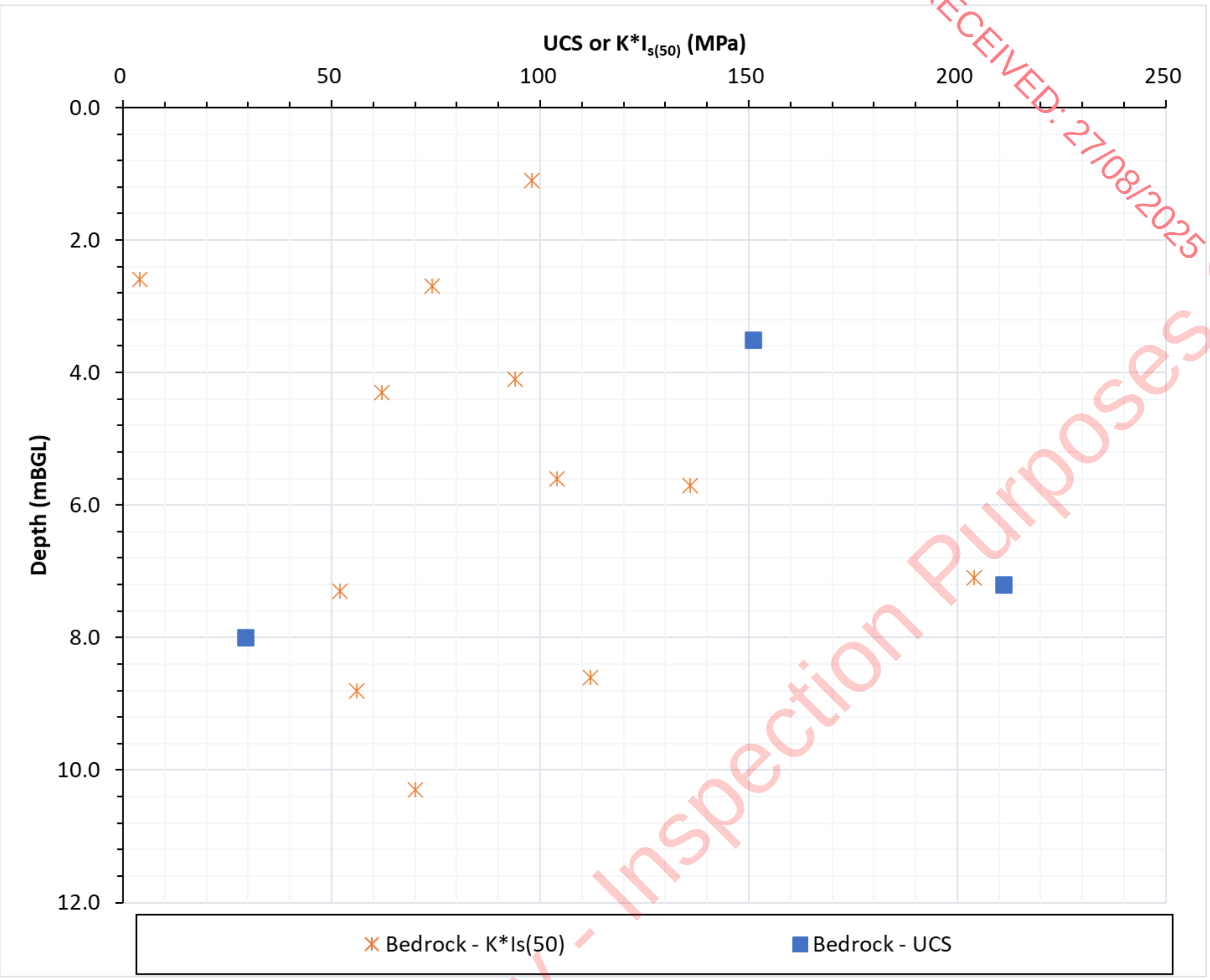

Figure 5-6: USC profile of sandstone

6 CHARACTERISTIC GEOTECHNICAL PARAMETERS

Characteristic geotechnical parameters have been recommended for use in the planning stage geotechnical design of the wind farm infrastructure. It should be noted that the current GI information is limited, and the recommended characteristic geotechnical parameters should be treated with caution. Characteristic geotechnical parameters have been assigned to materials based on the limit state required for the relevant piece of wind farm infrastructure. The following assumptions have been made regarding the preliminary characteristic geotechnical parameters:

- The WTG foundations, crane hardstands, substation platform, substation control building(s) and transformer foundations are to be constructed directly on either Glacial Till, Weathered Rock or Bedrock. Therefore, the strength and stiffness parameters for these three materials have been recommended.
- In addition to the Glacial Till, Weathered Rock or Bedrock, the access tracks and temporary compound may be constructed over Peat (floated construction) and Shallow Cohesive Deposits. Therefore, the strength parameters for these two materials have been recommended.

The characteristic geotechnical parameters recommended herein are based on measured and derived values of ground properties along with relevant correlations or published values. A combination of in-situ tests such as hand shear vanes, available laboratory test results and empirical correlations from the literature were used to derive the site-wide soil parameters of each stratum encountered across the examined Site.

The characteristic values have been assessed to be cautious estimates of the value governing the limit state. The selected values may be the best estimate of the probable value (e.g. unit weight), the low estimate (e.g. strength and stiffness parameters for settlement estimates) or the high estimate. The best estimate values may be considered as characteristic values for engineering behaviour where 'average' properties are most relevant for the limit state under consideration. Upper and lower bound estimate values have been derived using engineering judgement to provide a credible indication of the low and high distribution of the parameters, respectively. These parameters are not intended to represent absolute lower bound and upper bound lines, respectively but rather indicative values that might be used for specific design purposes.

