Iarnród Éireann
Irish Rail

Rosslare ORE Hub

EIAR Technical Appendixes

Technical Appendix 7:

Soils, Geology, Hydrogeology and Contamination

Geotechnical Interpretive Report

venterra | GDG GAVIN & DOHERTY GEOSOLUTIONS

NICHOLAS O'DWYER an RSK company

Co-funded by the European Union

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EXECUTIVE SUMMARY

Gavin & Doherty Geosolutions Limited (GDG) has been appointed by Nicholas O'Dwyer (NOD) to produce a Ground Investigation Report (GIR) to be considered in the design and site development for the newly proposed Offshore Renewable Energy (ORE) Hub at Rosslare Port for the eventual end user, Iarnród Éireann. The ORE Hub will accommodate several functions, such as new quayside infrastructure, pontoons and gangways to accommodate the berthage and mooring of Crew Transfer Vessels (CTV), landside infrastructure including warehousing, office infrastructure, parking and also associated dredging works. This report has been prepared in accordance with IS EN 1997-1:2005 (NSAI, 2005).

This GIR provides land-based and marine-based ground profiles and associated soil and rock parameters within the proposed development area. This assessment is based on the following information and results are summarised in the table below.:

1. Ground investigation data (project site investigations and historical investigations completed in the harbour area; including geophysical surveys),
2. Geotechnical laboratory data, and
3. Published and unpublished geological information.

This report includes ground investigations carried out by Causeway Geotech Ltd. between November 2023 and February 2024, including:

- 30 no. over-water nearshore marine boreholes;
- 12 no. over-water nearshore marine Cone Penetration Tests with pore water measurement (CPTU's) with dissipation tests, completed by sonic drilling in soil/rock)
- 18 no. over-water nearshore marine Cone Penetration Tests with pore water measurement (CPTU's) with dissipation tests
- 4 no. over-water nearshore marine Cone Penetration Tests with pore water measurement (CPTU's) with dissipation tests; completed with drill outs
- 59 no. over-water nearshore marine gravity corers (Vibrocores
- 3 no. land-based boreholes; sonic drilling in overburden extended by rotary core follow-on in soil/rock
- 5 no. grab samples
- Geotechnical and geoenvironmental laboratory testing
- Geophysical survey composed of:
    - Multibeam echosounder (MBES),
    - Marine magnetometer,
    - the sidescan sonar (SSS),
    - Sub-bottom profiler (SBP), and
    - Boomer seismic.

The findings of this ground investigation have been examined in this report, collating and interpreting the results of the site and laboratory data to develop a site wide ground model and summary of recommended engineering parameters to be used in the civil and geotechnical design.

The ground model is outlined in Section 5, and the recommended geotechnical parameters are outlined in Section 6.

The findings outlined in this report provide a broad set of material types and parameters for the entire site, highlighting potential design risks. Designers will need to conduct their own assessments of the local ground conditions relevant to their proposed design and conduct sensitivity analysis of the engineering parameters relevant to their proposed design.

1 INTRODUCTION

1.1 BACKGROUND

GDG was appointed by Iarnród Éireann (Irish Rail) as Lead Consultant in November 2021 to undertake Phases 1 & 2 of their overall seven-stage project to gain planning and statutory licence approvals and construct a new Offshore Renewable Energy Hub in Rosslare Europort. In July 2023, a multi-disciplinary project team comprising Nicholas O'Dwyer (NOD), acting as Lead Consultant, and GDG as their team partners was appointed to progress Phases 3 & 4 of the project. Options for undertaking Phases 5, 6 and 7 of the project were also submitted as Optional Services by the team of NOD and GDG at the time of tender, to carry the project through to completion.

Nicholas O'Dwyer (NOD) appointed Gavin and Doherty Geosolutions Ltd. (GDG) to produce a Ground Investigation Report (GIR) as part of the development of Rosslare Europort Offshore Renewable Energy (ORE) Hub, for the eventual end user, Iarnród Éireann.

This GIR and the related figures provide an interpretative summary of the available ground investigation data, relevant desk study information, in-situ and laboratory soil and rock testing, and other published data for the site. The report also summarises preliminary design parameters for the development of ORE Hub.

1.2 DESCRIPTION OF PROJECT

GDG has undertaken a pre-feasibility assessment of the project with Iarnród Éireann (Irish Rail) in 2019 / 2020 to demonstrate the suitability and commercial viability for the facility to act as a main Staging & Installation facility for East coast and Celtic Sea offshore windfarm construction. GDG was also the Lead Consultant for Phases 1 & 2 of the current project in 2022 and early 2023, which brought forward the overall concept of a new Offshore Renewable Energy (ORE) Hub within Rosslare Europort, to service the burgeoning offshore wind sector in Ireland.

The ORE Hub will accommodate several functions, such as new quayside infrastructure, pontoons and gangways to accommodate the berthage and mooring of Crew Transfer Vessels (CTV), landside infrastructure including warehousing, office infrastructure, parking, and also associated dredging works.

1.3 GEOTECHNICAL CATEGORY

The scheme has been identified to be Geotechnical Category 3 according to B.S. EN 1997-1:2005, in that it includes structures which fall outside the limits of Geotechnical Category 1 and 2 and that involve involving abnormal risks, difficulty in ground or loading conditions.

1.4 SCOPE OF REPORT

The scope of this report is as follows:

- Carry out a desk study of the site, including general and background geological information related to the proposed development area.
- Summarise the details of the ground investigations undertaken and all available geotechnical information, including, bathymetry survey, utility survey and available ground investigation data.
- Present the interpreted ground conditions and material properties for the main geological units encountered across the site.

GDG has reviewed the information available for the site, including site investigation information captured during the 2024 Causeway SI. The design parameters presented are based on the data reviewed to date, published values, and typical values used by GDG. Further analysis of the information may be required depending on the exact structural solutions adopted for specific locations and situations. These documents, together with this report, will constitute a Geotechnical Interpretive Report compliant with IS EN 1997-1: 2005.

2 THE SITE

2.1 SITE LOCATION

The proposed development site is located as the hinterland land and southeast region of the current Rosslare Europort in County Wexford, Ireland, near the south-eastern most point of the island of Ireland. Figure 2-1 shows the location and the national grid coordinate for the approximate centre of the Rosslare Europort ORE Hub Reclamation area is 52°15'18.69"N, 6°21'0.33"W. A full location plan can can be seen in Appendix A.

Figure 2-1 Site location (Bing Aerial, 2024)

2.2 CURRENT LAND USE AND SURROUNDING LAND USE

Rosslare Europort is Ireland's premier ferry Port servicing both passenger and commercial freight traffic to the United Kingdom and the European Mainland. The current Port, located east to the proposed development site is operated by Iarnród Éireann (Irish Rail) and offers twice-daily round services to the UK and direct services to the continent each day. It is connected to the national roads network from the N25 and N11 roads, which joins Rosslare to Waterford and Dublin.

Rosslare Europort has 4 berths that are primarily used for Roll-On Roll-Off ferry and freight traffic (RoRo). There is an additional small quay, Berth 5 also known as Fisherman's Quay. The area around the current harbour has developed into a small town with several shops, service industries, residential developments and community facilities as shown in Figure 2-2. The proposed development site currently comprises of marine area and a small craft harbour (Figure 2-3).

