APPENDIX 7.1 – GROUND INVESTIGATION REPORT AND GEOTECHNICAL INTERPRETIVE REPORT (2024)

IGSL Ltd

Clonburris Phase 3

Ground Investigation & Geotechnical Interpretative Report

Project No. 25279A

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FOREWORD

The following conditions and notes on the geotechnical site investigation procedures should be read in conjunction with this report.

Standards

The ground investigation works for this project (**Clonburris Phase 3**) have been carried out by IGSL in accordance with Eurocode 7 - Part 2: Ground Investigation & Testing (EN 1997-2:2007). This has been used together with complementary documents such as Engineers Ireland Specification for Ground Investigation (2nd Ed, 2016), BS 5930 (2015+A1:2020) and BS 1377 (Parts 1 to 9) and the following European Norms:

- EN 1997-2 Eurocode 7: 2007 – Geotechnical Design – Part 2: Ground Investigation & Testing
- EN ISO 22475-1:2006 Geotechnical Investigation and Sampling – Sampling Methods & Groundwater Measurements
- EN ISO 14688-1:2017 Geotechnical Investigation and Testing – Identification and Classification of Soil, Part 1: Identification and Description
- EN ISO 14688-2:2017 Geotechnical Investigation and Testing – Identification and Classification of Soil, Part 2: Principles for a classification
- EN ISO 14689-1:2017 Geotechnical Investigation and Testing – Identification, description & classification of rock

The Eurocode 7, Part 2 – Ground Investigation and Testing GI specification shall be read in conjunction with the Specification and Related Documents for Ground Investigation in Ireland, 2nd Edition, published by Engineers Ireland in 2016.

Reporting

No responsibility can be held by IGSL Ltd for ground conditions between exploratory hole locations. The engineering logs provide ground profiles and configuration of strata relevant to the investigation depths achieved and caution should be taken when extrapolating between exploratory points. No liability is accepted for ground conditions extraneous to the investigation points. Unless specifically stated, no account has been taken of possible subsidence due to mineral extraction, mining works or karstification below or close to the site.

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Boring Procedures

Where required, 'shell and auger' or cable percussive boring technique is employed as defined by Section 6.3 of IS EN ISO 22475-1:2006. The boring operations, sampling and in-situ testing meet with the recommendations set out in IS EN 1997-2:2007 and BS 1377:1990 and EN ISO 22476-3:2005. The shell and auger boring technique allows for continuous sampling in clay and silt above the water table and sand and gravel below the water table (Table 2 of IS EN ISO 22475-1:2006).

It is highlighted that some disturbance and variation is unavoidable in particular ground (e.g. blowing sands, gravel / cobble dominant glacial deposits etc). Attention is drawn to this condition, whenever it is suspected. Where cobbles and boulders are recorded, no conclusion should be drawn concerning the size, presence, lithological nature, or numbers per unit volume of ground.

In-Situ Testing

Where required, Standard Penetration Tests (SPT's) are conducted strictly in accordance with Section 4.6 of IS EN 1997-2:2007. The SPT equipment (hammer energy test) has been calibrated in accordance with EN ISO 22476-3:2005 and the Energy Ratio (E_r). A calibration certificate is available upon request. The E_r is defined as the ratio of the actual energy E_{meas} (measured energy during calibration) delivered to the drive weight assembly into the drive rod below the anvil, to the theoretical energy (E_{theor}) as calculated from the drive weight assembly. The measured number of blows (N) reported on the engineering logs are uncorrected. In sands, the energy losses due to rod length and the effect of the overburden pressure should be taken into account (see IS EN ISO 22476-3:2005).

Soil Sampling

Three categories of sampling methods are outlined in EN ISO 22475-1:2006. The categories are referenced A, B and C for any given ground conditions and are shown in Tables 1 and 2 of EN ISO 22475-1:2006. Reference should be made to EN 1997-2:2002 for guidelines on sample class and quality for strength and compressibility testing. Samples of quality classes 1 or 2 can only be obtained by using Category A sampling methods.

Class 1 thin wall undisturbed tube samples (UT100) were obtained in fine grained soils and strictly meet the requirements of EN 1997-2:2002 and EN ISO 22475-1:2006. Soil samples for laboratory tests are divided into five classes with respect to the soil properties that are assumed to remain unchanged during sampling, handling transport and storage. The minimum sample quality required for testing purposes to Eurocode 7 compatibility (EN 1997-2:2002) is shown in Table A.

Table A – Details of Sample Quality Requirements

EN 1997 Clause	Test	Minimum Sample Quality Class
5.5.3	Water Content	3
5.5.4	Bulk Density	2
5.5.5	Particle Density	N/S
5.5.6	Particle Size Analysis	N/S
5.5.7	Consistency Limits	4
5.5.8	Density Index	N/S
5.5.9	Soil Dispersivity	N/S
5.5.10	Frost Susceptibility	N/S
5.6.2	Organic Content	4
5.6.3	Carbonate Content	3
5.6.4	Sulphate Content	3
5.6.5	pH	3
5.6.6	Chloride Content	3
5.7	Strength Index	1
5.8	Strength Tests	1
5.9	Compressibility Tests	1
5.10	Compaction Tests	N/S
5.11	Permeability	2

N/S – not stated. Presume a representative sample of appropriate size.

Samples recovered from trial pits or trenches meet the requirements of IS EN ISO 22475-1. It is highlighted that unforeseen circumstances such as variations in geological strata may lead to lower quality sample classes being obtained.

Groundwater
The depth of entry of any influx of groundwater is recorded during the course of boring operations. However, the normal rate of boring does not usually permit the recording of an equilibrium level for any one water strike. Where possible, drilling is suspended for a period of twenty minutes to monitor the subsequent rise in water level. Groundwater conditions observed in the borings or pits are those appertaining to the period of investigation. It should be noted however, that groundwater levels are subject to diurnal, seasonal and climatic variations and can also be affected by drainage conditions, tidal variations etc.

Engineering Logging
Soil and rock identification has been based on the examination of the samples recovered and conforms with IS EN ISO 14688-1:2017 and IS EN ISO 14688-2:2017. Rock weathering classification conforms to IS EN ISO 14689-1:2017 along with discontinuities (bedding planes, joints, cleavages, faults etc) as classified in Section 6.4 of IS EN ISO 14689-1:2017 and Annex C of same. Rock mechanical indices (TCR, SCR, RQD) are defined in accordance with IS EN ISO 22475-1:2006.

Where peat has been encountered, samples have been logged in accordance with the Von Post Classification (ref. Von Post, L. 1992. Sveriges Gologiska Undersoknings torvinventering och nogra av dess hittils vunna resultat (SGU peat inventory and some preliminary results) Svenska Mosskulturforeningens Tidskrift, Jonkoping, Swedden, 36, 1-37 and Hobbs N. B. Mire morphology and the properties of some British and foreign peats. QJEG, Vol. 19, 1986.

Retention of Samples
After satisfactory completion of all the scheduled laboratory tests on any sample, the remaining material will be discarded. Unless a period of retention of samples is agreed, it is our normal practice to discard all soil samples one month after submission of our final report.

1. INTRODUCTION

IGSL has undertaken a programme of site investigation works in the area of the proposed Clonburris Strategic Development Zone, specifically in the area near the Balgaddy 38kV substation, west of the R136 Grange Castle Road (also referred to as the Outer Ring Road) and north of the Kildare Rail Link (Figure 1). The lands at Clonburris Phase 3, measuring approximately 34 acres, are currently characterised by transitional agricultural landscapes and border mature housing developments to the west and north. The exploratory hole records and an interpretation of the complete 'Phase 3' tranche of site investigation works feature in this report.

Figure 1 – Site Location Plan – intrusive locations plotted

Retrieved from the Ordnance Survey of Ireland

The investigation comprised rotary core drilling, trial pitting and slit trenching. In situ plate bearing tests and soakaway tests (to BRE365) were also performed on site. The investigations were executed in accordance with BS 5930, Code of Practice for Site Investigations (BS 5930:2015 +A1:2020) and EN 1997-2 Eurocode 7 Part 2 Ground Investigation & Testing and supervised by an IGSL geotechnical engineer.

Geotechnical, chemical and environmental laboratory testing was scheduled on a range of soil and rock samples. The geotechnical soil testing included moisture contents, Atterberg Limits and particle size distribution [PSD] testing. Soil reusability testing included Moisture Condition Value (MCV) tests, CBR and compaction testing. Suites of both chemical testing and environmental testing were undertaken on soils. A *"Pyrite Chemical Suite"* was scheduled on near rockhead samples taken from the base of trial pits TP06, TP07, TP16 and TP21. Rock strength testing on recovered cores comprised point load strength index testing [PLSI].

This report presents an interpretation of the data and an assessment of the key geotechnical issues. The exploratory hole locations are plotted on the site plan in Appendix 12.