It is recommended that additional GI campaigns are completed in advance of the detailed design. This additional GI is to provide greater confidence in the characteristic geotechnical parameters and to provide location-specific ground models for the critical infrastructure (WTG locations, the substation, etc.).

6.1 SELECTION OF CHARACTERISTIC SOIL PARAMETERS

6.1.1 UNIT WEIGHT

The unit weights (γ) of the underlying strata were assessed from:

- Engineering experience of Irish Peat materials including Long (2007), Long & Boylan (2013) and Quigley et al. (2016);
- The exploratory hole log descriptions in combination with BS 8004:2015 (Figure 6-1); and
- Engineering experience of Irish Glacial materials including Skipper et al. (2005), Long & Menkiti (2007), Long et al. (2012) and Farrell (2016).

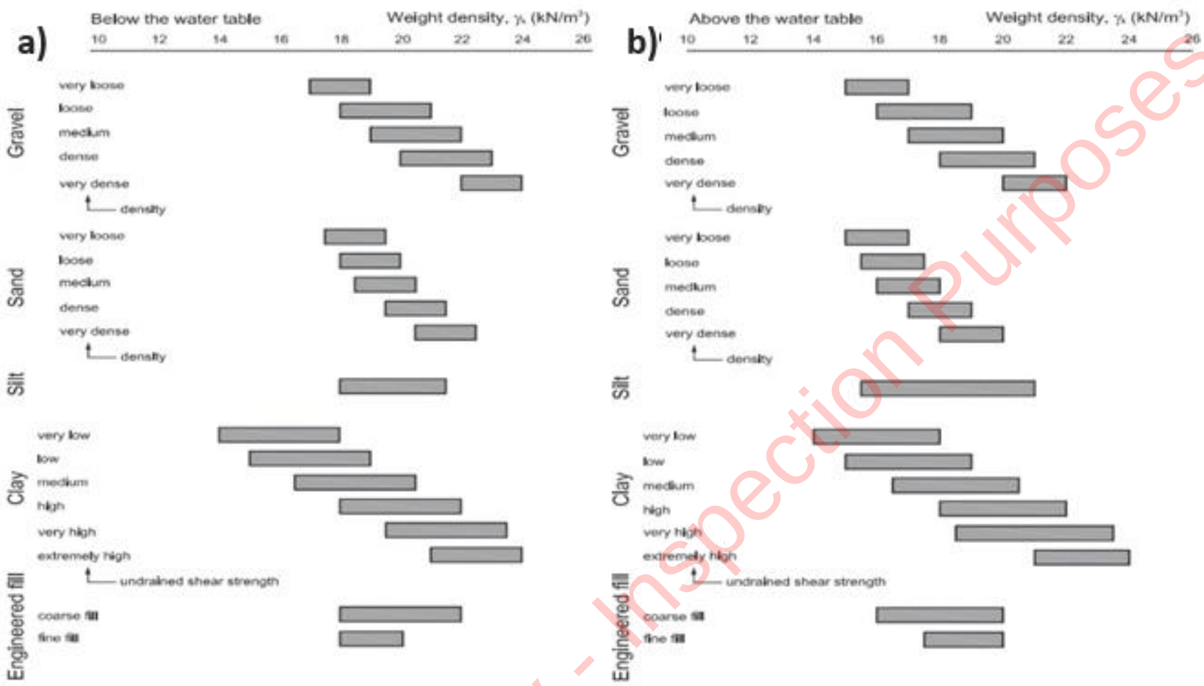

Figure 6-1: a) Dry and b) Bulk unit weight (derived from BS 8004:2015)

The estimation of the geotechnical parameters of Peat can be difficult due to the natural variability of the material. Long (2007) recorded unit weights between 15 kN/m^3 and 18 kN/m^3 for peat/peaty silt in estuarine sites in Sligo and Cork. Long & Boylan (2013) presented the densities recorded during one-dimensional Oedometer testing of peat samples from numerous sites around Ireland including Carrick-on-Shannon. When converted to unit weights the values measured ranged between 9.0 kN/m^3 and 11.6 kN/m^3. Quigley et al. (2016) recorded unit weights of approximately 10kN/m^3 for peat encountered in the Terryland River Valley, Galway. The best estimate of the characteristic γ_{sat} value of the peat is recommended to be 11 kN/m^3 for the preliminary design but should be treated with caution due to a lack of available data.

The dry and bulk unit weights for the cohesive, granular and mixed strata of the Shallow Cohesive Deposits and Glacial were reviewed using Figure 1 and Figure 2 of BS 8004:2015 (reproduced in Figure 6-1). These figures present broad ranges of the unit weights of typical soil types above the groundwater line (γ_{dry}) and below the groundwater line (γ_{sat}) based on their consistency as described in the exploratory hole logs. Farrell (2016) presented the unit weights of Irish glacial and interglacial soils to be between 20kN/m^3 and 22.5kN/m^3. The characteristic γ_{sat} value of the Shallow Cohesive Deposits and Glacial Till is recommended to be 18 kN/m^3 and 20 kN/m^3 respectively.

6.1.2 UNDRAINED SHEAR STRENGTH

The undrained shear strength (c_u) of the cohesive deposits has been assessed from:

- In situ hand shear vane test results;
- Engineering experience of Irish Peat materials including Long (2007), Long & Boylan (2013) and Quigley et al. (2016);
- The correlation with SPT N value after Stroud (1989); and
- Engineering experience of Irish Glacial Materials including Farrell et al. (1989), Donohue et al. (2003), Skipper et al. (2005), Long & Menkiti (2007), Long et al (2009), Long et al. (2012) and Farrell (2016).