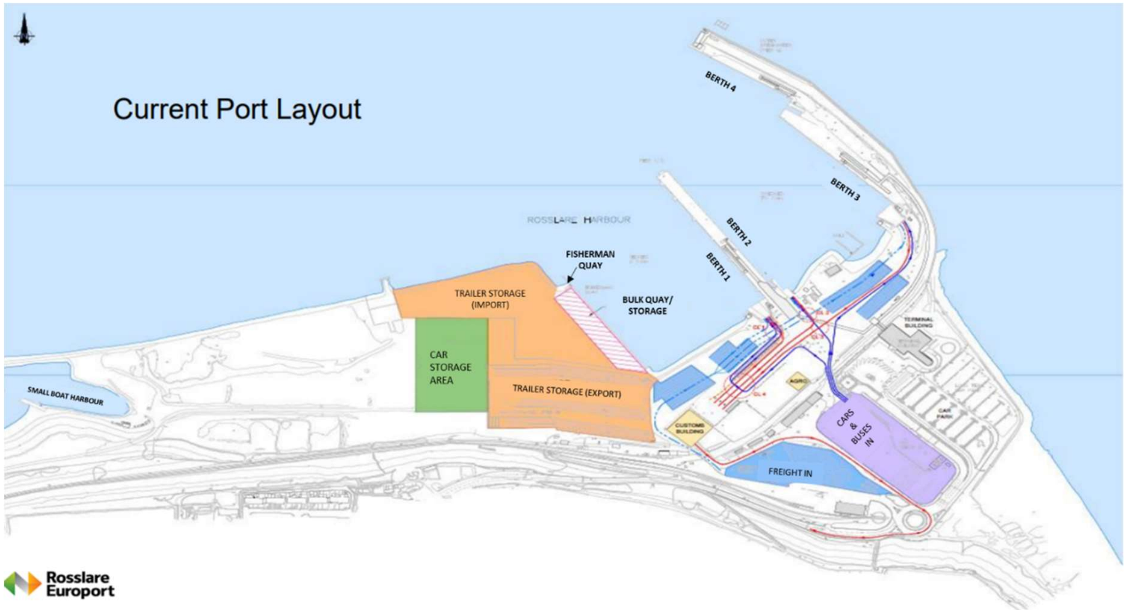


Figure 2-2 Existing Site Layout

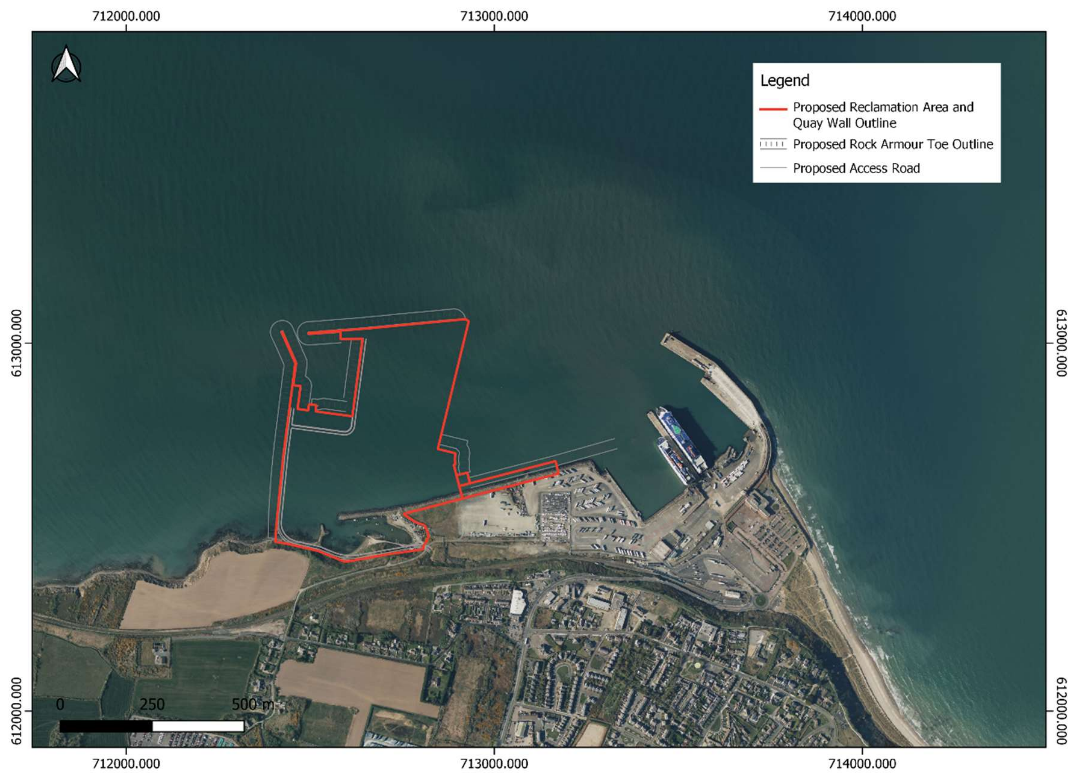


Figure 2-3 Aerial image looking towards the Rosslare port and ORE Hub development site (outlined in red) from south (Bing Aerial, 2024)

2.3 TOPOGRAPHY AND LAND FEATURES

An onshore topographic survey was carried out by Murphy Surveys in Summer 2023 in and around the harbour area and onshore part of the proposed reclamation site. For detailed topographic information please refer to the drawing no. MGS52642_T_ITM_Rev2_00.

2.4 TIDAL LEVELS

Rosslare Europort has a semi-diurnal tide regime: it registers two high tides and two low tides every day. Chart Datum taken as being 1.69m below OD Malin.

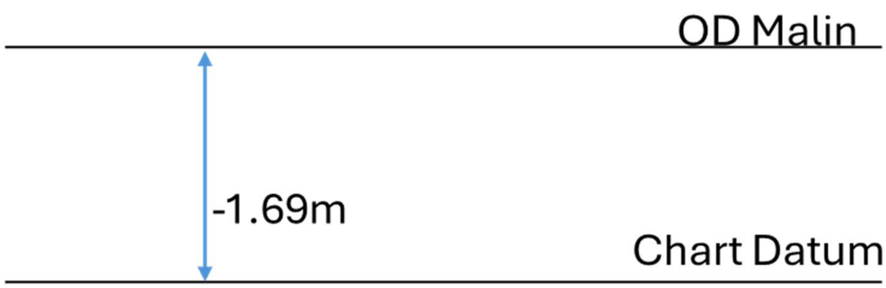


Figure 2-4 Chart Datum Relative to OD Malin for Rosslare

The mean average tidal levels considered in the design are presented in Table 2-1, with reference to UK Hydrographic Office Tide Tables 2019.

Table 2-1 Site Tidal Levels

Tidal Condition	Abbreviation	Chart Datum (m CD)	Ordnance Datum Malin Head (m ODM)
Highest Astronomical Tide	HAT	+2.6	0.91
Mean High Water Springs	MHWS	+2.3	0.61
Mean High Water Neaps	MHWN	+1.8	0.11
Mean Sea Level	MSL	+1.5	-0.19
Mean Low Water Neaps	MLWN	+1.1	-0.59
Mean Low Water Springs	MLWS	+0.7	-0.99
Lowest Astronomical Tide	LAT	+0.3	-1.39

2.5 BATHYMETRY

Several bathymetric surveys were carried out in and around the Rosslare Europort. A summary of the bathymetric surveys carried out around Rosslare Europort is provided in Appendix B. The latest Survey data we hold is:

- Hydromaster geophysical bathymetric survey, carried out in November 2023 around the proposed development site.
- Hydrographic Surveys Ltd (HSL) bathymetric survey, May 2023 encompassing part of the proposed development site, and the existing harbour.

An extract generated using the Hydromaster and HSL surveys is presented in Figure 2-5, and in Appendix B.

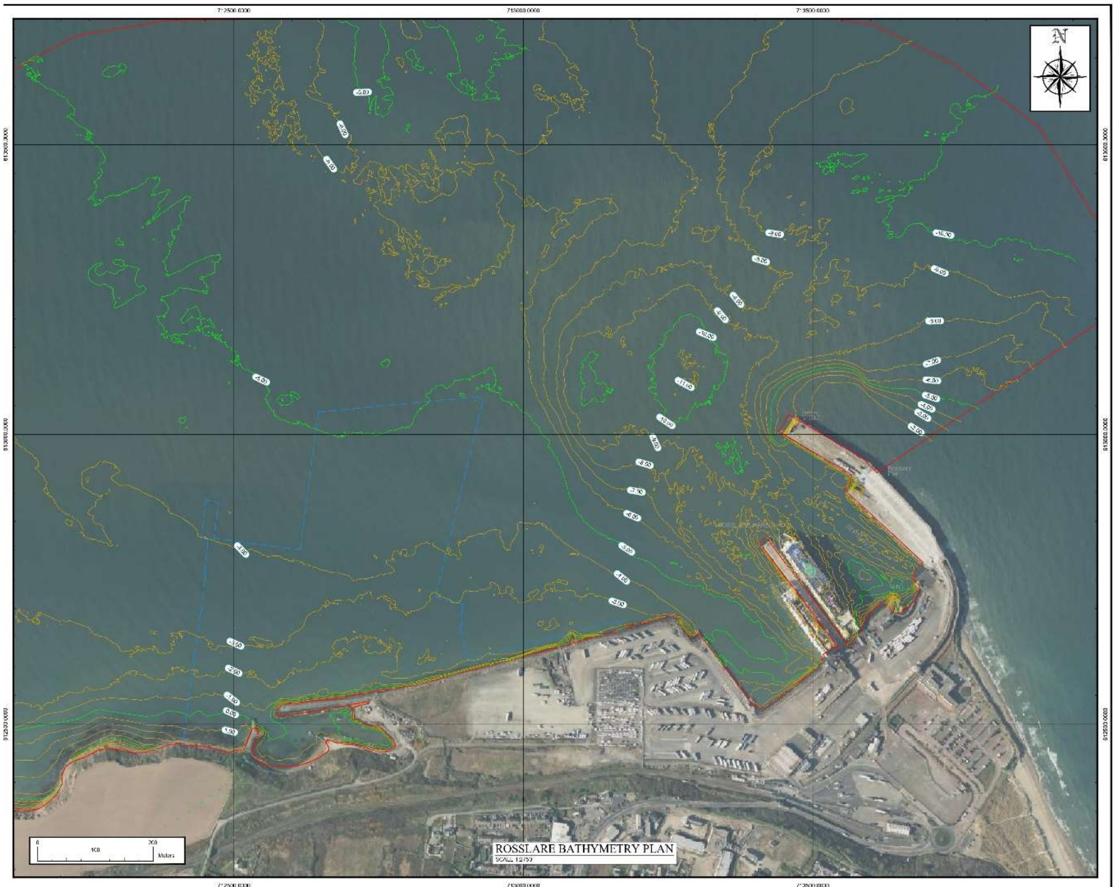


Figure 2-5: Bathymetry for the proposed development site from the Hydromaster and HSL 2023 surveys.

2.6 GEOLOGY

The bedrock geology and superficial geology of Rosslare Europort ORE development site are described below. The onshore data was made available through the Geological Survey of Ireland (GSI) website and offshore data taken from Infomar and EMODnet sources.

The bedrock geology from both the offshore and onshore portions of the site are presented at a 1:100,000 scale and are seen in Figure 2-6. The mapped offshore bedrock geology of the site is characterised by metamorphosed rock (Precambrian Gneiss) and onshore geology including the small craft harbour area bedrock geology characterised by Greenore Point Group. The Greenore Point Group comprises a thick sequence of foliated and banded amphibolites. The Greenore Point Group rocks are in faulted contact with younger Ordovician metasediments of the Tagoat Group (Milltown Formation and Grahmormack Formation) rocks.

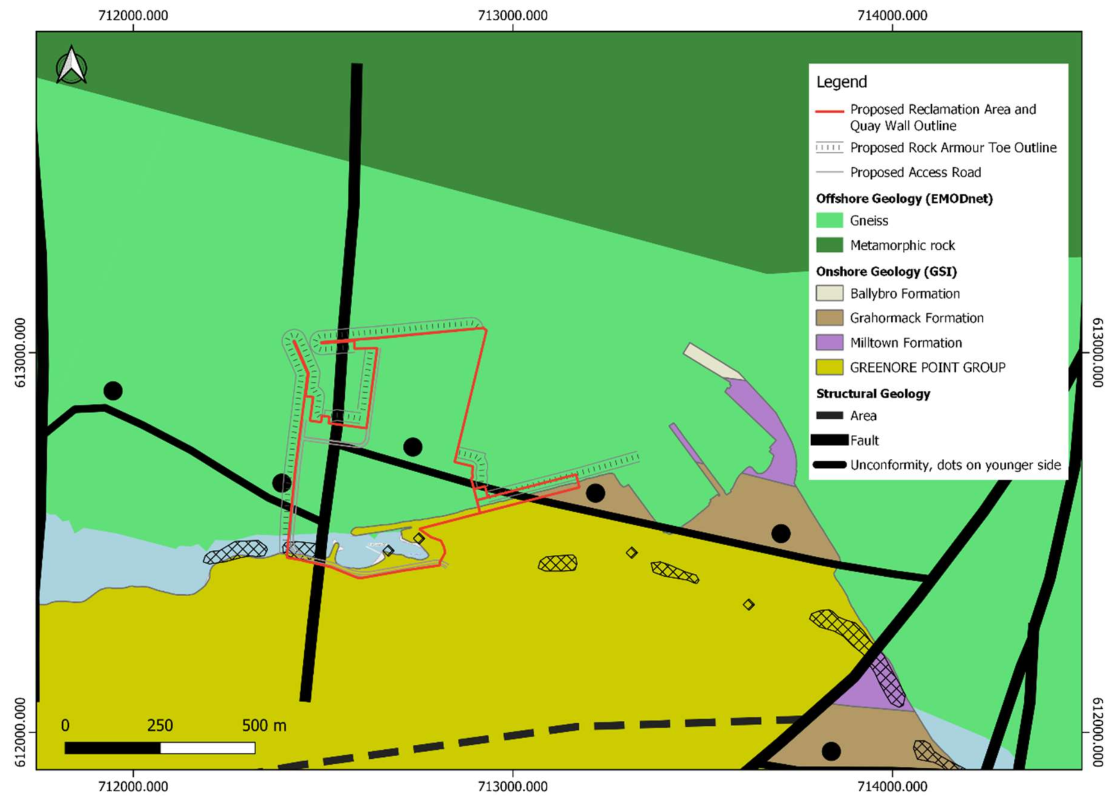


Figure 2-6 Bedrock Geology map at 100,000 scale (GSI,2022)