2. FIELDWORKS

2.1 General

The bulk of the geotechnical investigation works were carried out during March / April 2024 with the rotary drilling works following in May 2024. The site works comprised the following:

- Rotary Core Drillholes (6 No.)
- Trial Pits (29 No.)
- Slit Trenching (29 No.[1])
- Plate Bearing Tests (21 No.)
- Soakaway Tests (to BRE365) (7 No.)
- Groundwater Monitoring
- Surveying of Exploratory Hole Locations

[1] Slit trenches ST09A and ST25A were carried out to further explore the presence of buried cables locally

2.2 Rotary Core Drillholes

Rotary core drilling was carried out (holes denoted RC_) at six locations using a Comacchio GEO-405 rig. Symmetrex drilling was utilised within the overlying superficial deposits with coring techniques used in the underlying bedrock when encountered. Drillholes were taken to depths ranging 5.30m to 7.50m bgl. The rotary drilling in bedrock produced 78mm diameter cores. Bedrock was logged as weak to strong, medium to thinly bedded (to thinly laminated where fissile mudstone/shale), grey/dark grey/black, fine-grained, LIMESTONE. The rock was further described as predominantly argillaceous limestone with layers of calci-siltite limestone, local stylolites and with pyrite present. The rock mass was slightly weathered to moderately weathered at fissile mudstone/shale zones.

The cores were placed in 3m capacity timber boxes and logged by an IGSL engineering geologist. This included photography of the cores with a digital camera. Where rock core was recovered, a graphic fracture log is also presented alongside the mechanical indices. This illustrates the fracture state of the rock cores and allows easy identification of highly fractured / non-intact zones and discontinuity spacings. It should be noted that no correction for dip of the joints has been made and that the spacings shown are successive joint / core intersections within the core.

Standard Penetration Tests (SPT's) were performed during open hole drilling and given the nature of the soils, a solid cone was used. It is noted that the SPT N-Values reported are the number of blows for 300mm increment penetration (e.g. RC01 at 1.50m where N=23). These exclude the seating blow values, which represent the initial 150mm depth of penetration. Where partial penetration was achieved during testing, the number of blows is shown for the actual penetration depth achieved (e.g. RC06 at 4.50m where N=25/10mm). In accordance with Eurocode 7, the SPT hammer has been calibrated and the energy ratio (Er) value is incorporated on the engineering logs. It is highlighted that the SPT N-Values reported on the engineering logs are uncorrected for energy ratio.

The core log records are presented in Appendix 1 and this includes engineering geological descriptions, details of the bedding / discontinuities and mechanical indices (TCR, SCR and RQD's) for each core run. Core photographs are also presented in the aforementioned Appendix 1 and these illustrate the structure and fracture state of the bedrock. The SPT energy ratio calibration certificate features in Appendix 1.

2.3 Trial Pits

Trial pitting was performed at twenty-nine locations across the site. The trial pits were excavated, logged and sampled under the direction of an IGSL geotechnical engineer in accordance with BS 5930 (2015+A1:2020). Bulk samples (B) (typically 20 to 30kg) were taken as the pits progressed.

The bulk samples were placed in heavy-duty polyethylene bags. The trial pits were backfilled with the as-dug arisings and reinstated to the satisfaction of IGSL's site geotechnical engineer. The trial pit logs and photos are presented in Appendix 2 and include descriptions of the soils encountered, groundwater conditions and stability of the pit sidewalls.

2.4 Slit Trenching

Slit trenching was undertaken at twenty-nine locations on the site. The trenches were excavated on a mixture of both grassland and gravel-surfaced areas in lands both north and south of the L1058, Adamstown Avenue. The machine-assisted hand-dug trenches were opened to expose the track of existing buried services and were specifically set out to intercept same based on existing utility drawings. In certain areas, no services were recorded in the open trenchwork. Additional pits were undertaken at locations ST09A and ST25A where visible evidence of ground disturbance suggested services were likely at depth.

Detailed records of the pit findings including depth, diameter and type of service (where found) are presented in Appendix 3. The soil profile provided on the slit trench logs describes the majority of the soils across the transverse trench. Trench extremities (X and Y) were surveyed to ITM using GPS techniques. In addition, the locations of individual services exposed in the pits were also captured. Photographs taken during excavation are presented on the logs as well as separately in Appendix 3 of the report.

2.5 Plate Bearing Tests

Plate bearing tests were conducted at twenty-one locations at depths ranging 0.50m to 0.90m below ground level [bgl]. Plate testing was undertaken to evaluate the modulus of sub-grade reaction (Ks) and equivalent CBR value. A 450mm diameter plate was used for the tests with kentledge provided by a mechanical excavator. Two load cycle tests were performed and the load / settlement plots, Ks and equivalent CBR values are presented in Appendix 4.

2.6 Soakaway Test (to BRE 365)

Seven infiltration tests (SA01-SA07) were performed to assess the suitability of the sub-soils for dispersion of storm water through a soakaway system. The infiltration tests were performed in accordance with BRE Digest 365 'Soakaway Design'. To obtain a measure of the infiltration rate of the sub-soils, water was poured into each test pit, with records taken of the fall in water level against time. Following the first soak cycle, the procedure was repeated to ensure saturation of the sub-soils. The infiltration rate is the volume of water dispersed per unit of exposed area per unit of time, and is generally expressed as metres / minute or metres / second. Designs are based on the slowest infiltration rate, which is generally calculated from the final soak cycle. The soakaway design logs are presented in Appendix 5.

2.7 Groundwater Monitoring

Groundwater monitoring was undertaken following installation of standpipes in each of the rotary core drillholes. Groundwater levels were measured using an electric dipmeter. The levels recorded are shown in Appendix 6.

2.8 Surveying of Exploratory Hole Locations

Following completion of the exploratory works, surveying was carried out using GPS techniques. Co-ordinates (x, y) were measured to Irish Transverse Mercator and ground levels (z) established to Malin Head. The co-ordinates and ground levels are shown on the exploratory hole logs with locations shown on the exploratory hole plans in Appendix 12.

3. LABORATORY TESTING

Geotechnical laboratory testing was carried out at IGSL's INAB-accredited laboratory in accordance with the methods set out in BS1377; British Standard Methods of Test for Soils for Civil Engineering Purposes; British Standards Institute:1990. The laboratory applies best practice management systems as per International Standard IS EN ISO/IEC 17025. The geotechnical testing included moisture contents, Atterberg Limits, particle size distribution [PSD], MCV, CBR and dry density / moisture content relationship (compaction) testing. The results from geotechnical testing on selected trial pit soils are presented in Appendix 7.

Chemical analysis incorporating BRE SD1 Suite D was scheduled on recovered soils. The soil chemical results are presented in Chemtest report 24-16171 in Appendix 8. Eighteen soil samples were selected for Waste Acceptance Criteria (WAC) analysis as per the Rilta suite of testing. The results can be used to classify the material with regard to its potential for disposal to landfill. The results are enclosed in the aforementioned Chemtest report in Appendix 8. The same results formed the basis of a waste classification assessment which was undertaken by O'Callaghan Moran & Associates [OCM] in accordance with the Environmental Protection Agency (EPA) Guidelines on the Classification of Waste (2015). This report is presented separately in Appendix 9.

A "*Pyrite Chemical Suite*" to EN1744 'Tests for Chemical properties of Aggregates' was scheduled on four samples acquired from the base of four trial pits carried out on site. The samples were generally described as "Possible highly weathered rockhead recovered as grey brown clayey/silty GRAVEL". The chemical results are presented in Appendix 10.

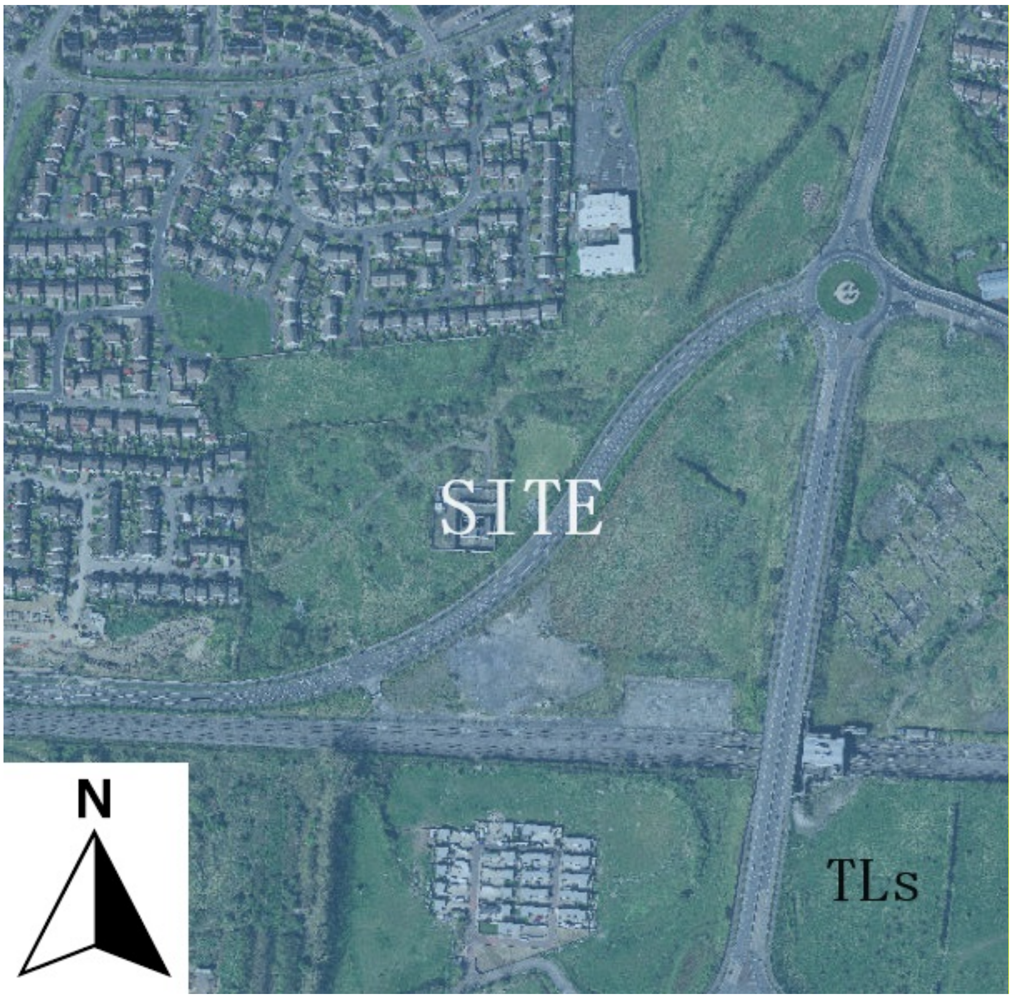
Finally, rock core strength testing comprised Point Load Strength Index [PLSI] testing. The results are presented in Appendix 11.