The characteristic c_u values have been assessed to be near the low estimate of the value as this parameter typically contributes to design resistances. It should be noted that other limit states (e.g. pile driving) may require the use of a higher estimate. The designer of such limit states shall determine a suitable characteristic c_u estimate if and when required.

The in-situ hand shear vane measurements were completed in the Peat, Shallow Cohesive Deposits and Glacial Till as described in Section 4.2. These results were deemed the primary source for the selection of the preliminary characteristic c_u value for each stratum.

The c_u range for the Peat was estimated using the hand-shear vane test results and engineering experience presented in various peer-reviewed papers:

- Long (2007) recorded undrained shear strengths in the peat of Sligo and Cork as follows:
  - Using field vanes, c_u values were measured between 1 kPa and 40 kPa;
  - Using anisotropically consolidated undrained compression triaxial tests, c_u values were measured between 30 kPa and 50 kPa; and
  - Using the Lunne et al. (1981) correlations with CPTu data recorded with a piezocone head, c_u values were measured between 10 kPa and 60 kPa.
- Quigley et al. (2016) used shear vane testing to record undrained shear strengths ranging from 4 kPa to 10 kPa for peat encountered in the Terryland River Valley, Galway.

As Peat is highly variable and its strength is sensitive to soil saturation, it is recommended to use a conservative range of characteristic values for this stratum. The c_u range for the Peat is recommended to be taken between 5kPa and 40kPa. The Designer should select a suitable characteristic undrained shear strength value within the recommended range based on the location-specific test data and the limit state under consideration.

SPTs were completed in the Shallow Cohesive Deposits and Glacial Till as described in Section 4.1. It is noted from the PSDs that the Tills are highly variable, with granular and cohesive beds/lenses frequent throughout the soil masses. The delineation between the granular and cohesive material in such a soil mass at a specific SPT test depth is difficult to accurately complete. Hence, all SPT N values were assumed to be within a cohesive material. The c_u of inorganic fine-grained materials can be evaluated from the SPT N values using the Stroud (1989) formula:

$$c_u = f_1 \times N$$

Where f_1 is a correlation factor determined using the plot produced by Stroud (1989) which has been reproduced in Figure 6-2. This figure indicates that the following f_1 (or c_u/N) factor should be used for soil within the relevant plasticity index ranges:

- For the Cohesive Shallow Deposits with an I_p range of 15% to 20% an f_1 factor of 5 was taken; and
- For the Glacial Till with an I_p range of 7% and 25% an f_1 factor of 5 was taken.

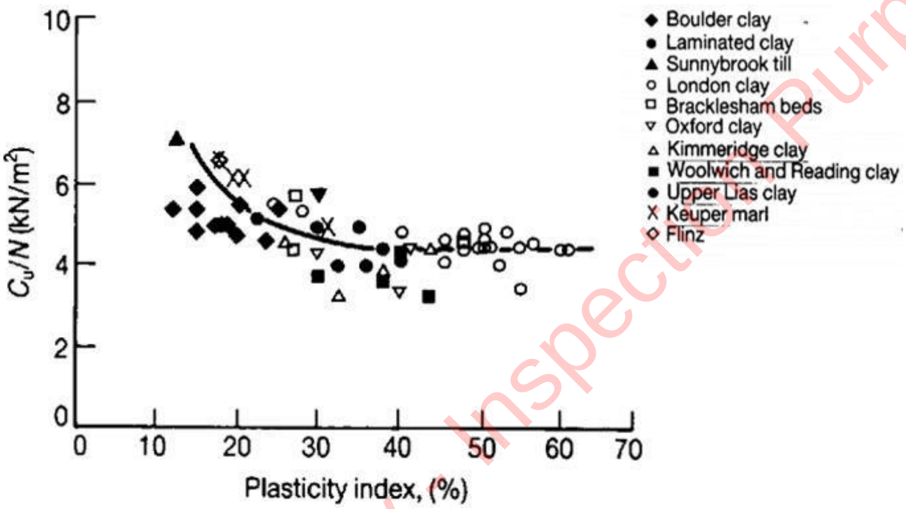

Figure 6-2: Correlation between SPT'N' and undrained shear strength (Stroud, 1989)

The c_u range for the Shallow Cohesive Deposits was estimated using the hand shear vane test results and SPT correlations. The hand-shear vane tests measured a c_u range of 35 kPa to 62 kPa. The c_u range for the Shallow Cohesive Deposits was also assessed using the Stroud (1989) correlation. With an f_1 factor of 5 used in combination with the SPT range of 3 to 9, the approximate c_u range is calculated as 15 kPa to 45 kPa. For the Shallow Cohesive Deposits, the undrained shear strength is recommended to be taken as 40 kPa. The Designer should select a suitable characteristic undrained shear strength value within the recommended range based on the location-specific test data and the limit state under consideration.

The c_u range for the Glacial Till was estimated using the hand shear vane test results, SPT correlations and engineering experience. The hand-shear vane tests measured a c_u range of 32 kPa to 180 kPa. The c_u range for the cohesive and mixed Glacial Till was also assessed using the Stroud (1989) correlation. With an f_1 factor of 5 used in combination with the SPT range of 23 to 50, the approximate c_u range is calculated as 115 kPa to 250 kPa. In addition to the Stroud correlation, guidance from the engineering experience of inorganic Irish Glacial Tills was also reviewed including Skipper et al. (2005), Long & Menkiti (2007), Long et al (2009), Long et al. (2012), Long et al. (2013) and Farrell (2016). The data presented by these authors indicate that the c_u value of Irish Glacial Tills has been measured between 100kPa and 1100kPa, although design values are often capped at

300kPa. For the Glacial Till, the characteristic undrained shear strength is recommended to be taken as 60kPa. The Designer should select a suitable characteristic undrained shear strength value within the recommended range based on the location-specific test data and the limit state under consideration.