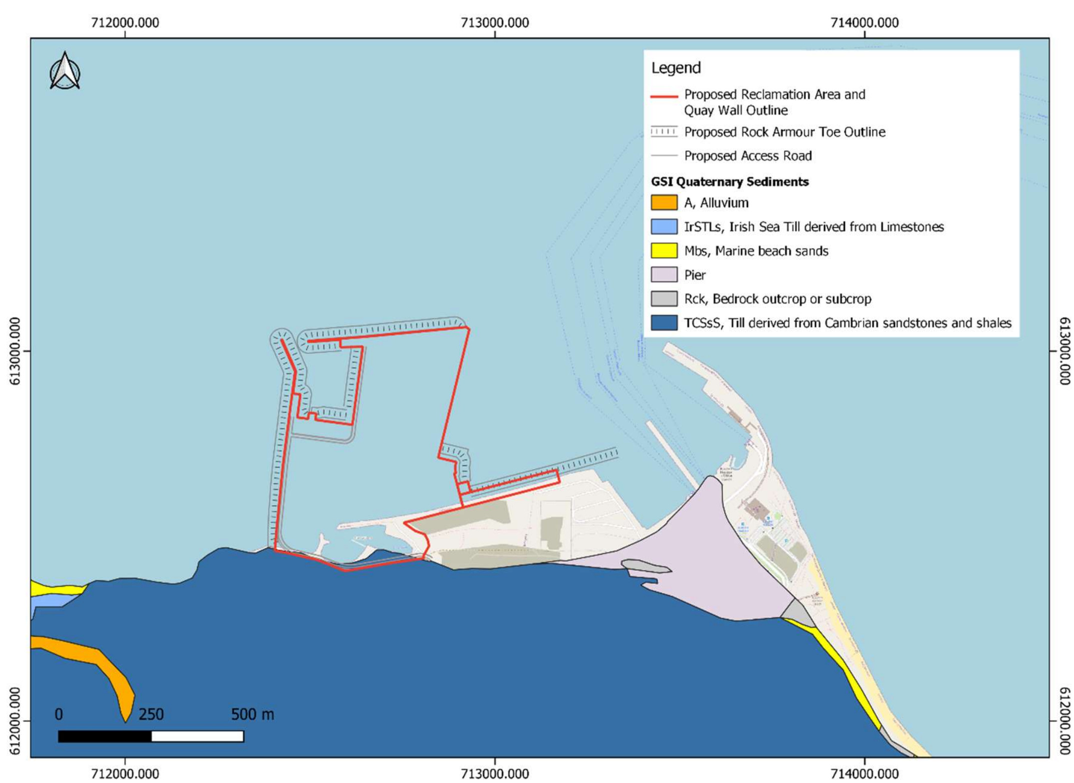


Figure 2-7 Quaternary Sediments map at 1:50,000 scale (GSI, 2024)

The onshore superficial geology was described as glacial till derived chiefly from Cambrian sandstones and shales (TCSsS), Irish Sea till, (IrSTLs) and marine beach sand along the coast line. Alluvium deposits and rock outcropping (Rck) were identified near to the site as well. Based on the onshore available data, it is believed the superficial deposits at the site comprises alluvium overlying glacial deposits. No offshore superficial geology information is available. For the marine region of the site. The Quaternary Sediments map (GSI) for the site is presented in Figure 2-7.

There were also recordings of the seabed substrate map taken from Emodnet dataset that is produced at a 1:250,000 scale, underlying the site. These substrate classes are defined in accordance with the Folk scale and are shown in Figure 2-8. Based on the information the seabed composition of the proposed development site is recorded as mainly sand.

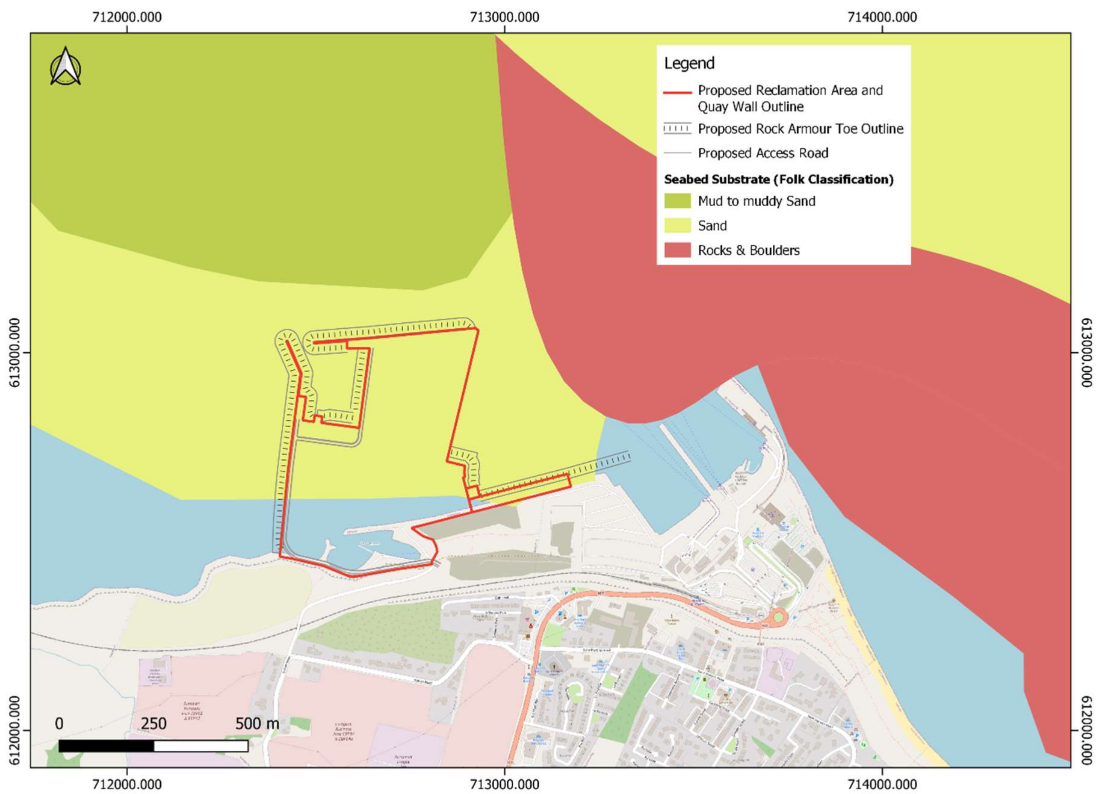


Figure 2-8 Seabed Sediment Substrate Map Folk Classification 250k Scale (EMODnet 2024)

2.7 SITE HISTORY AND PREVIOUS LAND USE

The historical data for the development of Rosslare Europort is very limited. Construction of Rosslare Europort in the late 1860's resulted in an interruption of the natural alongshore movement of sediment into Rosslare Bay. At construction, to accommodate this alongshore movement and allow sediment bypassing, a viaduct connecting an offshore pier to the shore was provided. However, shelter afforded by the offshore pier resulted in sediment settling in the lee of the pier thus necessitating a programme of maintenance dredging. Rosslare Europort was known as Rosslare Fort to locals when its first construction began in 1867. The first major development finished in August 1906 by the Great Western Railway and the Great Southern and Western Railway to facilitate traffic between Great Britain and Ireland. Between 1845 and 1855 approximately 2319 acres of the north slobs in Wexford Harbour and 2293 acres of the south slobs were reclaimed. The reclamation resulted in a 50% decrease in the inter-tidal area in the harbour and a 10% reduction in the tidal volume.

Ordnance Survey Ireland (OSi) National Townland and Historical Map Viewer allows for viewing of the historic 25-inch and 6-inch mapping series. Figure 2-9 taken from the 6-inch map series shows the site is developed with lighthouse, harbour station, pier, and rail line by 1969. Originally Rosslare Europort Station was located on a berth from land, accessed only by a wrought iron railway bridge. In the 1970s and 1980s land reclamation and the expansion of the harbour removed this feature, though some of the bridge supports remain. GDG are aware that significant port infrastructure construction work was completed along Fisherman's Quay in c1992. Based on the OSi 1995 data (Figure 2-10) the site underwent development around this period to include a new pier structure, small boat harbour and reclamation area.

In 2004, Rosslare Europort handled over 1.1 million passengers, 307 thousand passenger vehicles, 117 thousand units of roll-on/roll-off cargo, and 35 thousand trade vehicles. In 2005, the port added a new tow tractor unit, upgraded pier structure and mooring facilities at Berths 1 and 2.Figure 2-11, taken from the OSi 2006 shows all the upgrade works completed.