4. DESK STUDY

4.1 Online GSI Database

The Quaternary Soils plot for the area (Figure 2 - retrieved from GSI website) reaffirms the findings of the investigation and highlights the presence of clay-dominant till (TLs) derived from the ubiquitous Carboniferous Limestone of the area. Shallow outcrop or subcrop is also flagged in the area, to the east of the Outer Ring Road.

Figure 2 - Quaternary Soils Plot for the Clonburris Phase 3 Site



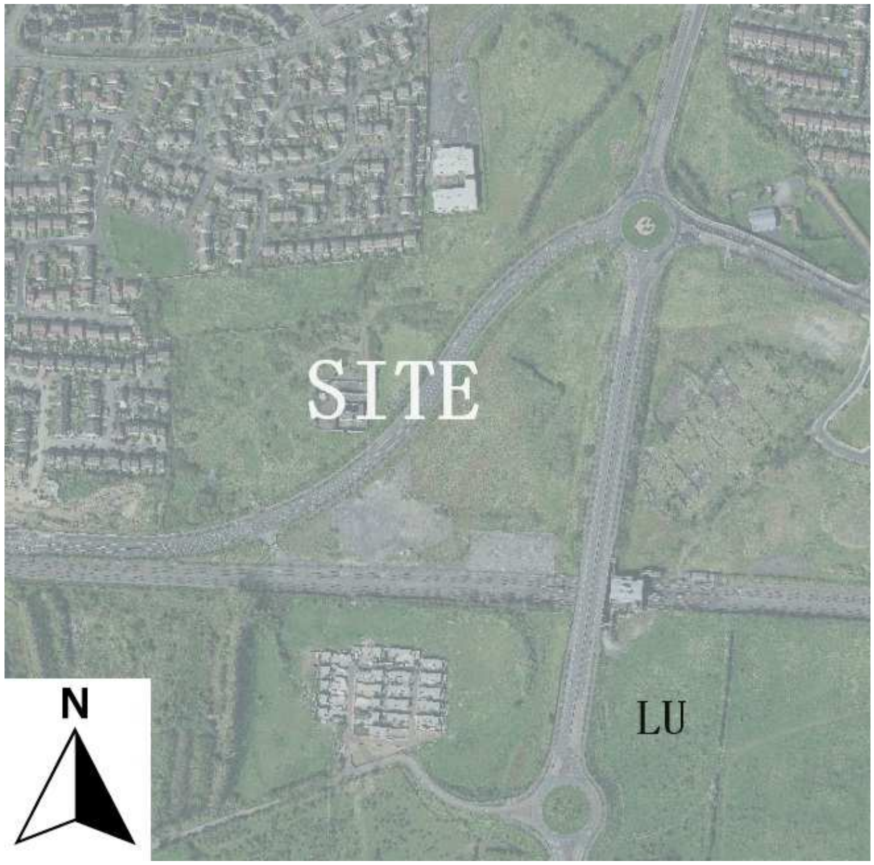
Map Key **TLs** - Till derived from Limestones

Reference to the GSI map for the area (Figure 3, 1:100,000 Solid Geology series) shows that the site is underlain by Lower Carboniferous, Lucan Formation. The Lucan Formation (Nolan 1986, 1989) forms the bulk of the basinal rocks throughout the geologically termed 'Dublin Basin', and is characterised by graded, intraclastic skeletal packstone/grainstone interbedded with anoxic calcareous mudstone / black shale, laminated calcisiltite and argillaceous micrite (i.e. impure limestone with clay minerals).

Its base is defined by the first appearance of thick graded beds of limestone, and a marked decrease in the proportion of interbedded shale, compared with the underlying Tober Colleen Formation. The Lucan Formation is widely known as the Calp Limestone (Marchant and

Sevastopulo, 1980) but is also referred to as the Upper Dark Limestone and has long been a source of building materials and aggregate for Dublin. The Calp is largely undifferentiated geologically.

Figure 3 - Bedrock Geological Map for the Clonburris Phase 3 Site (retrieved from the GSI website)



Key: **LU** = Lucan Formation

4.2 Online OSI Database / Aerial Imagery

Inspection of historic 25" drawings for the area reveals the presence of what appears to be a reservoir in the east of the site nearing the current R136 Outer Ring Road. A '*Pump*' is present in the same area in the 1897-1913 OSI drawing. This prominent feature endures and can be viewed in the 25" Cassini drawing of the 1930's (Figure 4A & 4B).

Aerial orthophotography reveals an interesting development from 1995 (See Figure 5). A trackway crosses east-west across the site which leads to an area of apparent tipping / stockpiling. The area of disturbed ground persists into the colour imagery dating from 1996-2000. The Balgaddy 38kV Substation also appears in the image as does the linear scar of trenchwork resulting from works on a 900mm diameter gas transmission main.

The drawings from 2012 show the Outer Orbital Route under construction. The R136 roundabout is in place but spurs to both Balgaddy / Neillstown to the east and Ballyowen / Lucan to the north are as yet undeveloped. Soil stockpiling is again noted, this time to the west and northwest of the newly constructed R136 roundabout. The current road network, including Adamstown Avenue can be viewed in the more recent 2013-2018 orthophotograph.

Figure 4A, 4B & 4C – Historic OSI drawings with 'Pump' reference for the Kishoge site Fig 4A Ordnance Survey of Ireland 25" drawing dated 1897-1913 depicting '*Pump*'. **Fig 4B** OSI 25" Cassini drawing with similar ponded feature to that of the 1897-1913 drawing. **Fig 4C** OSI 2013-2018 aerial image showing existing landscape with the R136 forming the eastern site boundary.

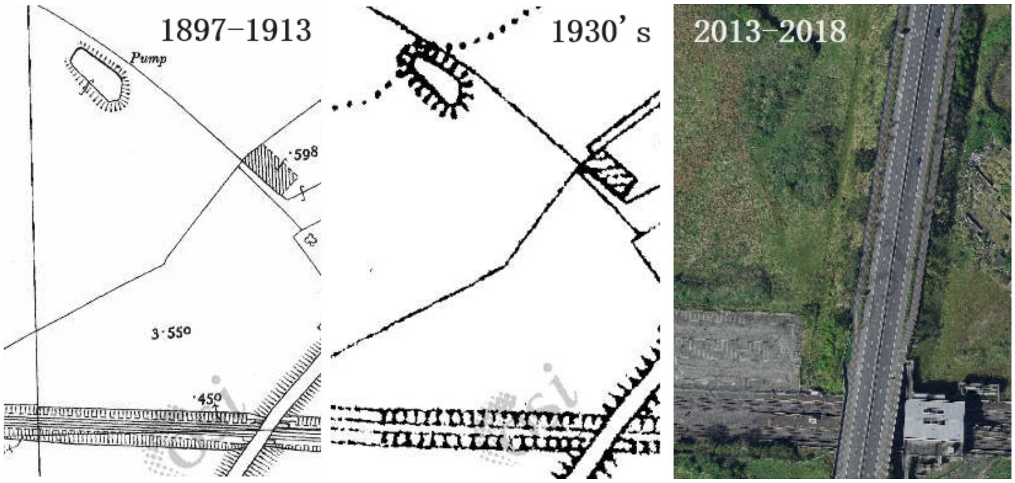


Fig 4A **Fig 4B** **Fig 4C**

Figure 5 –OSI aerial orthophotographs for the Kishoge site 1995 - 2018 (Tailte Éireann)

5. GROUND CONDITIONS & GROUNDWATER

5.1 Ground Profile - Superficial Deposits

The following is a summary of the ground conditions encountered across the Phase 3, Clonburris SDZ site, west of the R136 Outer Ring Road / Grange Castle Road.

MADE GROUND

North of Adamstown Avenue

- Given the recent disturbance documented in the aerial orthophotographs from 1995 (See Figure 5), the existence of Made Ground soils in the stratigraphy is not unexpected.

- Extensive CLAY fill was uncovered in TP02. TP02 lies in an area flagged as having soil disturbance dating from the period of R136 road construction (See Figure 5). Dark brown and dark grey sandy gravelly CLAY soils extended to 1.70m (56.67m OD) where a greyish brown CLAY/SILT was unearthed. This was remarked as containing organic matter. Although classed pitside as Made Ground, this may be a buried topsoil / organic subsoil deposit. This extended to 2.30m (56.07m OD) at which point natural stiff soils were logged.