6.1.3 ANGLE OF SHEARING RESISTANCE AND EFFECTIVE COHESION

The effective stress shear strength parameters of the overburden materials were assessed using:

- Engineering experience of Irish Peats including Hanrahan et al. (1967), Rowe and Mylleville (1996), Landva (1980a), Landva (1980b), Carling (1986), Farrel and Hebib (1998a), Farrel and Hebib (1998b), Rowe, Maclean and Soderman (1984b), McGreever and Farrel (1988a), McGreever and Farrel (1988b), Hungr and Evans (1985), Madison et al. (1996), Dykes and Kirk (2006), Warburton et al (2003a), Warburton et al (2003b), and Entec (2008);
- The Peck et al. (1974) correlation between SPT N and ϕ' for granular materials; and
- Engineering experience of Irish Glacial Materials including Farrell et al. (1989), Donohue et al. (2003), Skipper et al. (2005), Long & Menkiti (2007), Long et al (2009), Long et al. (2012) and Farrell (2016).

The characteristic effective angle of shearing resistance (ϕ') and effective cohesion (c') values have been assessed to be near the low estimate of the value as this parameter contributes to design resistances in the design. The Designer should select a suitable characteristic ϕ' value for each stratigraphic unit based on the location-specific test data and the limit state under consideration.

The in situ effective cohesion of the soils underlying the study area would be anticipated to vary both with depth and horizontal distance. This parameter is highly sensitive to saturation and disturbance and cannot be reliably correlated with previous experience. Therefore, c' is recommended to be conservatively taken as zero for all soil materials unless other methods are used to determine a satisfactory value (e.g. back-analysis of in situ conditions).

The φ' range for the Peat was estimated using engineering experience with reference to published values from literature as summarised in Table 6-1. These peer-reviewed sources presented typical c' values in the range of 0 to 10kPa, and angle of shearing resistance values in the range of 21° to 43° for peat. Following a review of the available literature, the characteristic c' of peat is recommended to be taken at 1kPa, and the ϕ' of the peat is recommended to be taken as 21°.

The ϕ' of granular materials may also be estimated using the relationship published by Peck et al. (1974). This relationship is typically used for soils with clean sands and gravels, and correlates the SPT N values and the effective angle of shearing resistance using the plot reproduced in Figure 6-3. It is noted that the relationship is defined for SPT N values greater than 10 and lower values should be treated with caution.

Table 6-1: Angle of shearing resistance and cohesive for peat from the literature

Reference	Cohesion, c’ (kPa)	Angle of shearing resistance, ϕ’ (°)
Hanrahan et al. (1967)	5 to 7	36 to 43
Rowe and Mylleville (1996)	2.5	28
Landva (1980)	2 to 4	27.1 to 32.5
Landva & Pheeney (1980)	5 to 6	-
Carling (1986)	6.5	0
Farrel and Hebib (1998a)	0	38
Farrel and Hebib (1998b)	0.61	31
Rowe, Maclean and Soderman (1984)	3	27
McGreever and Farrel (1988a)	6	38
McGreever and Farrel (1988b)	6	31
Hungr and Evans (1985)	3.3	-
Madison et al. (1996)	10	23
Dykes and Kirk (2006a)	3.2	30.4
Dykes and Kirk (2006b)	4	28.8
Warburton et al (2003a)	5	23.9
Warburton et al (2003b)	8.74	21
Entec (2008)	3.8	36.8

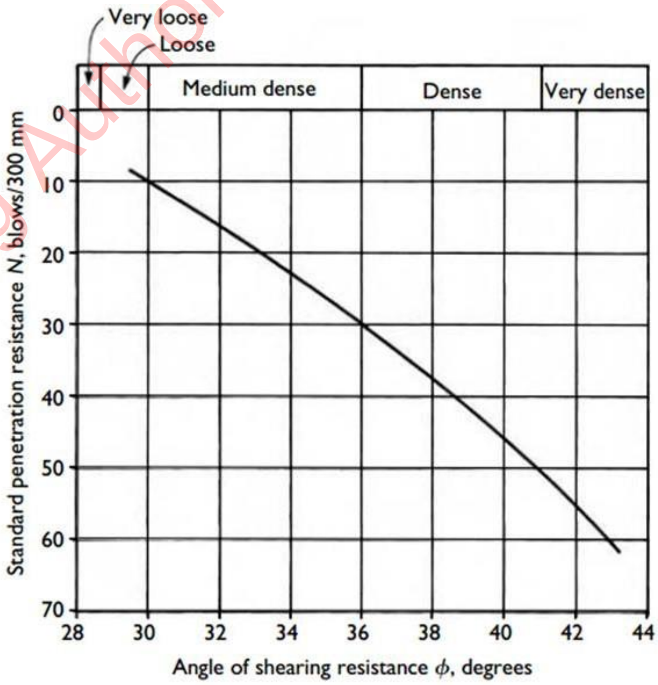

Figure 6-3: Peck’s relationship between SPT N and angle of shearing resistance (1974)

Following Peck et al. (1974), the characteristic ϕ' value of the Glacial Till was estimated to be 34° to 40° for the typical SPT N range of 23 to 50. However, as the Glacial Till contains cohesive content and beds/lenses, it is recommended to use a lower-bound value.

In addition to the above, guidance from the engineering experience of Irish Glacial Tills was also reviewed including Skipper et al. (2005), Long & Menkiti (2007), and Long et al. (2012). These peer-reviewed sources presented typical angles of shearing resistance values in the range of 34° to 38° for Irish Glacial Tills.