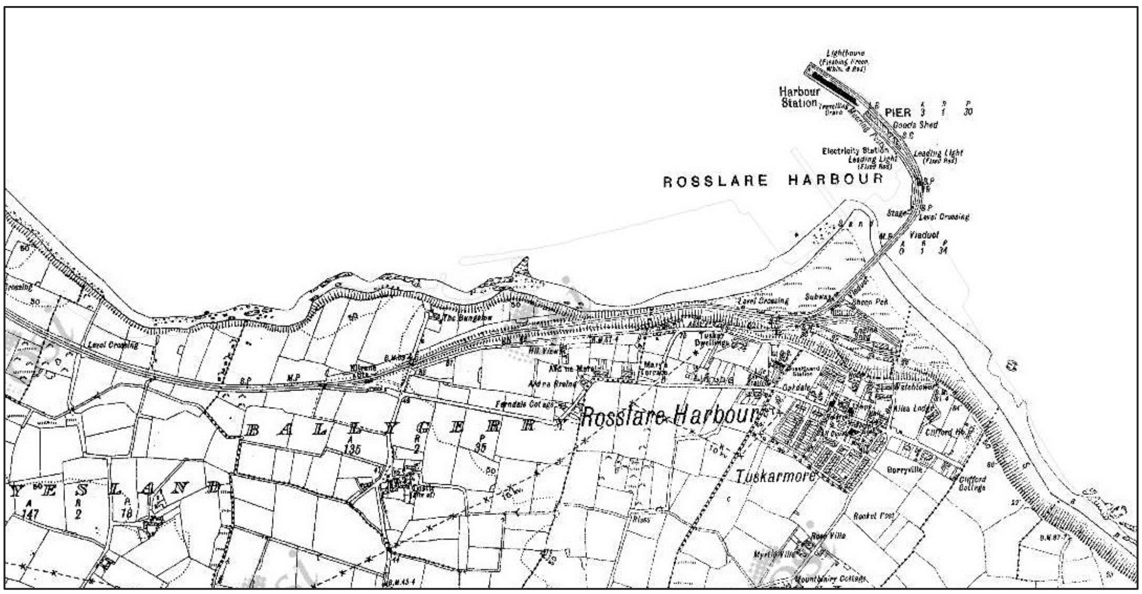


Figure 2-9 OSi Historic 6" last edition (1952-1969) map

Figure 2-10 OSi Aerial image from 1995

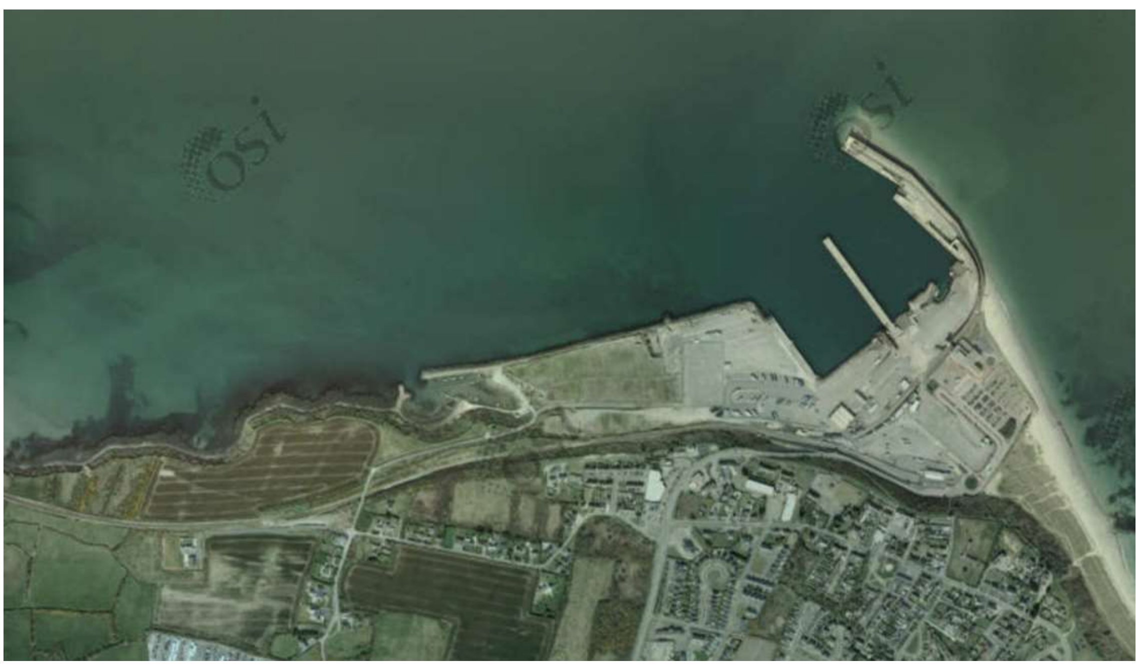


Figure 2-11 OSi Aerial image from 2006

2.8 DESIGNATED SITES

Figure 2-12 shows the SSSI sites around the 5km radius of the proposed study site. Recently, a candidate cSPA site 'Seas off Wexford' has been designated that crosses the proposed development site where further Environmental assessment is recommended.

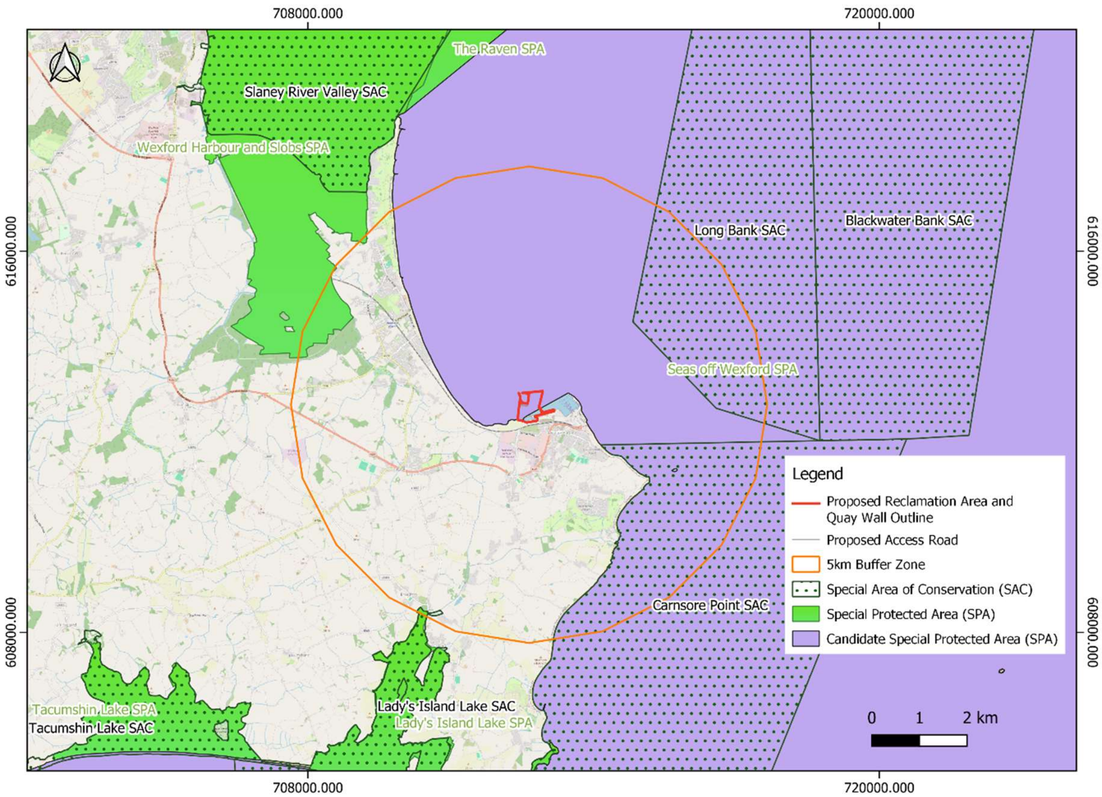


Figure 2-12 Map of SSSI around the site.

2.9 UNDERGROUND FEATURES

Based on the national database there is no indication of submarine cable or pipelines at the location of the site. No other underground features are known to GDG based on the information made available for the site.

3 PROPOSED DEVELOPMENT

The facilities will be designed and constructed such that they will be future-proofed to cater for future port operations, when at some point in time, site usage may change from offshore renewables to include other trades such as Roll-On Roll-Off (Ro-Ro) and Roll-On-Roll-Off-Passenger-Ship/Ferry (RoPax). During the consultations which have taken place thus far with Irish Rail and also consultation with the ORE industry and suppliers, the choice of a 330m minimum berth length with heavy lift capacity alongside has been critical in determining an appropriate layout for the facility. In addition, a new 240m long Roll-On Roll-Off (RoRo) berth is proposed using an open pile suspended deck, alongside the existing revetment structure in current reclaimed lands to further aid the delivery of offshore wind components. This open-piled structure allows wave absorption in the revetment beneath the deck rather than adding a reflective vertical wall to the harbour. The proposed design is shown in Figure 3-1, and in Appendix A.

Though it has not been identified as a critical requirement for the offshore wind industry, given the nature of their business, Irish Rail has requested that layouts are developed on the basis of based on accommodating a future rail freight siding, should there be a decision to implement this at a later point in time.

The proposed scheme will target the following main features:

- 21 hectares (circa 210,000 m2) of land reclamation for ORE quayside and component storage/staging at the westerly side of the existing Rosslare Europort.
- 330m long, heavy lift berth for accommodating ORE installation vessels approximately on a north by north-easterly alignment.
- 240m long Roll-On Roll-Off berth for delivery of components directly transported by modular vehicles into the storage yard for ORE. Sloping ramp at the stern end of RO-RO berth for vessels door/ramp to land upon and allow direct vehicular access into the vessel.
- Dredging the approach channel at circa 300m wide and to an effective depth of -10m Chart Datum (CD) with a declared depth of -9mCD to allow for sand mobility/sediment build-up and a depth buffer between future maintenance dredging campaigns.
- Main berth pocket for Staging and Installation (S&I) vessels is proposed as an effective dredged depth of -12mCD with a declared depth of -11mCD over an area of 330m x 60m wide.
- Scour and bed protection in the berth pocket under heavy impacts from repeated leg penetrations by S&I vessels. The precise details of necessary scour protection will only be determined after site investigation work is complete. There is the prospect of encountering bedrock in the dredged pocket and thus final orientation of the berth, location of the 330m pocket, and approach channel will need careful refinement in this design phase.
- Expected 3 hectares (equivalent to circa 30,000m2) of small boat harbour with much-improved access to small boats from deeper water offshore than currently exists. This facility will benefit existing small craft users and facilitates Operation & Maintenance (O&M) vessels as well as tugs and local fishing boats. Approx 44 berths for local vessels.
- O&M facility in the small boat harbour with warehousing, offices and carparking.
- RNLI berth(s) in the small boat harbour with administration building and car-parking.
- Dedicated access road to small boat harbour.
- Allow for future connection by rail spur into the site for future rail infrastructure.