- In the same field parcel as TP02, nearby TP04 to the south also revealed a thick sequence of Made Ground. Underlying the cover of topsoil, from 0.40m bgl the Made Ground was described as dark grey sandy gravelly silty CLAY with boulders (up to 700mm), cobbles, plastic and steel. A strong organic odour was remarked. The pit ended in Made Ground at 2.50m bgl (57.66m OD).

- At TP07, near the northern boundary of the site close to Oldbridge housing estate, a layer of Made Ground was logged from 0.15m to 1.10m bgl (55.15m OD). It was signaled by the inclusion of rare concrete blocks, rare rubbish / plastic and steel. However, it is thought that these anthropogenic inclusions may be localised in their distribution. A stiff Clay is logged from 1.10m (54.45m OD).

- Rare plastic / rubbish is observed in the topsoil excavated at TP12 to a depth of 0.35m bgl.

- The most extensive collection of rubbish / plastic was logged in TP13. It was measured from ground level to 1.40m bgl. Anthropogenic content was >2% in this area, being rare to occasional. The dig location is linked with the historical soil disturbance noted from 1995 coupled with being proximal to the construction of the adjacent Balgaddy substation. Stiff clay was viewed from 1.40m to the pit base at 2.50m. For the same reasons as TP13, the uppermost soils in nearby TP16 also contained Made Ground to 0.60m. With an absence of rubbish / plastic, the soil disturbance is more likely to relate to construction activity at the time of the substation construction.

South of Adamstown Avenue

- At TP19, to the northeast of the site, a thin cover of topsoil was found to overlie an equally thin layer of clay Made Ground containing rare plastic / rubbish, wood, red brick and concrete fragments. This extended to 0.40m bgl. Stiff indigenous soils were found immediately below this layer.

- Overall, across the southern part of the site, south of Adamstown Avenue, there were variable thicknesses of Made Ground exposed during pitting. In TP24, Made Ground descended to a total of 0.70m bgl (58.93m OD). The Clay soil contained rare plastic / rubbish, pipe fragments and cobbles and boulders. Elsewhere, the deepest accumulation of Made Ground was at TP20 where a 1.40m thick layer of Made Ground was identified. Rare plastic / rubbish was found to 1.0m (58.03m OD). Possible Made Ground extends from 1.40m to 1.90m. However, this is thought to be an indigenous layer of slightly organic subsoil, likely buried decades earlier by the overlying mixed clays.

- At only one pit south of Adamstown Avenue is there an absence of Made Ground. This occurs in TP21 where natural soils extend from ground level to 1.90m bgl (56.28m OD) ending on possible rockhead.

- Excavated into an embankment placed towards the southeastern flank of the site, up to 3.20m (to 58.69m OD) of Made Ground was found in TP28 with 2.60m (to 59.55m OD) in pit TP29.

Figures 6A – 6C – Trial pit sidewall profiles showing Made Ground.
Fig 6A TP04, positioned ca. 2m higher than nearby trial pits, found dark grey slightly sandy slightly gravelly CLAY (MADE GROUND) to 2.50m bgl (57.66m OD). Rare plastic / rubbish and steel combined with a strong organic odour were noted.

Fig 6B At TP13, >2% plastic / rubbish was recorded in the Clay Made Ground to a depth of 1.40m (56.02m OD).

Fig 6C At TP22, Topsoil overlies a dark brown to brown sandy gravelly CLAY with rare plastic and cobble-sized concrete blocks. Possible Made Ground continues to 1.50m with an organic signature. This is thought to be buried topsoil / subsoil.

TOPSOIL

- Where naturally occurring topsoil was unearthed, it was found to be present in layers ranging 200mm to 450mm thick. A gradational lower transition was present whereby the topsoil was underlain by a SILT/CLAY subsoil, almost devoid of gravel.

GLACIAL DEPOSITS

- A fine-grained light brown occasionally mottled orange brown SILT/CLAY subsoil layer, generally firm in consistency, was found underlying the topsoil. Occasionally this was noted as firm to stiff with grey brown mottling also observed.

- Where indigenous deposits were encountered, the soils increased in strength to stiff and were found to contain an increasing gravel-sized clast content with depth. Colour change to grey was observed with depth.

- A stiff dark grey layer completed many of the pits. This was increasingly gravelly, with angular cobble and boulder-sized fragments frequently noted. Towards the base of this layer, the increased volume of angular tabular and platy material caused the layer to be described as a "Possible Weathered Rockhead" horizon. This was noted in six of the twenty-nine pits namely TP05, 06, 07 and 08 as well as TP16 and TP21.

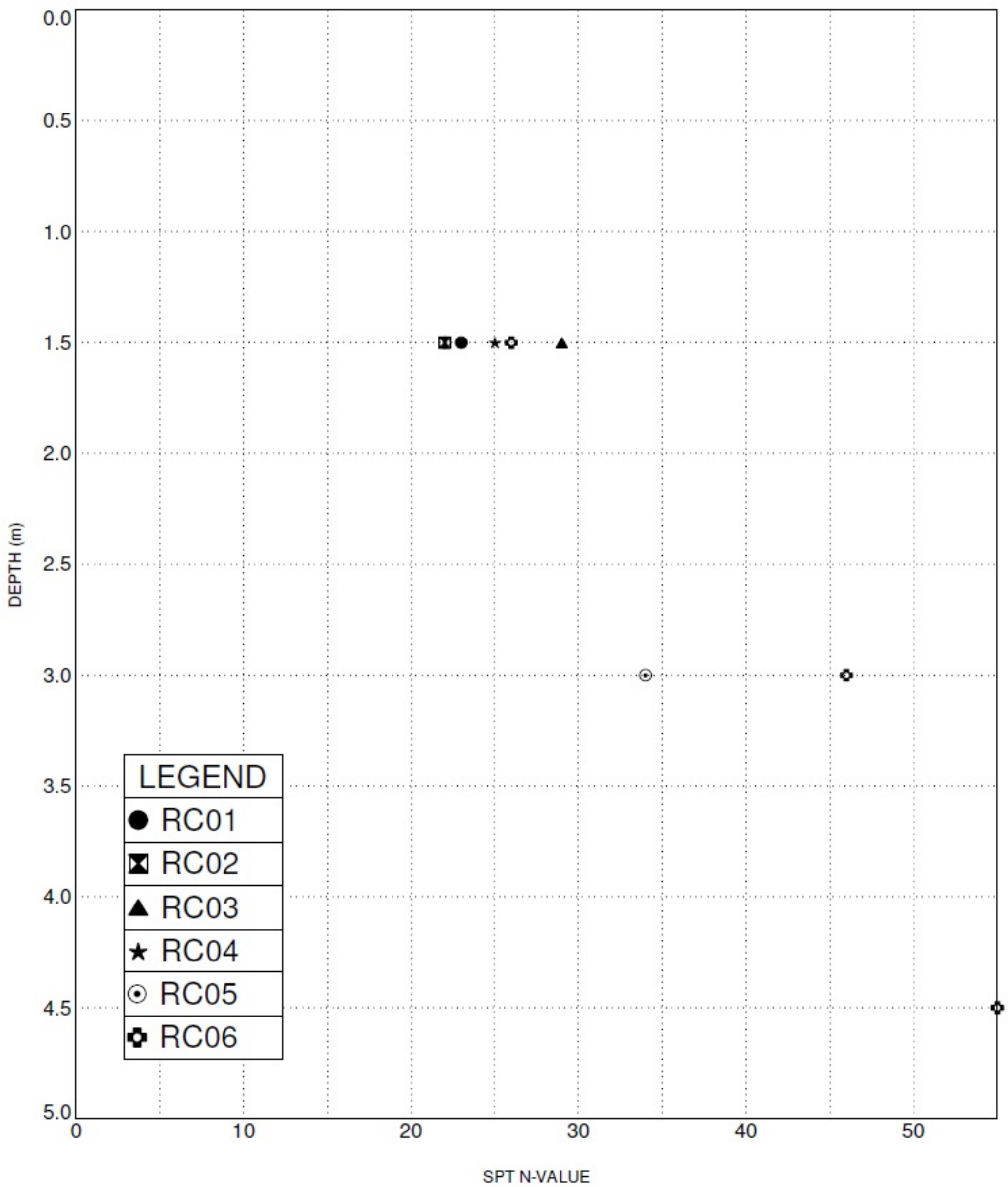
- Rotary drilling revealed bedrock at depths ranging 2.30m to 2.70m north of Adamstown Avenue with rock coring commencing at the deeper depths of 4.30m and 4.50m south of the Avenue. However, in both RC05 and RC06 south of the roadway, a layer of "clayey COBBLES" was intercepted shy of rock. This may well be a layer of weathered rock.

Figures 7A – 7B – Natural ground sidewall profiles photographed during trial pitting. Fig 7A TP05 with topsoil overlying firm to stiff and stiff brown mottled grey & light brown sandy gravelly CLAY with cobbles to 1.30m. Stiff grey blue sandy gravelly silty CLAY to 1.70m. A possible highly weathered rockhead recovered as clayey/silty GRAVEL from 1.70m to the pit base at 2.0m. Slow water entry at 1.70m.