Following a review of the empirical correlations and the available literature, the characteristic ϕ' value for the Glacial Till is recommended to be taken as 32°. The Designer should select a suitable characteristic undrained shear strength value within the recommended range based on the location-specific test data and the limit state under consideration.

6.1.4 SOIL STIFFNESS

The Young's moduli of the overburden materials have been assessed using:

- The CIRIA Report 143 (1995) correlation between SPT N value and Young's modulus for cohesive and granular materials;
- The Clarke (2017) correlation between undrained shear strength and undrained Young's modulus (E_u) for inorganic cohesive materials; and
- The Clayton (2011) correlation between Poisson's ratio, undrained Young's modulus and drained Young's modulus (E') for inorganic cohesive materials.

The characteristic E_u and E' values have been assessed to be near the low estimate of the value as this parameter is typically used as a divisor to estimate ground movements during foundation design. It should be noted that other limit states may require the use of a higher estimate. The designer of such limit states shall determine a suitable E_u and E' estimate if and when required. The Designer should select suitable characteristic stiffness values for each material within the above ranges based on the location-specific test data and the limit state under consideration.

For granular material, E'/N (MPa) ratio values are summarised in CIRIA 143. CIRIA 143 states that for both normally consolidated and overconsolidated sands according to Stroud et al. (1989), with a factor of safety of 3 on bearing capacity, a reasonable approximation is:

$$\frac{E'}{N_{60}} = 1(MP\alpha)$$

However, the guidance further states; "But, judging from Stroud's re-analysis of the available case records, most foundations have a factor of safety considerably in excess of 3, so that for normally consolidated sands E'/N60 may rise to about 2 MPa and for over consolidated sands and gravels it may rise to 16, at very small strain levels." This is illustrated in Figure 6-4 which has been reproduced from CIRIA Report 143. It is anticipated that the ratio of unfactored bearing pressure to unfactored bearing resistance (q_{net}/q_{ult}) would be in the range of 0.1 to 0.3 for the underlying overconsolidated granular Glacial Till. Thus, the assumed value for E'/N was taken to be 2 MPa, noting that the N in this equation was taken as the uncorrected value.

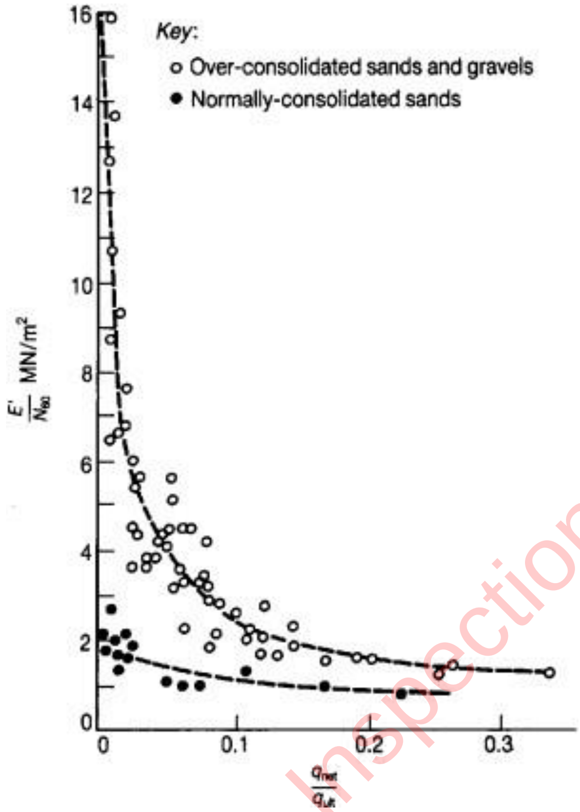

Figure 6-4: Relationship between drained static Young's modulus, penetration resistance and degree of loading (Stroud, 1989)

The drained and undrained Young's Moduli of the cohesive lenses of the Glacial Till were assessed following the recommendations of Clarke (2017). The undrained static Young's modulus (E_u) for the Cohesive Glacial Till materials is assumed to be in the range of 500 to 1500 times c_u. Conservatively, the correlation factor has been taken to be 500 times c_u. The long-term drained static Young's modulus (E') of the cohesive strata is based on the following relationship from Clayton (2011):

$$E' = \frac{1+\upsilon}{1.5} E_u$$

Where υ is Poisson's ratio and is assumed to be in the range of 0.2 to 0.35. Conservatively, υ was assumed to be 0.2 resulting in:

$$E' = 0.8E_u$$

The c_u range for the Glacial Till was estimated using the SPT correlations and engineering experience. The short-term E_u range of the cohesive and mixed Glacial Till was estimated to be 16 MPa to 90 MPa using the Clarke (2017) correlation. The long-term E' values of the cohesive Glacial Till were then estimated to be 13 MPa to 72 MPa using Clayton (2011). It is recommended to take an average E_u value of 50MPa and an average E' of 40MPa for the cohesive and mixed Glacial Till.

Using a correlation factor of 2 times N (ranging from 23 to 50), the characteristic E' range for the mixed and granular Glacial Till is estimated to be between 46 MPa to 100 MPa. This range is similar to the cohesive Glacial Till and therefore the Glacial Till is recommended to be modelled with the same E' of 40MPa for all consistency types.

6.2 SELECTION OF CHARACTERISTIC ROCK PARAMETERS

The characteristic rock parameters have been assessed using:

- The results from geotechnical laboratory testing;
- Published values in Waltham (2009) and Look (2014); and
- Correlations between RQD, UCS and effective stress strength parameters following Kulhawy and Goodman (1980 and 1987).

The characteristic rock parameters have been assessed to be near the low estimate of the value as this parameter typically contributes to design resistances.

Based on the geotechnical laboratory results presented in Section 5.4, a characteristic UCS value of 30MPa is proposed. This value broadly agrees with the ranges presented in Waltham (2009) and Look (2014) for Sandstone.