- Access road to small craft harbour and site access to the ORE yard from the new roundabout proposed at the western end of the terminal.

Figure 3-1 Rosslare Europort ORE Design Layout

The various phases of the overall project and the respective anticipated timelines are set out in Table 3-1.

Table 3-1 Rosslare Europort Offshore Wind Hub – Project Phasing & Timelines

Project Phase	Anticipated Timeline
Phase 1 – Project Scope and Approvals	November 2021 – February 2023
Phase 2 – Project Concept, Feasibility & Option Selection	
Phase 3 – Preliminary Design	July 2023 – September 2024
Phase 4 – Planning and Statutory Process	
Phase 5 – Detailed Design and Tender Process	October 2024 – September 2025
Phase 6 – Contract Award, Construction and Implementation	October 2025 – December 2027*
Phase 7 – Close Out and Review	January 2028 – December 2028

** subject to gaining planning approval, and also to construction timescale based on contractor's programme*

4 GROUND INVESTIGATIONS

To adequately define the geotechnical conditions at the proposed location, a site investigation comprosed of geotechnical and geophysical campaigns, was carried out by Causeway Geotech between October 2023 to May2024. The site investigation was undertaken to provide geotechnical and geoenvironmental information to facilitate the design of the new offshore wind hub development within the boundary of Rosslare Europort. The full Causeway Factual Report (2024) can be seen in Appendix C. The SI plan is provided in Figure 4-1.

The geotechnical campaign comprised the following:

- 30 no. over-water nearshore marine boreholes; 24 no. completed by sonic drilling in overburden with rotary core follow-on in soil/rock off a jack-up barge (JUB), 6 no. completed by sonic drilling.
- 12 no. over-water nearshore marine Cone Penetration Tests with pore water measurement (CPTU's) with dissipation tests, completed by sonic drilling in soil/rock off a jack-up barge (JUB)
- 18 no. over-water nearshore marine Cone Penetration Tests with pore water measurement (CPTU's) with dissipation tests; completed off a jack-up barge (JUB)
- 4 no. over-water nearshore marine Cone Penetration Tests with pore water measurement (CPTU's) with dissipation tests; completed with drill outs off a jack-up barge (JUB)
- 59 no. over-water nearshore marine gravity corers (Vibrocores); completed from a multi-cat vessel
- 3 no. land-based boreholes; sonic drilling in overburden extended by rotary core follow-on in soil/rock
- 5 no. grab samples
- Geotechnical and geoenvironmental laboratory testing.

The geophysical surveys carried out at the site are comprised the following:

- Multibeam echosounder (MBES),
- Marine magnetometer,
- the sidescan sonar (SSS),
- Sub-bottom profiler (SBP), and
- Boomer seismic.

Findings and interpretations of the geophysical survey are outlined in the Hydromaster (2024) geophysical report in Appendix C.

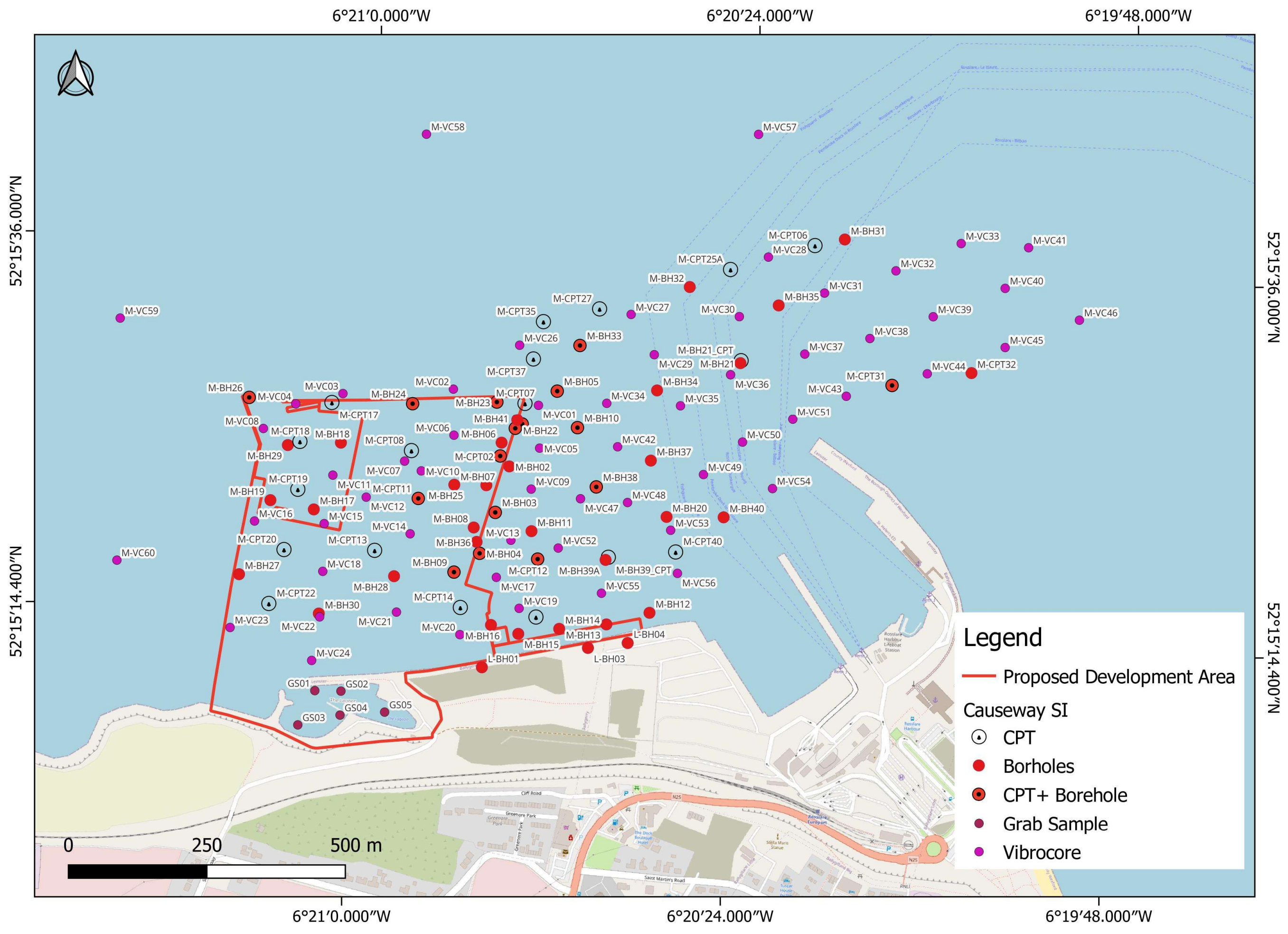


Figure 4-1: Causeway 2024 Site Investigation Plan

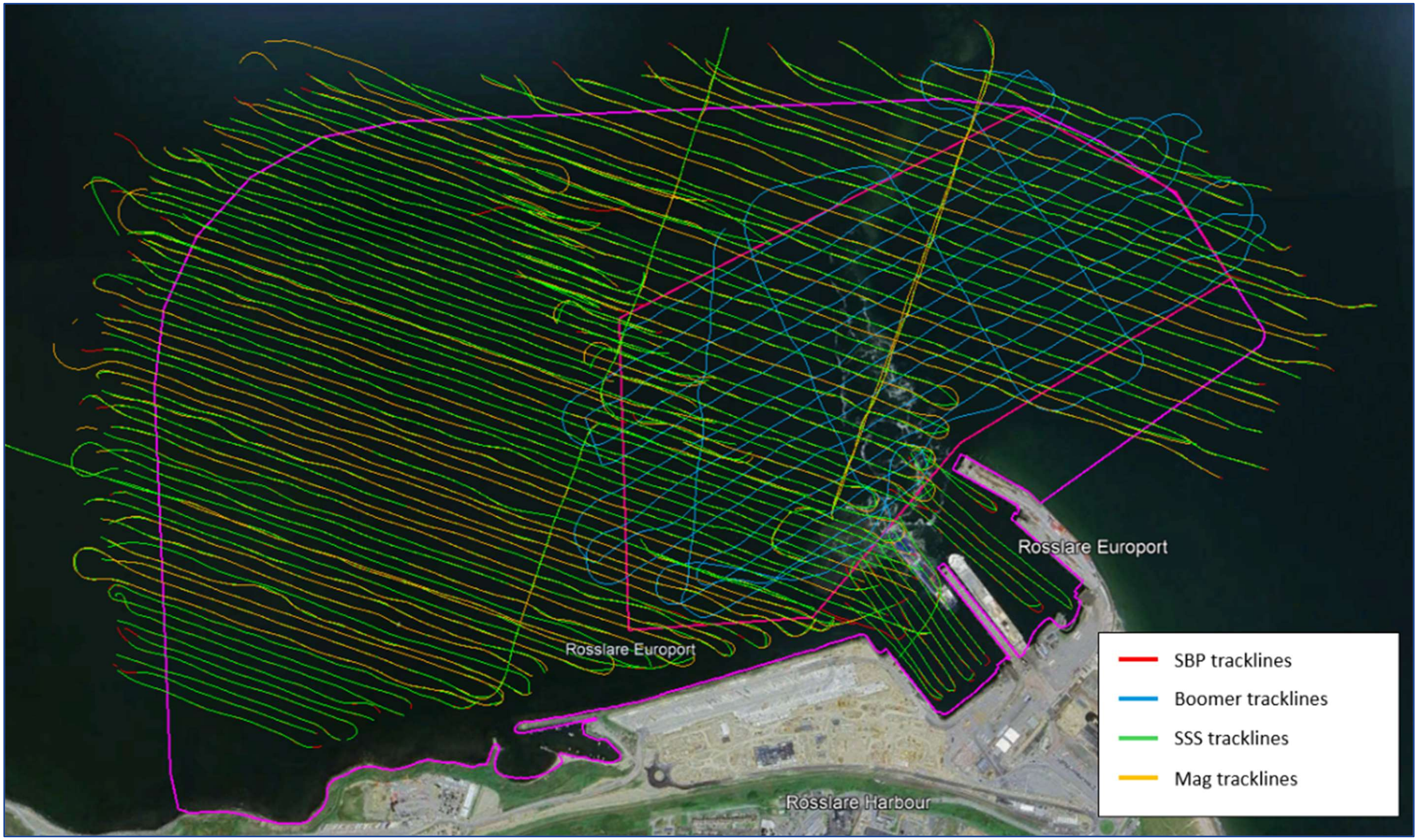


Figure 4-2: Geophyscial survey tracklines

5 Ground Conditions

5.1 Land Boreholes

5.1.1 Made Ground

Made Ground was present in all land boreholes with a thickness of 3.3 to 7.5m. This stratum was found to be typically medium dense to very dense sandy slightly silty gravel. L-BH04 described the lower part of the made ground as rock armour.