Fig 7B At TP07, topsoil was found overlying a firm brown slightly sandy slightly gravelly SILT/CLAY with rare anthropogenic content. Stiff grey sandy gravelly silty CLAY persisted from 1.10m to 1.80m. From 1.80m to the pit base at 2.50m, possible highly weathered rock was recovered as clayey/silty GRAVEL. Again, as with TP05, a moderate water strike was recorded towards the base of the pit at 2.30m.

In-situ testing was undertaken during the construction of drillholes RC01-RC06. The standard penetration test [SPT] allows for an appraisal of the ground stiffness. A plot showing the blowcounts generated from testing at each hole is presented in Figure 8. It illustrates the occurrence of stiff soils from 1.50m. The depths of rock proven in holes RC01-RC04 allowed only one SPT test to be performed ahead of coring, ie., at 1.50m bgl. Two of the three deeper tests were carried out in RC05 and RC06 in a stratum comprising "clayey COBBLES" at a depth of 3m.

Figure 8 – SPT Plot versus Depth for Rotary Drillholes (uncorrected for energy ratio)



5.2 Bedrock

As referenced earlier in Section 4, the GSI rock map for the area (Figure 3, 1:100,000 Solid Geology series) shows that the Lucan Formation underlies the site. The formation is comprised of argillaceous bioclastic limestones and interbedded shales.

Rotary drilling was conducted at six locations. At each location, drilling penetrated a thin cover of glacial till with core recovery in the underlying bedrock commencing at depths ranging from 2.30m to 4.50m. Figure 9 shows the core recovery in RC02.

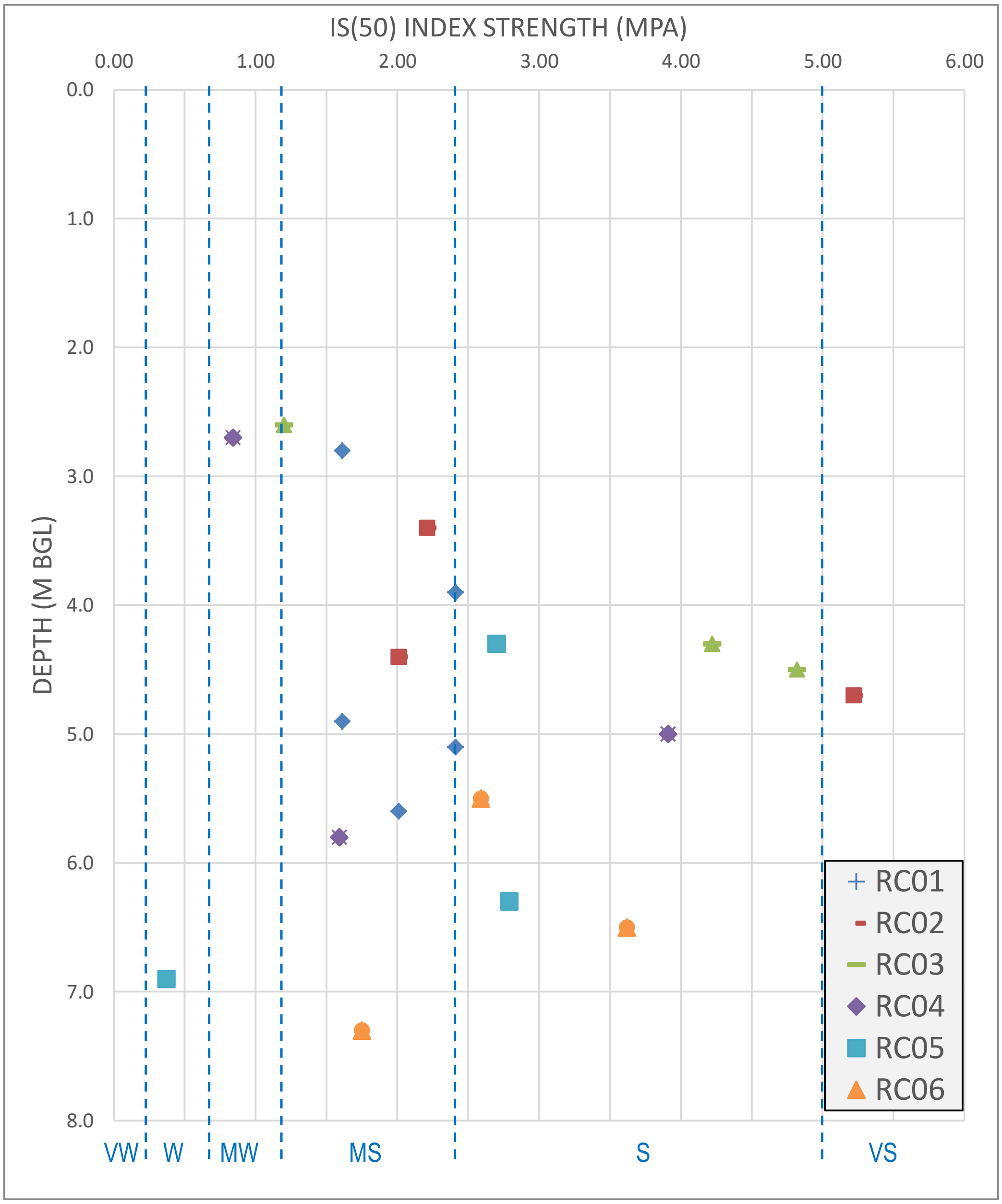
The recovered cores were logged as weak to strong, medium to thinly bedded (to thinly laminated where fissile mudstone/shale), grey/dark grey/black, fine-grained, LIMESTONE. The rock was further described as predominantly argillaceous limestone with layers of calci-siltite limestone, local stylolites and with pyrite present. The rock mass was slightly weathered to moderately weathered at fissile mudstone/shale zones.

Figure 9 - Cores in RC02 from 2.60m bgl to 5.60m

Discontinuity spacings in the rotary cores generally ranged from medium (200 to 600mm) to closely spaced (60 to 200mm), rarely widely (600 to 2000mm) spaced. The discontinuity surfaces were typically smooth to locally rough, planar to locally curviplanar. Apertures were tight to locally partly open with local clay smearing. Discontinuities host calcite veinfill (from 1-20mm thick). Dips are subhorizontal, 10° to 15°, rarely 40° to 45° and very locally 70°.

The point load strength index (PLSI) test data produced $I_s(50)$ values ranging from 0.37 to 5.22 MPa with a mean value of 2.49 MPa. The PLSI strengths plotted in Figure 10 form a broad scatter but are predominantly located to the right of the plot. This implies the cores are generally medium strong to strong. Points to the left of the plot are suggestive of weak rock. They are likely to have resulted from tests undertaken on cores sampled from weaker interbedded shale / mudrock as opposed to the more prominent strong calcisiltite limestone.

Figure 10 – $I_s(50)$ strengths obtained from diametrial Point Load Strength Index testing



VW = Very Weak, W = Weak, MW = Moderately Weak, MS = Medium Strong, S = Strong, VS = Very Strong (ISO 14689:2017 (E))

Using a correlation factor (K) of 20 to assess compressive strength, this suggests a characteristic strength envelope in the order of 7.4 to 104.4 MPa and categorizes the bedrock as weak (5 to 12.5MPa) to lower bound very strong (100 to 250MPa). The visual strength descriptors determined during engineering geological logging marry well with the overall plot scatter in Figure 10.

ISO 14689:2017 (E) rock strength parameters are drawn on Figure 10 to allow correlation between UCS and Point Load Strength tests. A correlation factor (K) of 20 was used to plot the ISO 14689:2017 (E) MPa strength divisions on the Point Load strength ($I_s(50)$) plot.

5.3 Groundwater

Groundwater strikes were intercepted during a number of the excavations on site. Table 1 lists the strike level as well as the intensity of water ingress, the type of stratum in which the ingress was observed, and at what depth (if any) the water equilibrated at upon completion of drilling. The potential exists for seasonal changes in groundwater level. The works were carried out during March, April and May 2024. It is likely that groundwater will be subject to seasonal variations.

Table 1 – Water measurements in on-site exploratory holes

Location	Exploratory Hole No.	Water Struck m bgl	Stratum Description	Rate of Flow	Remarks / Stratum of water ingress (m OD)
North of Adamstown Avenue	TP02	**1.20**	Interface of dark brown and underlying dark grey CLAY (**MADE GROUND**)	Seepage	
		1.70	Interface of dark grey CLAY and underlying organic SILT/CLAY (Probable **MADE GROUND**)	Slow	
	TP05	**1.70**	Possible Highly Weathered Rockhead	Slow	Water entry shy of possible rock
	TP06	**1.70**	Possible Highly Weathered Rockhead	Moderate	Water entry shy of possible rock
	TP07	**2.30**	Possible Highly Weathered Rockhead	Slow	Water entry shy of possible rock
	TP08	**1.0**	CLAY overburden	Seepage	Slow water entry shy of possible rock
		2.0	Possible Highly Weathered Rockhead	Slow	
	TP09	**2.0**	Possible Rockhead	Slow	Water entry shy of possible rock
	TP11	**0.90**	Firm brown mottled grey yellow sandy gravelly silty CLAY	Seepage	Water entry shy of possible rock
		1.90	Possible Rockhead	Slow	
	TP12	**1.50**	Stiff CLAY	Slow	
	TP13	**1.0**	Dark brown sl sandy sl gravelly CLAY with rare to occasional rubbish / plastic **(MADE GROUND)**	Seepage	
		1.60	Stiff CLAY	Seepage	
		2.40	Stiff to very stiff CLAY	Slow	