Waltham (2009) recommends a dry density of approximately 2.2 t/m^3, a friction angle of 40° to 45° and shear strength in the range of 4 MPa to 15 MPa for intact rock. Table 9.2 of Look (2014) proposes a typical unit weight range of 18kN/m^3 to 23kN/m^3 for an extremely to distinctly weathered Sandstone, and a range of 23 to 26 kN/m^3 for slightly weathered to Fresh Sandstone.

Table 9.12 of Look (2014) indicates the friction angle of limestone ranges between 25° (soft rock) to 45° (hard rock) for Sandstone and the cohesion ranges between 1 MPa (soft) to 30 MPa (hard).

The RQD values measured from the recovered cores were typically less than 70%. The correlations included in Kulhawy and Goodman (1980 and 1987) for a rock with RQD values typically ranging between 0% and 70% are a φ' of 30° and a c' equal to 0.1 times UCS (= 3,000 kPa)

Based on the various data sources, characteristic rock parameters are recommended as follows:

- Weathered Sandstone Rock: a unit weight of 20 kN/m^3, a friction angle of 34° and a cohesion of 0kPa.
- Intact Sandstone: a unit weight of 23 kN/m^3, a friction angle of 34° and a cohesion of 1,000 kPa.

6.3 SUMMARY OF CHARACTERISTIC GEOTECHNICAL PARAMETERS

The characteristic geotechnical parameters recommended for use in the preliminary design phase are summarised in Table 6-2. The majority of the characteristic parameters are typically based on low estimates, with the characteristic unit weights based on the best estimates. Variations from this table may be required for other limit states, temporary works designs and constructability-related assessments.

Table 6-2: Summary of characteristics geotechnical parameters (recommended values)

Parameter	Symbol (unit)	Characteristic value				
		Peat	Shallow Cohesive Deposits	GlacialTill	Weathered Rock	Sandstone
Unit weight	γ' (kN/m^3)	11	18	20	20	23
Undrained shear strength	c_u (kPa)	5-40 (site specific)	40	60	--	--
Effective cohesion	c' (kPa)	1	0	0	0	1,000
Effective friction angle	ϕ' (°)	21	30	32	34	34
Undrained Young's modulus	E_u (MPa)	--	--	50	-	-
Drained Young's modulus	E′ (MPa)	--	--	40	50	-
Unconfined compressive strength	UCS (MPa)	-	-	-	-	30

7 GAP ANALYSIS

In this section, a gap analysis was completed to identify any deficiencies or lack of necessary data in the existing information for the investigated plates/ground models. The gap analysis is based on the IDL's factual ground investigation report (no. revision 24_CE_108/02), issued in January 2025. Any additional factual information that may become available after the issue of this GIR shall be reviewed by the relevant designer and incorporated into the relevant design stage. Any additional information may result in alterations to the soil parameters proposed in this revision of the GIR. Recommendations are made for additional GI below. The expectation is that additional GI will be carried out after planning consent has been granted and prior to detailed design.

All of the rotary core holes, trial pits and Russian Core Augers scheduled have been completed and logs issued to GDG. AGS data has been provided for the exploratory holes, and the completed in-situ and geotechnical laboratory testing. It is noted that neither boreholes nor trial pits were completed at WTG2 and WTG5 due to access issues, the substation location was only subjected to two trial pits, and only one borehole was completed at each of the other four WTG locations. It is recommended that at least one borehole is completed at each of WTG2 and WTG5 to visually confirm a depth to competent bedrock. It is also recommended that additional trial pits and a borehole are completed to establish the variation of the ground conditions across the substation footprint and to confirm the depth of rock. Furthermore, it would be beneficial to complete geophysical profiling across the WTG footprint to establish the variation of the ground conditions for differential settlement calculations which are typically critical design requirements.

Continuous groundwater monitoring was not completed within the rotary core holes. As a result, the variation of the groundwater level across diurnal, lunar and seasonal periods was not established.

However, the groundwater strikes and seepage noted in the intrusive exploratory holes suggest that the groundwater levels are as shallow as the existing ground level, which would be typical of a peat-covered area. As such, this most onerous condition is likely satisfactory for the planning stage.

It is noted that advanced geotechnical tests (e.g. UU triaxial tests, CIU triaxial tests, oedometer tests, etc.) have not been completed to date. However, the recovery of undisturbed samples from the underlying sediments is likely to prove difficult. The Shallow Cohesive Deposits are typically a relatively thin stratum which is difficult to target for sampling unless continuous core sampling is carried out using a Geobor S rig. Additionally, the recovery to Irish Glacial Tills is known to be challenging, due to the variable grading of the material. In particular, the presence of gravels and cobbles results in either refusal of open tube samplers (UT100 or U100) and significant disturbance of cores recovered by continuous Geobor S or Sonic drill coring. It would be preferable to recover undisturbed samples of the Shallow Cohesive Deposits and Glacial Tills in locations where they are to be a bearing stratum. However, it is likely impractical and/or uneconomical to attempt to recover undisturbed samples based on the reported site conditions. The best approach for the geotechnical design may be to include geophysical surveying for in situ small-strain stiffness evaluation using techniques such as Seismic Refraction and Multichannel Analysis of Surface Waves (MASW), and/or construction stage testing and validation.

8 GEOTECHNICAL RISK REGISTER

GDG understand that under the Safety, Health & Welfare at Work (SHWW) (Construction) Regulations 2013, our duties as designers are generally to:

- Identify any hazards that the design may present.
- Where possible, eliminate the hazards or reduce the risk.
- Communicate necessary control measures, design assumptions or remaining risks to the Project supervisor Design Process (PSDP) so they can be dealt with in the Safety and Health Plan.
- Co-operate with other designers and the PSDP and Project Supervisor Construction Stage (PSCS) as appropriate.