5.1.2 Granular Glacial Till

The granular deposits generally consist of sandy slightly clayey fine to coarse Gravel with occasional cobbles of amphibolite. Gravels are found to be various in shape ranging from subrounded to angular, though are typically described as subangular to angular. The layer varies in thickness from 0.6m to 5.7m. Some shell fragments and occasional boulders were encountered in this stratum.

5.1.3 Bedrock

Bedrock was encountered at levels ranging from -2.06 mCD to -4.21 mCD. Rock head was found to get deeper moving seaward. The bedrock is typically described as weak to very strong greenish grey amphibolite with occasional feldspar or quartz veins. No clear weathered zone is identified in land boreholes, but the amphibolite is described as highly fractured.

5.2 Marine boreholes

5.2.1 Cohesive Marine Deposits

The cohesive marine deposits comprise mainly of silt and clay from recent deposits. The marine sediments were not encountered in all exploratory holes but were generally present across most of the surveyed area. No distinct area where marine sediment was absent could be identified, however marine sediments were not identified in M-BH40, an area which had been recently dredged prior to the Causeway (2024) survey. The description from the borehole logs varies slightly. However, generally, the top material is described as very soft to firm sandy gravelly CLAY or slightly sandy gravelly clayey SILT. Sand is fine to coarse. Gravel is subangular to subrounded fine to coarse. Occasional layers of sand with frequent shell fragments are also recorded. This is the most superficial layer. It varies from seabed level, and its thickness may vary from 0.15 to 8.2m but is typically around 2m in thickness. Organic material is noted in some locations. One organic matter content test was undertaken on a sample from within this stratum, returning a result of 0.06%.

Disturbed and undisturbed samples were collected in the marine deposits and were tested for composition and strength parameters.

5.2.2 Cohesive Glacial Till

The second layer underlying the superficial marine deposits are largely cohesive glacial tills. The glacial till was not encountered in all exploratory holes but was generally present across most of the surveyed area. These are typically described as stiff to very stiff brown to grey sandy gravelly CLAY with occasional cobbles. The thickness of the layer ranges from 0.5m to 9m but is typically around 2.8m.

Disturbed and undisturbed samples were collected in the glacial till material and were tested for composition and strength parameters.

5.2.3 BEDROCK

Bedrock was encountered in all boreholes, but due to the limited depth ranges accessible, was not encountered by all CPTs, or by vibrocores. Rockhead was found to be variable, but levels typically fell with distance from shore. This can be seen in Figure 5-1.

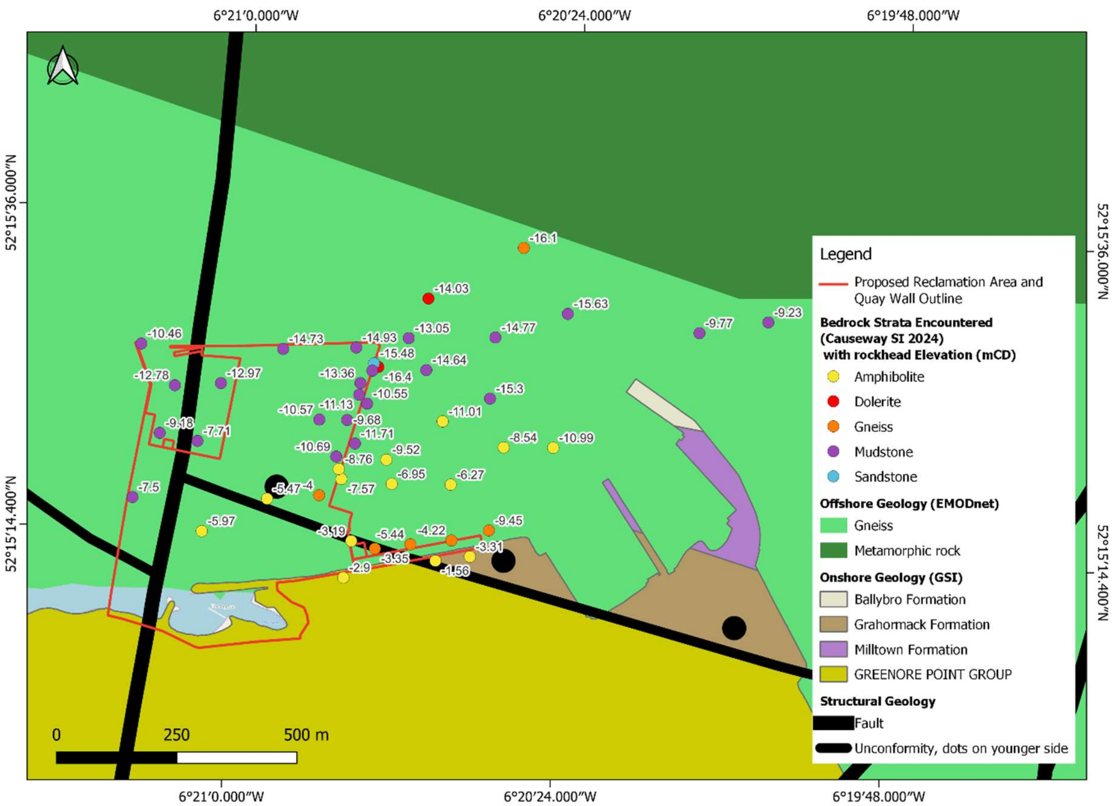
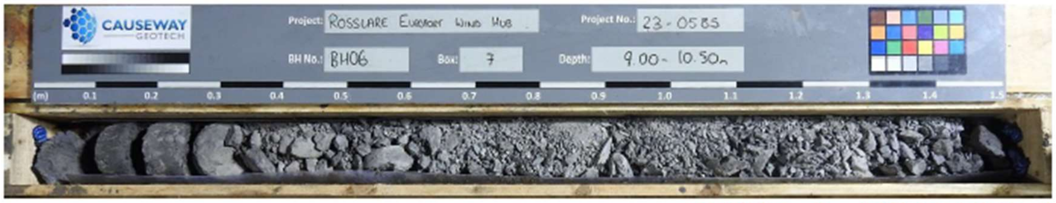


Figure 5-1: Bedrock strata encountered and rockhead elevations, with existing GSI and EMODnet geological mapping.

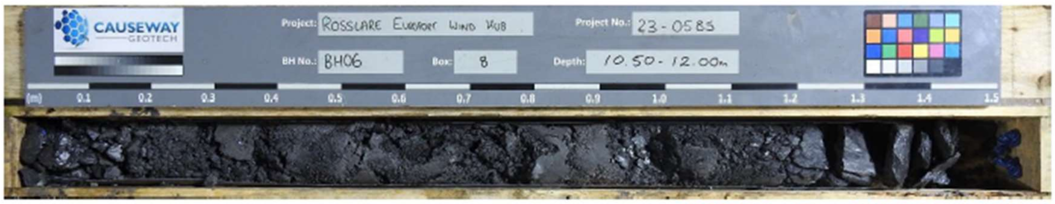
5.2.3.1 WEATHERED MUDSTONE

At the rockhead indicator, a zone of weathered MUDSTONE of variable thickness is identified and typically described as grey sandy silty clayey angular fine to medium GRAVEL. This weathered band ranges from 0.3 to 10.3m in thickness but is typically around 2.1m.

The recovered core samples are of poor quality with a typical core recovery (TCR) typically ranging from 1.5% to 100%, and an average of 81%. The solid core recovery (SCR) defined as the percentage of the recovered core run which contains solid cylindrical pieces of core with their full diameter, is highly variable, typically ranging from 0-71%, with an average of 26% recorded. The rock quality designation (RQD), measured as a percentage of the drill core in lengths of 10 cm or more, typically ranges from 0-46%, with an average of 10% recorded, indicating a high degree of discontinuities. Typical core recovery of the weathered mudstone can be seen in Figure 5-3.



M-BH06 – Box 7 (9.00-10.50m)



M-BH06 – Box 8 (10.50-12.00m)

Figure 5-2: Weathered mudstone recovered from M-BH06.

Due to the highly fractured and blocky nature of this material, recovery of full-diameter core samples was difficult, and samples often became friable when recovering from the core liner. This led to difficulties in obtaining samples for point load and uniaxial compression testing.

5.2.3.2 MUDSTONE

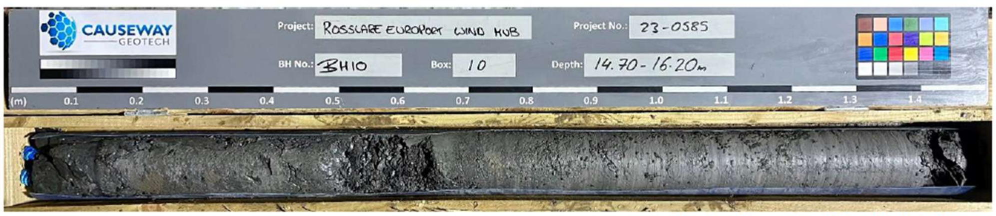
Stratigraphically, the first competent bedrock stratum to be encountered by exploratory holes is described as MUDSTONE. The rotary borehole locations describe extremely weak to medium strong, but predominantly weak, thinly laminated grey MUDSTONE.

The recovered core samples are of moderately poor quality with a typical core recovery (TCR) typically ranging from 38% to 100%, and an average of 84%. The solid core recovery (SCR) defined as the percentage of the recovered core run which contains solid cylindrical pieces of core with their full diameter, is highly variable, typically ranging from 0-100%, with an average of 39% recorded. The rock quality designation (RQD), measured as a percentage of the drill core in lengths of 10 cm or more, typically ranges from 0-62%, with an average of 12% recorded, indicating a high degree of discontinuities. Solid core recovery, illustrating the thinly laminated mudstone can be seen in Figure 5-3. Poor core recovery, demonstrating the highly fractured, friable nature of much of the mudstone can be seen in Figure 5-4.