Location	Exploratory Hole No.	Water Struck m bgl	Stratum Description	Rate of Flow	Remarks / Stratum of water ingress (m OD)
North of Adamstown Avenue	TP15	**1.60**	Stiff dark blue to black cobbly CLAY / Possible Highly Weathered Rockhead	Slow	Water entry shy of possible rock
	TP16	**1.90**	Interface of Stiff CLAY and underlying Possible Highly Weathered Rockhead	Seepage	Water entry shy of possible rock
	TP17	**2.30**	Stiff dark blue to black cobbly CLAY / Possible Highly Weathered Rockhead	Seepage	Water entry shy of possible rock
	TP18	**1.60**	Stiff dark blue to black cobbly SILT / Possible Highly Weathered Rockhead	Seepage	Water entry shy of possible rock
	SA04	**0.60**	Base MADE GROUND / Uppermost firm/stiff CLAY	Seepage	No soakage reported in pit during course of test
	SA05	**1.40**	Base Test Pit in stiff CLAY	Seepage	No soakage reported in pit during course of test
	ST01	**0.80**	CLAY (MADE GROUND)	Seepage	-
	ST03	**2.30**	CLAY (MADE GROUND)	Seepage	-
	ST10	**1.50**	Clayey GRAVEL and cobbles (MADE GROUND)	RAPID	-
	ST12	**1.60**	Possible Rock	Slow	Water entry shy of possible rock
	RC01	-	Water resting in Upper Bedrock	-	Depth to water post drilling **3.10** / 54.72m OD (End depth 5.70m) Dips taken in range 1.94-2.15m bgl
	RC02	-	Water resting in Upper Bedrock	-	Depth to water post drilling **3.70** / 52.79m OD (End depth 5.60m) Dips taken in range 2.17-2.42m bgl
	RC03	-	Water resting in Upper Bedrock	-	Depth to water post drilling **2.90** / 53.59m OD (End depth 5.50m) Dips taken in range 1.93-2.22m bgl
	RC04	**1.90** (54.57)	Lower Superficial deposits / Nearing Rockhead	Slow	Depth to water post drilling **1.40** / 55.07m OD (End depth 5.30m) Dips taken in range 1.27-1.38m bgl

Location	Exploratory Hole No.	Water Struck m bgl	Stratum Description	Rate of Flow	Remarks / Stratum of water ingress (m OD)
South of Adamstown Avenue	TP19	**1.40**	Possible Highly Weathered Rockhead	Slow	Water entry shy of possible rock
	TP20	**2.50**	Stiff to very stiff CLAY	Seepage	
	TP21	**1.90**	Possible Highly Weathered Rockhead	Slow	Water entry shy of possible rock
	TP23	**0.50**	SAND / SILT/CLAY (MADE GROUND)	Seepage	
		1.50	Stiff to very stiff CLAY	Seepage	
	TP26	**1.30**	Possible organic MADE GROUND overlying stiff to very stiff CLAY	Seepage	
	SA06	**1.40**	Clay at base of pit	Moderate	10^{-5} m/s permeability
	ST15	**1.10**	Water along ESB Trenchwork	RAPID	Water flowing along service
	ST17	**1.90**	Water along GNI Trenchwork	RAPID	Water flowing along service
	ST21	**0.40**	SAND & GRAVEL (MADE GROUND)	Moderate	
	ST22	**1.0**	SILT/CLAY (MADE GROUND)	Seepage	
	ST24	**1.30**	CLAY/SILT w/cobbles & boulders (Possible MADE GROUND)	Moderate	
	ST25	**1.0**	Firm / stiff Silty CLAY	Slow	
	RC05	**3.90** (55.42)	COBBLES / Possible Highly Weathered Rockhead	Slow	Depth to water post drilling **4.0** (End depth 7.30m) Dips taken in range 3.34-3.76m bgl
	RC06	**4.70** (55.05)	Upper Bedrock	Slow	Depth to water post drilling **3.20** / 56.55m OD (End depth 7.50m) Dips taken in range 3.19-3.44m bgl

Aside from some deep-seated water strikes in bedrock to the south of Adamstown Avenue - where the rock descends to greater depth - for the most part water strikes were evidenced at the interface of the clayey overburden and underlying bedrock. The proliferation of Made Ground mantling much of the area saw the frequent occurrence of perched water seepages and slow water entries. There was a transitional zone logged at some locations where a fractured angular Gravel and Cobble layer were noted. Water entry was often associated with this 'rockhead' layer / upper weathered rock horizon.

Water entry was observed on a 'Rapid' scale where services were present and where a pathway was formed by the trenchwork. This was very evident in slit trenches ST10, ST15 and ST17.

The water levels recorded by the driller immediately after boring and coring were sited either within the rock or near rockhead.

5.4 Geotechnical Parameters

The ground conditions and associated properties of the superficial deposits and bedrock have been discussed in the previous sections. On foot of the field and laboratory test results, recommended geotechnical parameters are presented in Table 2. It is highlighted that the parameters shown are derived values (not characteristic values) in line with EN1997-1 CL 3.4.3. Characteristic design parameters should be carefully selected by contractors and their designers taking into consideration the ground conditions and engineering properties at particular areas within the Clonburris site.

Table 2 – Recommended Geotechnical Parameters

Parameter	Fine Grained or Cohesive Glacial Till	Coarse Grained or Granular Soils	Bedrock
Bulk Unit Weight (kN/m^3)	22	20	25
Angle of Friction (∅)	34°	Varies 30 to 38°	28° Mudstone /Shale 36° Limestone
Undrained Shear Strength	40 kPa (soft / firm) 80 kPa (firm / stiff till)	NA	UCS Varies 60 to 100 MPa intact strong bedrock
Stiffness (Eu)	30 MPa (soft / firm till) 60 MPa (firm / stiff till)	70 to 100 MPa	10 GPa (intact strong bedrock)

5. GROUND ASSESSMENT & ENGINEERING RECOMMENDATIONS

6.1 General

In light of the investigation findings, the following ground engineering items are discussed:

- Bearing Capacity & Foundations
- Ground Bearing Slab
- Groundwater / Infiltration
- Slopes / Batters
- Pavement Construction
- Buried Concrete
- Earthworks Testing
- Waste Acceptance Criteria [WAC] & Environmental Testing – *Soils destined for Landfill*

6.2 Bearing Capacity & Foundations

Firm to stiff and stiff brown and grey brown glacial till soils were frequently logged towards the base of trial pits. These were exposed either under natural firm colour-mottled subsoil or underlying placed Made Ground soils. The Made Ground soil composition varied spatially throughout the site dependent on the historical use and the placement of soil stockpiles / formation of banked soil mounds (TP04, TP28 & TP29). There was a frequent occurrence of rubbish / plastic in many of the pits to depths of 1.70m / 56.67m OD (TP02) and 1.40m / 56.02m OD (TP13). Elsewhere, at former hardstanding areas, there was found a surficial layer of placed hardcore gravel (TP23-TP26).

In a number of pits, a thin layer of clayey SILT, often remarked as having an organic signature, was found underlying the Made Ground. This is thought to be a buried topsoil or organic subsoil.

During trial pitting, the soils were remarked as increasing in stiffness nearing rockhead. The firm to stiff and stiff over-consolidated glacial deposits should provide an allowable bearing capacity of 150 kN/m^2. Should higher bearing pressures be required, the alternative is to position the foundations on the shallow upper bedrock located at ca. 2.30 – 2.75 bgl (corresponding to 55m OD to 53.50m OD). The depth to rockhead falls to the south of the site where a layer of clayey COBBLES was first encountered from 3.0m (RC05 & RC06). South of Adamstown Avenue, the rock was cored between 4.30m and 4.50m depth bgl (ca. 55m OD). Extending foundations to this depth would likely require excavate and replace with low grade concrete from deep rockhead depths to base of foundation.

Stiff to very stiff dark grey black, bluish black, occasionally dark brown sandy gravelly CLAY was documented towards the base of a number of trial pits. Where this was intercepted, shy of rockhead, allowable bearing capacities in the over-consolidated glacial till would rise to 200kPa. Given this layer's proximity to eventual rockhead, there is a possibility for water ingress which may promote water-softening in the clay.

If foundations are placed on the medium strong and strong argillaceous limestone this should be capable of safely supporting bearing pressures of 1250 to 1500 kN/m^2. However, given the inherent variability in rock mass strength and the potential for some localised variations in weathering grades, the poorer quality calcareous mudstone or shale (weak) must be carefully considered in terms of potential for differential settlement and long-term performance (the mudstone can be locally weathered to a very weak rock or stiff clay). For structural design purposes, it would be prudent to size foundation pads using a safe or allowable bearing pressure of 750 kN/m^2. The proviso with the above is that horizons or zones of weak mudstone or muddy limestone be removed and replaced with low grade concrete.

Foundations constructed on such variably weathered bedrock require careful examination by a suitably experienced (competent) geotechnical engineer or engineering geologist. Plate load tests (minimum of 600mm diameter), if practical given dig depths, are particularly useful in evaluating performance under loading and deciding on a suitable formation depth. Foundation excavations are anticipated to reveal an irregular or saw-tooth profile with beds of strong limestone adjacent to very weak mudstone. This is not unusual in this area of north-west Dublin, hence the input and advice of a geotechnical engineer / engineering geologist during the foundation construction works.