The items included in Table 8-1 have been identified as plausible geotechnical risks and should be incorporated into any risk registers or assessments for the project as a whole. Mitigation measures have been recommended for each geotechnical risk. The recommended mitigation measures are not mandated as part of the design process nor override a designer's responsibility to assess and eliminate or mitigate risks identified in this GIR. The Designer of each design element shall be responsible for determining and designing the final mitigation measures at the detailed design stage.

The hazards and/or risks identified in the project GRR are not part of an exhaustive list. Additional hazards or risks may exist that have not been identified at this preliminary stage of the design process. All designers shall review the hazards and risks associated with the relevant design element and satisfy themselves that all hazards have been eliminated or mitigate any remaining risks as far as reasonably practicable. The Designer shall also take all reasonable steps to provide the design sufficient information about aspects of the design of the structure or its construction or maintenance as will adequately assist clients, other designers, and contractors to comply with their duties under the Regulations.

Table 8-1: Geotechnical Risk Register (GRR)

Item no.	Hazard	Risk description	Recommended mitigation
1.	Variable ground conditions.	Variable stratigraphic profiling resulting in adverse or unforeseen ground models and unsatisfactory geotechnical design.	This GIR presents a review of the factual GI completed to date. This GIR proposes stratigraphic models within the confines of the proposed wind farm site. The stratigraphic models were developed based on the intrusive exploratory hole information. Element designers shall satisfy themselves that the ground models presented in this GIR are representative of the relevant location within the site. The designers may also develop alternative ground models that are representative of the relevant location within the site while paying due consideration to the limitations of the available ground investigation information.
2.	Incorrect estimation of characteristic soil strength parameters.	Geotechnical failure of structures due to insufficient bearing resistance, sliding resistance, loss of stability or lateral passive resistance.	This GIR proposes characteristic total and effective strength parameters for the soil and rock materials encountered within the confines of the proposed flood defence scheme. The element designers shall satisfy themselves that the parameters presented in this GIR are representative of the stress state of the soil at the relevant limit state. The Designer may also choose different characteristic values that are representative of the stress state of the soil at the relevant limit state while paying due consideration to the limitations of the available ground investigation information. All geotechnical design shall be carried out in accordance with the relevant design code at the time of design. In general, the design principles of I.S. EN 1997-1:2005+A1:2013 (Eurocode 7) shall be followed. Partial factors shall be applied to the characteristic soil parameters, actions, and resistances during Ultimate Limit State checks to produce design values of the applied actions and resistances. The design values shall mitigate the risk of geotechnical failure.
3.	Incorrect estimation of characteristic soil	Excessive vertical settlement structures	This GIR proposes characteristic stiffness parameters for the soil and rock materials encountered within the Site. The element designers shall satisfy themselves that the parameters presented in this GIR are representative of the stress state of the soil at the relevant

Item no.	Hazard	Risk description	Recommended mitigation
	stiffness parameters.	resulting in serviceability failure.	relevant limit state. The Designer may also choose different characteristic values that are representative of the stress state of the soil at the relevant limit state while paying due consideration to the limitations of the available ground investigation information. All geotechnical design shall be carried out in accordance with the relevant design code at the time of design. In general, the design principles of I.S. EN 1997-1:2005+A1:2013 (Eurocode 7) shall be followed. The design shall include Serviceability Limit State checks to estimate the ground movements of the relevant structure.
4.	Underestimation of peat depths	Potentially leading to subsequent failure of structures.	Geotechnical investigations (GI) have been conducted across the whole site area. The design team is responsible for developing the testing criteria to address and mitigate the risk of encountering greater peat depths at these locations.
5.	Low-strength soil (Peat or Shallow Cohesive Deposits)	Failure of low-strength soil during excavation resulting in inundation and/or damage to property or individuals	Peat and low-strength Shallow Cohesive Deposits were encountered within the footprint of the proposed development. The measured strength of these materials suggests they may be at risk of instability due to site operations. Due to the limited GI completed within the Site to date, it is recommended to extend the research to characterise the local geotechnical parameters and delineate the extent of the low-strength soils.
6.	Raised groundwater level	Reduction in soil strength and stiffness resulting in inadequate geotechnical design resistances.	Continuous groundwater monitoring was not completed within the rotary core holes. As a result, the variation of the groundwater level across diurnal, lunar and seasonal periods was not established. However, the groundwater strikes and seepage noted in the intrusive exploratory holes suggest that the groundwater levels are as shallow as the existing ground level, which would be typical of a peat-covered area. It is therefore recommended that a conservative groundwater level is taken for design to mitigate against possible increases in porewater pressures or reductions in design resistances. As a minimum, the design groundwater levels should coincide with the upper-bound groundwater profile recorded in the vicinity of the proposed design element.