Due to the highly fractured and blocky nature of this material, recovery of full diameter core samples was difficult, and samples often became friable when recovering from the core liner. This led to difficulties in obtaining samples for point load and uniaxial compression testing.

M-BH10 - Box 8 (11.70-13.20m)

M-BH10 - Box 9 (13.20-14.70m)



M-BH10 - Box 10 (14.70-16.20m)

Figure 5-3: Competent mudstone bedrock core recovery from M-BH10, illustrating the fine lamination encountered within the mudstone.

M-BH07 – Box 9 (10.05-11.55m)

M-BH07 – Box 10 (11.55-13.05m)

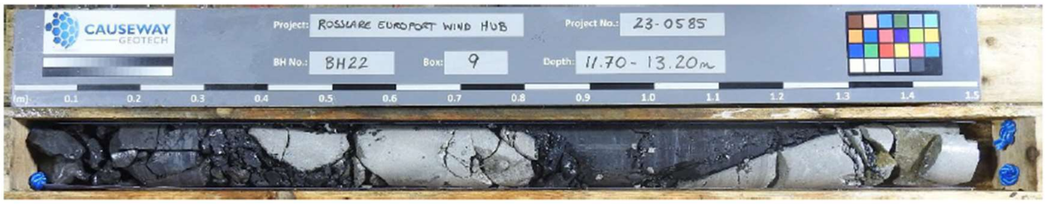
M-BH07 – Box 11 (13.05-14.05m)

Figure 5-4: Highly fractured and fissile mudstone recovered from M-BH07, illustrating the poor recovery prevalent within the mudstone strata.

5.2.3.3 SANDSTONE

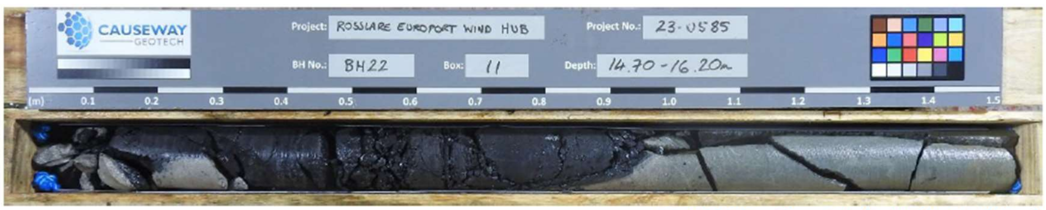
Sandstone bedrock is encountered in M-BH22 and M-BH41, with the sandstone found to be underlying the mudstone in M-BH41. This unit is described as moderately weak to medium strong light grey fine to coarse-grained SANDSTONE, sometimes interbedded with thick beds of moderately weak dark grey MUDSTONE, locally highly fractured. Two joint sets at 10-30° and 60-70° respectively are noted in M-BH22, with sub-horizontal fractures noted in M-BH41. The sandstone is potentially a blockier member within the mudstone strata, and likely indicates a more proximal depositional environment within the mudstones. This unit is likely a subunit of the mudstone strata and is only encountered in two boreholes. As such, characteristic parameters have not been defined, though testing has been carried out on samples taken from this material. The results of these can be seen in Section 5.11.

The recovered core samples are of good quality with a TCR typically ranging from 82% to 100%, and an average of 93%. The SCR is highly variable, typically ranging from 4-86%, with an average of 48.5% recorded. The RQD typically ranges from 1-31%, with an average of 15% recorded, indicating a high degree of discontinuities. An example of the sandstone core recovery can be seen in Figure 5-5.



M-BH22 - Box 9 (11.70-13.20m)

M-BH22 - Box 10 (13.20-14.70m)



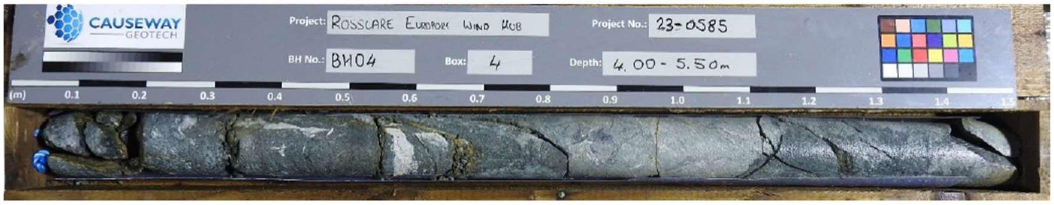
M-BH22 – Box 11 (14.70-16.20m)

Figure 5-5: Sandstone core recovery from M-BH22, interbedded with mudstone.

5.2.3.4 AMPHIBOLITE/GNEISS

The dominant bedrock lithology encountered in the nearer shore exploratory hole locations is typically described as strong to moderately strong greenish grey amphibolite or gneiss. A weathered band is encountered at the rockhead indicator in some places, varying from 0.3 to 1m in thickness. The competent amphibolite is generally a dark coarse-medium grained banded metamorphic rock. It is intersected by smooth, planar, tight locally purple clay-smeared, locally quartz, chlorite, calcite or plagioclase feldspar-filled, locally moderately iron-oxide or chlorite-stained fractures of sub-vertical, 45° and locally sub-horizontal dip.

The recovered core samples are variable, but generally of good quality with a TC typically above 90%. The SCR is variable, typically ranging from 0-100%, with an average of 71.3% recorded. The RQD typically ranges from 0-98.5%, with an average of 36.6% recorded, indicating a high degree of discontinuities. Highly fractured sections of core made recovery of whole core samples difficult in some localised areas. Examples of amphibolite core recovery can be seen in Figure 5-6, and gneiss core recovery can be seen in Figure 5-7.

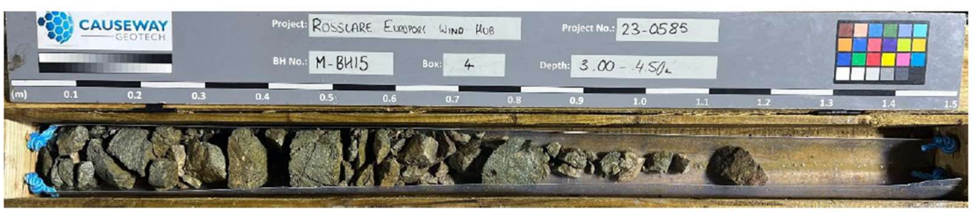


M-BH04 – Box 4 (4.00-5.50m)

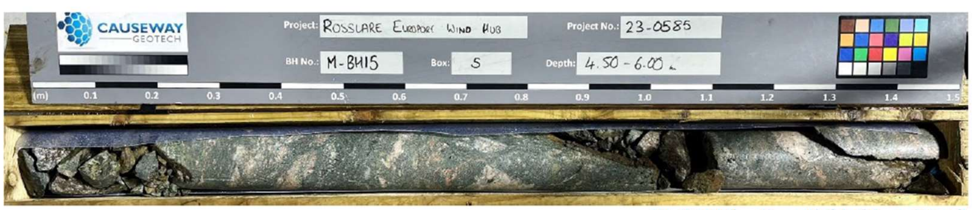
M-BH04 – Box 5 (5.50-7.00m)

M-BH04 – Box 6 (7.00-8.50m)

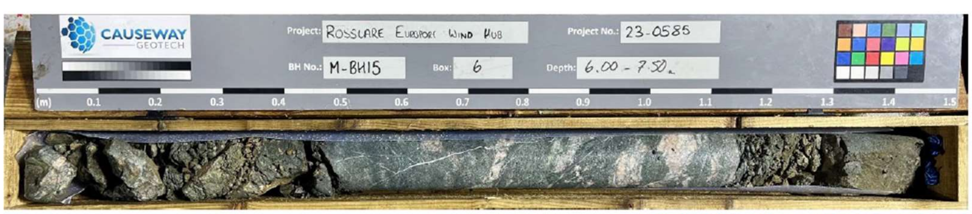
Figure 5-6: Amphibolite core recovery from M-BH04.



M-BH15 – Box 4 (3.00-4.50m)



M-BH15 – Box 5 (4.50-6.00m)



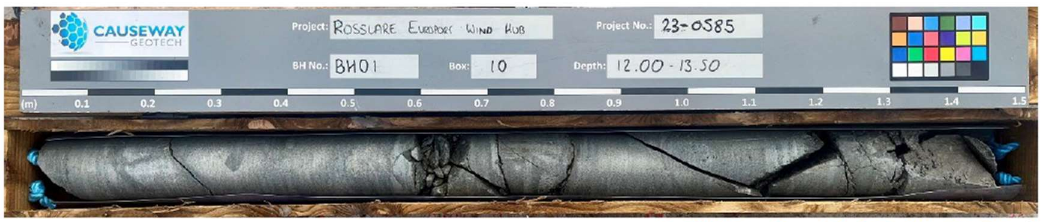
M-BH15 – Box 6 (6.00-7.50m)

Figure 5-7: Gneiss core recovery from M-BH15, illustrating competent core alongside highly fractured, poorly recovered core runs.

5.2.3.5 DOLERITE

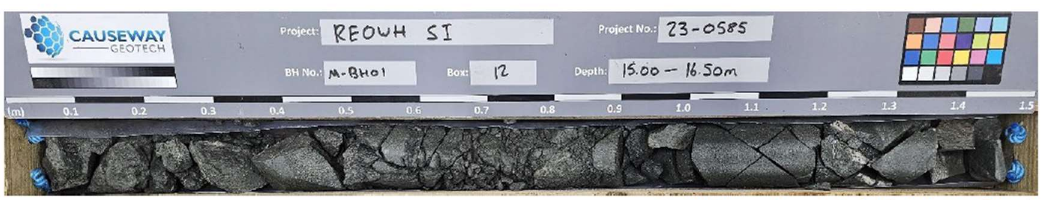
Isolated intrusions of dolerite are identified in M-BH01 and M-BH33, likely intruded within the Ordovician mudstones, but this is unproven. The unit is typically described as weak to medium strong dark grey DOLERITE, with rare inclusions of plagioclase feldspar. The dolerite is typically described as being slightly weathered, and is intersected by sub-horizontal, sub-vertical and 55-65° joint sets. These are typically described as planar-undulating, rough with occasional silt infill and brown staining on the joint surfaces.