6.3 Ground Bearing Slab

In order to support conventionally loaded ground bearing slabs, it is recommended that a firm (medium strength) formation is reached. It will therefore be necessary to remove any soft / low strength upper soils / Made Ground before placement of the hardcore layers. With reference to the pit findings, it is anticipated that stripping to depths of between 0.6 and ca. 1.40m bgl should be sufficient in most instances to reach firm soils. This is likely to fluctuate depending on localised thicknesses of Made Ground.

It is recommended that T0 Struc hardcore be used in conjunction with T1 hardcore and these should meet the requirements of Annex E SR21:2014+A1:2016. Proof rolling the formation (static rolling with roller having a mass per metre width of roll of not less 5400 kg) is advised to counteract disturbance or loosening due to the bulk excavation works. Under no circumstance should vibratory or dynamic rolling be used on the formation soils as this may lead to dilation where silt-dominant soils are present, producing characteristic ‘cow-bellying’.

Imported granular fill ‘hardcore’ used in any foundation application or under concrete floor slabs should meet the requirements of Annex E of SR 21:2014+A1:2016. Both T0 and T1 hardcore fills should be rigorously tested (independent of the quarry source) to ensure that they meet the physical, durability, chemical and mineralogical characteristics as set out in the aforementioned Annex E of SR 21:2014+A1:2016. Independent testing on samples of the proposed source hardcore is strongly recommended in advance of the material being used on the site. As a minimum, particle size gradings, chemical tests (total sulphur and acid soluble sulphate) and simplified petrology are advised to screen the material and independently assess compliance with Annex E, SR21;2014+A1;2016.

Should the existing hardcore found on site (TP23-TP27) be assessed as being mudrock-containing (potentially pyritiferous mudrock), it would be recommended to remove the stone and stockpile separately for either disposal from site or for use under flexible pavements / berms. It would be important that the stone not be left lying in areas where is would ultimately be overlain by concrete floors or concrete footpaths.

6.4 Groundwater / Infiltration

As noted in Section 5.3 and listed in Table 1, shallow groundwater strikes between 1.0m and 2.0m depth were frequently observed in open excavations as seepages or as ‘slow’ ingress. Water intercepted in Made Ground was regarded as isolated, or localised perched seepages rather than representing actual water bodies. Intense water strikes were observed in slit trenches along existing service corridors. The ‘rapid’ water entry in three of the twenty-nine slit trenches serves to highlight the impermeability of the surrounding natural stratigraphy relative to the disturbed soils / permeable pipe surround. Shallow pits were not left open for a long duration to allow for natural water ingress / groundwater re-equilibration.

Water entry was observed in the drillholes during their construction, from 1.90m (54.57m OD) to 4.70m bgl (55.05m OD), generally associated with rockhead. Where strikes were not recorded, the levels of water dipped in the drillholes upon completion often mirrored that of the upper rockhead level. It should be noted that water levels measured in drillholes immediately upon completion can often be artificially heightened given the introduction of an air/mist flush during coring.

However, water monitoring in June and August 2024 at each of the six drillholes revealed the water levels settled just above the core commencement depths / rockhead.

Overall, based on the ground investigation findings, groundwater is likely to be found in or just above the uppermost bedrock. Groundwater flows here will be governed by fracture state and flows or ingress would be expected to occur along the more open joints or discontinuities. There is a strong likelihood that prominent or copious inflows will be uncovered at localised zones within the upper bedrock / weathered rockhead horizon. This is a well-known feature of the Calp Limestone bedrock in Dublin, where groundwater inflows occur or tend to concentrate along the more fractured beds or weathered zones.

Provision should be made for de-watering during excavation works and groundworks, especially where trenches or open cut areas are required below the glacial soil / bedrock interface. A combination of perimeter drains (open drains) connected to strategic sumps is expected to be used to control groundwater. As mentioned in Section 5.3, the potential does exist for there to be seasonal changes in groundwater level. The works were carried out during spring / early summer 2024. It may be the case that the various waterbodies at depth are subject to seasonal variations.

Soakaway tests were conducted on the site at seven locations. The tests were carried out in what were deemed to be both Made Ground CLAY soils (TP01, 04 & 07) and the natural firm and stiff overburden soils. The impermeable nature of the soils may account for the low infiltration rates obtained.

It is likely that such CLAY soils would not be suitable for conventional soakaways being classified as offering only low natural infiltration (Table 3).

Table 3 – Measured infiltration rates (f) expressed as exposed area (metre) per unit time (minute)

Soakaway Test No.	Depth of Test (m bgl)	*f* (m/min)	*f* (m/sec)
SA01 (Cycle 1)	1.20	0.00461 m/min	7.68E -05 m/sec
SA01 (Cycle 2)	1.20	0.00074 m/min	1.24E -05 m/sec
SA02	1.50	0.00025 m/min	4.21E -06 m/sec
SA03	1.50	0.000077 m/min	1.28E -06 m/sec
SA04	1.50	0 m/min	0 m/sec
SA05	1.40	0 m/min	0 m/sec
SA06	1.40	0.00125 m/min	2.079E -05 m/sec
SA07	1.80	0.00112 m/min	1.866E -05 m/sec

6.5 Slopes / Batters

A maximum slope angle of 1V to 1.5H (33º) should be possible for temporary batters constructed within the upper medium strength indigneous fine grained soils. A slope angle of 1V to 2H (26°) should be appropriate for long term batters in the same soils. Where deep excavation works are required in the superficial deposits, the use of trench box support is advised. In addition, the uppermost fine subsoils will be susceptible to softening and degradation and surface water or groundwater ingress can lead to a significant reduction in shear strength. Perched water can exist locally and this should be considered in risk assessments for excavations. This is especially true in layers of Made Ground. By the nature of their unconsolidated, unengineered placement, anthropogenic soils such as those observed on site are expected to be highly unstable.

If anticipated, excavations into uppermost rock should be assessed by a suitably qualified engineering geologist. The angle which freestanding faces in limestone bedrock can be cut to will be influenced by, among other factors, bed thickness and angle of bedding, discontinuty spacing, clay infill and groundwater entry / seepage. Man-entry into any deep excavation should be appropriately assessed and an AF3 form completed. The AF3 form details the thorough examination of an open excavation as well as documenting daily worksite inspections.

Site operatives or personnel should not enter unsupported excavations and should be informed of potential risks. Where site operatives or engineering staff work in close proximity to temporary slopes or batters, these should be inspected and approved by a suitably experienced civil engineer, preferably with geotechnical experience. Where there is a risk of spalling of battered slopes, the use of a geogrid is recommended. The geogrid should be anchored at the top and bottom of the ridge face to contain particles such as gravel, cobbles and / or boulders that may become dislodged.

6.6 Pavement Construction

Twenty-one plate load tests were conducted at depths ranging 0.50m bgl to 0.90m bgl. The plate load test permits an assessment of the in-situ stiffness of the upper soil. The test results are reported in Appendix 4 and summarised in Table 4. The range of equivalent CBR values measured was 0.40% and 13.7% on the initial loading cycle (Cycle 1) and 0.6% - 18.8% on the reload cycle (Cycle 2). The reload cycle demonstrates modest improvement in performance of the subgrade following initial loading. It is likely that following excavation of the formation, that use of a smooth drum roller ahead of hardcore placement will deliver a similar improvement in subgrade performance.

Table 4 – Equivalent CBR % Values obtained in Plate Bearing Testing

Test No. (%)	Depth	CBR at Load Cycle (%)	CBR at Re-Load Cycle
CBR 01	0.50	2.6	18.8
CBR 02	0.50	2.3	4.7
CBR 03	0.50	2.4	3.4
CBR 04	0.50	3.0	4.3
CBR 05	0.50	1.6	4.2
CBR 06	0.50	0.4	0.6
CBR 07	0.70	2.3	4.9
CBR 08	0.50	1.5	6.3
CBR 09	0.50	1.3	1.5
CBR 10	0.90	2.7	4.6
CBR 11	0.50	13.7	18.7
CBR 12	0.50	2.3	5.0
CBR 13	0.50	1.9	4.4
CBR 14	0.50	2.1	4.2
CBR 15	0.50	4.9	6.5
CBR 16	0.50	2.6	5.8
CBR 17	0.60	1.3	2.2
CBR 18	0.50	3.3	10.0
CBR 19	0.50	3.6	9.7
CBR 20	0.50	1.7	12.5
CBR 21	0.50	8.5	11.9

In accordance with the Design Guidance for Road Pavement (HD 25-26/10:2010), the lower-end equilibrium CBR values should be used to determine capping layer thickness. Disregarding some plate test results where subgrade disturbance or the presence of Made Ground may have derived uncharacteristically lower results, a CBR design value of 2.5% should be adopted for these buried firm clay soils.

In the case of the test undertaken at CBR06, a CBR design value of <2% should be applied to the near surface soils (0.50m bgl) in their current state. It is possible that Made Ground exists at this location and so extraction and removal of same is recommended prior to road construction.

Ahead of road construction, and following compaction of the soils, a further set of plate testing (450 or 600mm diameter) should be undertaken to assess the improvement in stiffness of the formation. An improvement should see a reduction in the build-up of capping stone required. Alternatively, slightly deeper excavation may be necessary to locate a more resilient subgrade.