Item no.	Hazard	Risk description	Recommended mitigation
7.	Presence of gravel and oversized particles	Groundwater flow due high permeability of gravels and oversized particles encountered across the Site (Silt layer)	Gravels and oversized were encountered across the Site. These soils may be at risk of instability during excavation works. The presence of granular layers (due to their high permeability) could pose issues where temporary excavations are proposed without side supports. Where excavations are required for temporary or permanent works, the relevant Designer shall assess the risk and design suitable mitigation measures were deemed appropriate.
8.	Peat	Failure of peat slopes resulting in peat slides.	Peat or highly organic alluvium has been identified across the majority of the Site. The Peat Stability Risk Assessment is outside the scope of this GIR. The peat depths are typically shallow (less than 1.5m thick), however, the strength was noted to be variable across the site based on hand shear vane results. It is recommended to extend the research to characterise the local geotechnical parameters (particularly undrained shear strength and moisture content) and delineate the extent of the peat to allow a more comprehensive Peat Stability Risk Assessment. During construction, the adjacent land should be visually monitored with any deformation reported to and assessed by a suitably qualified geotechnical engineer.
9.	Karst features	Cavities or voids leading to insufficient pile resistances and geotechnical failure of the proposed Blueway platform.	The underlying bedrock lithology is understood to be Sandstone based on the intrusive GI holes completed by IDL, with possible beds of siltstone and mudstone at greater depths based on the GSI's (2024) bedrock geology mapping. Karst features have not been identified within the study area of the project based on the available online geological mapping data (GSI, 2024). As limestone bedrock does not occur within the site boundary, karst features are not considered to be a risk.
10.	Steel and concrete elements in aggressive ground.	Reduction in material strength resulting in structural failure of the platform.	The site has been identified to be underlain by organic materials which are low in pH and typically high in Sulphates, which are chemically aggressive towards both steel and concrete. The Structural Concrete Designer shall design and specify a suitable concrete mix in accordance with I.S. EN 206:2013+A2:2021 and/or the BRE (2005) Special Digest 1. The

Item no.	Hazard	Risk description	Recommended mitigation
			Structural Steel Designer shall include satisfactory provisions for protection against corrosion and/or apply a loss of thickness due to corrosion in accordance with I.S. EN 1993-5:2007.
11.	Construction plant working over low-strength soils.	Planting becoming stuck or engulfed in low-strength soils.	Peat is present on the site. All temporary access is recommended to be carried out using wide-tracked plant, floating platforms or bog mats. These should be designed by a suitably qualified Temporary Works Designer.
12.	Existing services.	Striking of existing services resulting in damage to existing infrastructure, disruption to local residents and businesses, and/or causing delays to construction.	This is an inherent risk particularly associated with excavation works and cannot be eliminated in full. However, the site is located in a rural high-elevation area with little to no apparent development outside of forestry tracks and is therefore deemed unlikely to contain a significant quantity of services. It is recommended to obtain updated service drawings during the detailed design stage. The risk shall be managed at the construction stage by a competent contractor who shall review the full suite of service maps and survey all locations prior to the commencement of site operations to identify any conflicts with flood defence alignment(s). Particular vigilance should be maintained in relation to uncharted services. Where conflicts are noted, the contractor should take the appropriate action (e.g. diversions) to mitigate against damaging services and disrupting neighbouring stakeholders.

9 CONCLUSION

GDG has completed the GIR, as requested by JC Mont-Fort, assessing the ground conditions for the proposed Illaunbaun Wind Turbine based on the recent ground investigations. A desk study has been conducted assessing all relevant geological, hydrogeological and hydrological data of the Site, and the surrounding environs. A detailed outline of ground investigation works undertaken by Irish Drilling Ltd in September 2024 is outlined in this GIR and this data has informed site-specific characterisations of ground conditions for the preliminary design of the wind farm infrastructure.

Geotechnical soil parameters have been proposed for the soil materials encountered beneath the Site including:

- The bulk unit weight of the soil and rock materials;
- The undrained shear strength of the cohesive soil materials;
- The effective friction angle of the soil and rock materials; and
- The drained and undrained Young's moduli of the soil materials.

The proposed characteristic soil parameters are presented in Table 6-2. The majority of the characteristic parameters are typically based on low estimates, with the characteristic unit weights based on the best estimates. Variations from this table may be required for other limit states, temporary works designs and constructability-related assessments (e.g., pile driving). These tables may be subject to change in later revisions of the GIR should further information become available and justify such alterations. GDG has also identified several geotechnical risks and provided recommendations for mitigation measures in a geotechnical risk register.

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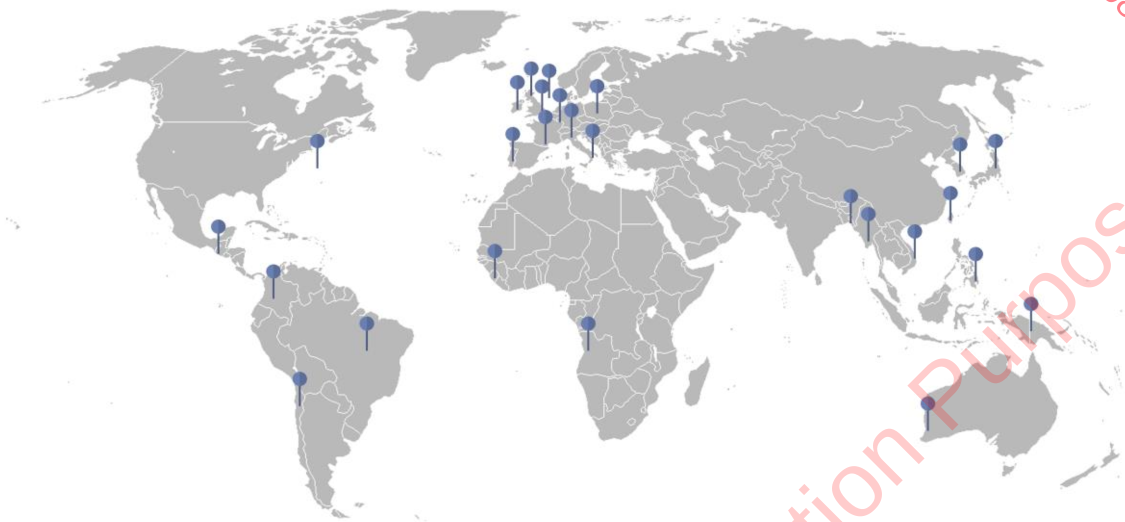

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