Assuming a design CBR value of 2.5% for the upper soils then a minimum 6F capping thickness of 400mm and a sub-base thickness (UGM) of 150mm is recommended to support the road pavements / car park. If or where very low strength subgrade occurs (CBR <1%) either geogrid reinforcement or the use of starter material (Class 6A / 6B) could be considered to provide a suitable foundation layer especially for access or haul / spine roads if they traverse low strength subgrades. Such a mechanically stabilized layer could consist of a layer of geogrid with 500 to 600mm of granular fill (well graded aggregate with maximum particle size of 75mm). Where geogrid is not utilized then approximately 500mm build-up of Class 6A / 6B starter layer material could be considered in conjunction with a capping layer (Class 6F capping in line with Series 600 of TII SRW). This should provide a satisfactory foundation layer to adequately support the pavement (150mm of unbound granular material (UGM) in accordance with Table 2.1 of CC-SPW-00800, TII August 2022). The aforementioned Class 6A / 6B material could be used in conjunction with ca. 300mm of 6F capping material. This should provide a robust foundation layer.

The time of year will play a role in sub-grade strength especially during winter or early Spring where heavy rainfall would cause degradation / wash-out of the formation. If there are particular concerns regarding the condition of the formation soils, then additional plate bearing tests should be considered during construction to verify or validate the stiffness / density of the formation soils and adequate capping thickness.

The durability of the capping material should be confirmed as capping will be exposed to the elements (especially if the works are undertaken during the winter / spring period). It is important that argillaceous sedimentary rocks (i.e. muddy limestone, calcareous mudstone, shale, etc.) are not used as capping or as a starter layer. These have high potential to give rise to degradation (i.e. poor durability and soundness) and slaking and therefore would not be suitable.

All granular fills / unbound granular mixtures (UGM) used in pavement construction should be tested and approved in advance of being used in pavement construction. They should meet the compositional, chemical and soundness requirements as prescribed in the TII publication entitled *Road Pavements – Unbound and Hydraulically Bound Mixtures* (CC-SPW-00800 – dated August 2022).

Compaction / Placement of imported granular fill or hardcore will need to achieve low air voids (<5%) and ensure that settlement is not an issue. The number of roller passes and mass per metre and width of roll should meet the guidelines in I.S. 888:2016 *Annex B: Compaction requirements for unbound mixtures Table B.1.* It is recommended to use a smooth drum roller (without vibration) with a mass per metre of roll of not less than 5400kg. Unbound mixtures should not be laid in layers greater than 150mm if using this compaction method.

6.7 Buried Concrete

The chemical analysis tests on natural soil samples (BRE SD1 analysis suite) show pH (2.5:1) values ranging from 8.3 to 9.6. The sulphate aqueous extract (SO_4) results from trial pit samples determined values of <10mg/l to 380mg/l. This would suggest the ‘as-received‘ soil samples tested could be categorised as BRE Class DS-1.

Table C1 ACEC for greenfield sites in BRE SD 1 (2005) can be used in the selection and design of concrete. If mobile groundwater conditions prevail at the site and given the pH values obtained from the testing, then ACEC class AC-1[d] would be expected to be appropriate for buried concrete in the soils. In line with I.S. EN 206-1:2013, concrete could be manufactured to Class XA1 where founded or positioned in the upper soils (Class XA1 being ≥ 2000 and ≤ 3000 SO_4^{2-} mg/kg).

In the absence of sulphate analysis conducted on the bedrock, should footings be extended to rock, the guidance given in IS EN 206;2013 (Concrete: Specification, Performance Production and Conformity) states that the most onerous value for a single chemical characteristic determines the concrete class. In terms of concrete manufacture to IS EN 206-1:2002, it would be prudent to have concrete manufactured to Class XA2 if founding in bedrock. This is advised on the knowledge of the argillaceous limestone and calcareous mudstone bedrock present in the Dublin Calp Limestone and potential for oxidation and sulphate attack.

6.8 Earthworks Testing

To evaluate the re-use properties of the upper soils, a programme of earthworks laboratory testing was conducted. This comprised CBR, Moisture Condition Value (MCV) and Dry Density / Moisture Content relationship. Bulk samples were acquired from trial pits excavated across the site with testing conducted on the material at their natural or 'as-received' moisture contents. Earthworks testing was undertaken on those samples listed in Table 5. Their respective depth intervals and soil descriptions are shown in the aforementioned table.

Table 5 – Sample description of soils used in Earthworks testing

Exploratory Hole No.	Sample Depth	Sample Description
TP01	0.80	Brown slightly sandy, slightly gravelly, CLAY
TP03	0.80	Brown sandy, gravelly, CLAY
TP06	0.80	Brown slightly sandy, slightly gravelly, CLAY
TP08	0.80	Brown sandy, gravelly, CLAY
TP10	0.80	Brown slightly sandy, slightly gravelly, CLAY
TP14	0.80	Brown slightly sandy, gravelly, CLAY
TP16	0.80	Brown sandy, gravelly, CLAY
TP17	0.80	Brown slightly sandy, slightly gravelly, CLAY
TP19	0.60	Brown sandy gravelly SILT/CLAY
TP24	1.20	Brown slightly sandy, slightly gravelly, CLAY

The samples, ahead of being subject to reusability testing, each have their >20mm fraction removed. The resultant earthworks testing (on natural 'as-received' samples) produced laboratory CBR results in the range 0.4 to 17.8% with MCV's of 1.2 to 11.8. Moisture contents ranged from 11 - 26%. Maximum dry densities were proven to range between 1.56 and 2.05mg/m^3 at moisture contents of 14% to 17% (refer to Table 6).

The moisture contents in the 'native' CLAY/SILT are elevated (11-26%) when compared to the moistures at which the soils achieve their maximum dry density. Compaction tests revealed optimum moisture contents of 14-17%. This explains the occasionally low CBR % values, as well as the low MCV values, some of which demonstrate the soils to be wet of optimum. Overall, the testing suggests the bulk of the soils, if handled carefully, would be classed as acceptable for re-use as Class 2 materials (2C1 Stony Cohesive material – high fines content) in line with Series 600 TII SRW.

Table 6 - Summary Details of Laboratory Testing samples

Hole No.	Depth	Lab CBR Value % (Moisture Content %)	MCV at Natural Moisture Content (Moisture Content %)	Dry Density / Moisture Content Relationship
TP01	0.80	4.0 (24)	**8.3 (25)**	Max Dry Density = 1.63mg/m^3 at 15.2% OMC
TP03	0.80	0.4 (19.7)	1.2 (20.3)	-
TP06	0.80	4.5 (14.7)	3.7 (18.5)	Max Dry Density = 1.90mg/m^3 at 12.7% OMC
TP08	0.80	3.7 (15.5)	7.3 (16.7)	-
TP10	0.80	13.1 (26.8)	5.9 (29.5)	Max Dry Density = 1.60mg/m^3 at 17.2% OMC
TP14	0.80	17.8 (11.6)	6.0 (14.8)	Max Dry Density = 2.05mg/m^3 at 6.2% OMC
TP16	0.80	12.6 (20.2)	**11.8 (24.4)**	-
TP17	0.80	1.0 (14.5)	**8.7 (14.3)**	Max Dry Density = 1.99mg/m^3 at 9.1% OMC
TP19	0.60	11.1 (22.8)	**10.4 (22.7)**	Max Dry Density = 1.68mg/m^3 at 14.2% OMC
TP24	1.20	11.5 (24.4)	**10.6 (25.1)**	Max Dry Density = 1.56mg/m^3 at 14% OMC

Bold font = optimum and dry of optimum (8-14)

Given the occasionally very low and low MCV results (minimum MCV of 8 normally required for Class 2 soils) some of the soils could be modified and strengthened by the addition of lime / cement binders. Treatment with lime or lime / cement (soil stabilization) would increase MCV (limits of 8 to 12 advised) and CBR (15% recommended by plate load test method).

If the design makes provision for ground improvement or soil stabilization methods, then trial mix laboratory testing and a field demonstration trial (footprint of c. 10 x 10m) are advised. The key objective of a field trial would be to assess the performance of the modified soils with lime or lime / cement binders using earthwork plant. This would allow for in-situ testing (plate load, nuclear gauge, sand replacement and CBR mould samples) to measure CBR / stiffness, relative compaction (percentage degree of compaction) and air voids.

It is vital that if soil stabilisation process is chosen, that any soil stockpiles are graded and shaped so that surface water cannot collect or pond. Similarly, careful control of excavation, transporting, stockpiling, placing and compaction is advised to ensure that degradation of the shallow soil deposits does not occur. This is extremely important as poor earthworks management would render the fine silty soils as unsuitable for re-use.

In summary, according to laboratory testing, without reworking / drying or modification with lime (calcium oxide), the natural, uppermost, surficial fine-grained clay/silt would be suitable for re-use. However, it is possible the addition of binders would be required to produce a consistently acceptable sub-formation layer (high strength Class 2 engineered fill with an MCV 8 to 14).

6.9 Waste Acceptance Criteria [WAC] & Environmental Testing

Soil samples were taken across a range of depths from trial pits. Samples were analysed for their compliance to the criteria set out in the 2002 European Council Decision (2003/33/EC). O'Callaghan Moran & Associates conducted a waste characterisation assessment of the samples in accordance with the Environmental Protection Agency (EPA) Guidelines on the Classification of Waste (2015). This report, together with conclusions and recommendations, is presented in Appendix 9.

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