The recovered core samples are of good quality with a TC typically above 90%. The SCR is variable, typically ranging from 12-100%, with an average of 68.8% recorded. The RQD typically ranges from 0-98.5%, with an average of 18.7% recorded, indicating a high degree of discontinuities. An example of dolerite core recovery can be seen in Figure 5-8.



M-BH01 – Box 10 (12.00-13.50m)

M-BH01 – Box 11 (13.50-15.00m)

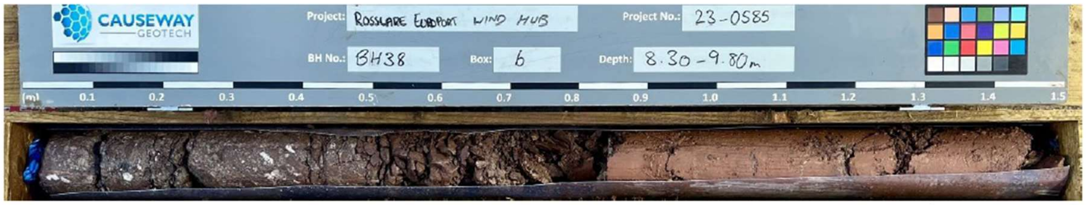


M-BH01 – Box 12 (15.00-16.50m)

Figure 5-8: Dolerite core recovery from M-BH01.

5.2.3.6 MYLONITE

In M-BH38, mylonite was encountered between 8.8 and 11.3m BGL. This unit was described as extremely weak to medium strong narrowly foliated light grey highly fractured MYLONITE, highly reduced in strength. One highly closely spaced foliation fracture set at 60-65° is noted, described as planar, smooth, with light grey clay smear on the fracture surfaces. This is interpreted in the Causeway report as possible fault gouge. An example of the mylonite core recovery can be seen in Figure 5-9.



M-BH38 – Box 6 (8.30-9.80m)

Figure 5-9: Mylonite (possible fault gouge) recovery from M-BH38.

5.3 Cone Penetration Testing

A total of 24no. cone penetration tests (CPT)we carried out at the site. CPT testing involves the penetration of a static cone into the subsurface or seabed at a constate rate, the cone measures the changes in the tip resistance, sleeve friction and porewater pressure through various sensors on the cone. The test is pushed consistently through the soil until a refusal depth or agrees termination depth, developing a results profile through the subsurface. The recorded cone parameters and their relative ratios can be used to correlate soil types and engineering parameters.

All CPT tests were carried out from the marine Jack Up platform using the sonic drilling rigs, Royal Eijkelkamp CPT'n Drill system. CPTs were carried out with porewater pressure measurement and dissipation testing was completed at various depths within the locations to characterise the settlement and consolidation parameters of the material. A variety of test types were carried out as part of the Causeway (2024) campaign, including:

- 12 no. marine Cone Penetration Tests with pore water measurement (CPTU's) carried out within sonic borehole locations,
- 18 no. stand-alone marine Cone Penetration Tests with pore water measurement (CPTU's),
- 4 no. marine Cone Penetration Tests with pore water measurement (CPTU's); completed with drill outs following shallow refusals of the test.

CPTs were carried out from the seabed mudline, with occasional flush depths below the mudline where the installed proprietary casing sat within the seabed. In general, the CPTs performed well with test profiles extending through the soft to firm marine sands, silts and clays, with tests ceasing or refusing within the high strength glacial tills and/ or the weathered bedrock materials beneath these strata. Common refusal criteria for the CPT locations were tip resistance, rod bend and maximum cone inclinations.

The cone application class was recorded in accordance with ISO 22476-1. This method is a quality control for the resulted data, measuring the differences in the zero readings of the cone before and after the test. Most test results indicate application Class 1 in both relative and absolute classes. Five tests indicated a Class 2 application class in the absolute tip resistance an acceptable test performance for the firm to stiff material in which these tests were performed. Two class 3 tests results were recorded however second tests were completed at these locations with improved results.

5.3.1 Material characterisation by CPTs

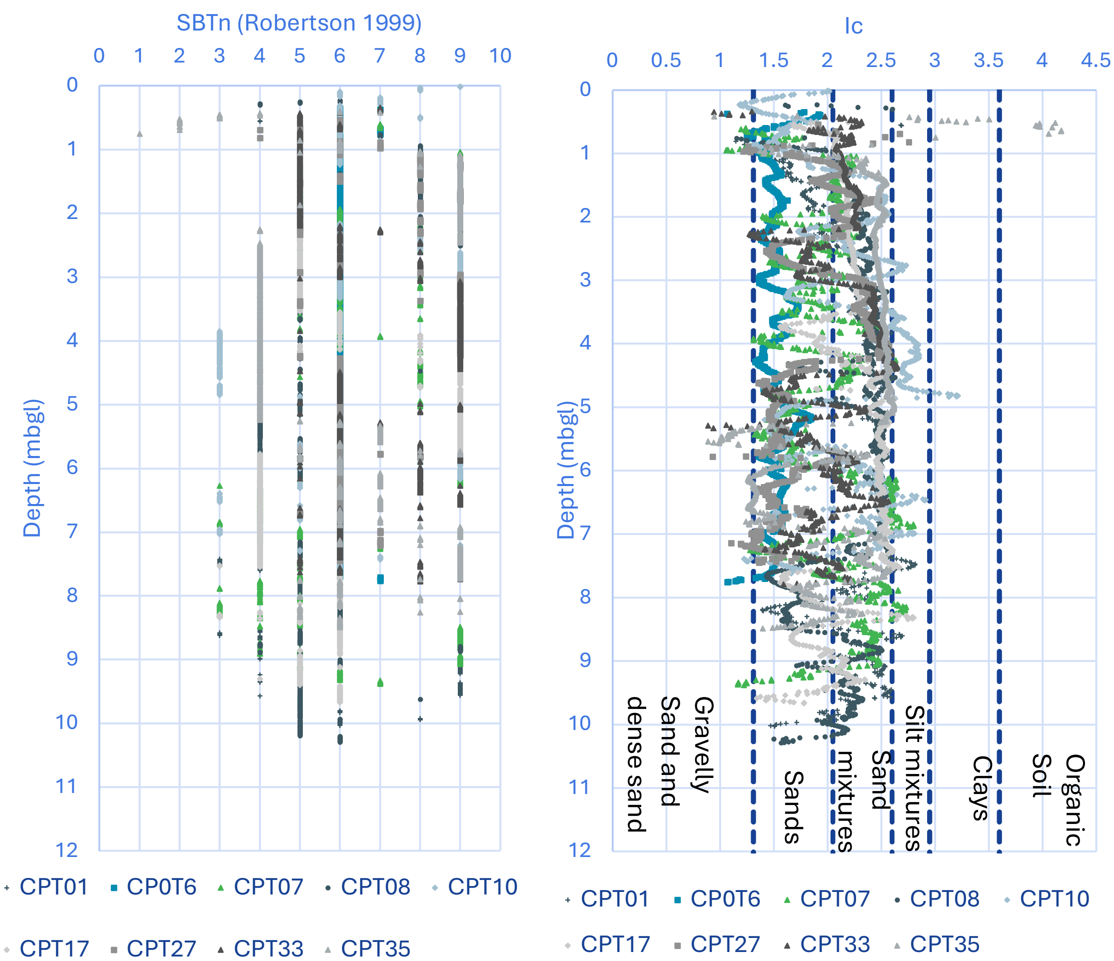
CPTs can be processed to derive a 'soil behaviour type' based on Robertson (2010). The soil behaviour characterisation is the term used for identification of the soil type based on the CPT results alone and the relative ratios of the collected senor data from the CPT. This provides a guide as to the soil type being encountered but requires some interpretation. During the ground investigation campaign, a number of over-sampling 'drilling-outs' were carried out which enable a comparison of the CPT test results and the physical sample of the soil strata.

In general, the CPTs within the marine sediments identified a mixture of sands, silts and clays with some occasional organic materials in the upper 0.5m of the seabed. Once the test entered the underlying glacial till material, there is a clear rise in the granular material with large variation in the porewater pressure due to granular materials and cavitation on the cone within over consolidated cohesive materials.

Due to the number of CPTs carried out across the site, a smaller number of tests are outlined within this report which represent the extent of the marine sediments across the site. The full set of CPT results with derived parameters are outlined in the Causeway Geotech (2024) Factual Report.

The derived SBT values and material types outline a very variable material composed of interbedded clean sand, sand-silt mixtures and silt mixtures with the occasional coarse sand or gravel band, and organics material at shallow depths.

The soil behaviour types (SBT) as derived from Robertson (1999) and a soil behaviour parameter (I_c) adapted for SBT by Jefferies and Davies (1993) for location CPT01, 06, 08, 10, 17, 27, 33 and 35 are outlined in Figure 5-10.



Zone	Soil Behavior Type	I_c
1	*Sensitive, fine grained*	N/A
2	*Organic soils – clay*	> 3.6
3	*Clays – silty clay to clay*	2.95 – 3.6
4	*Silt mixtures – clayey silt to silty clay*	2.60 – 2.95
5	*Sand mixtures – silty sand to sandy silt*	2.05 – 2.6
6	*Sands – clean sand to silty sand*	1.31 – 2.05
7	*Gravelly sand to dense sand*	< 1.31
8	*Very stiff sand to clayey sand**	N/A
9	*Very stiff, fine grained**	N/A

Figure 5-10: CPT characteristic parameters

The results of the CPT soil behaviours types suggests a similar strata type to that identified in the borehole and vibrocore locations with some variation in the primary component of the soil, often suggesting a granular primary soil component whereas the borehole have suggest a more cohesive primary component to the marine deposits. This is examined further in Section 5.7.3.

5.4 GROUND MODEL SUMMARY

For the purpose of the design and this GIR, only the Causeway (2024) GI locations relevant to the proposed location of the ORE hub are considered, as outlined in Section 4. The land boreholes show that the sequence of strata encountered is comprised of Made Ground, glacial till, and amphibolite bedrock. Marine boreholes, CPTs and vibrocore locations indicate that the ground consists of cohesive marine deposits, overlying glacial tills, with a range of bedrock geological units encountered beneath the glacial till, including amphibolite, gneiss, schist, mudstone, sandstone, dolerite and mylonite. A summary of the ground model for the land locations is given in Table 5-1, and a summary of the ground model for the marine locations is given in Table 5-2.

Table 5-1: Ground Summary for Land locations.

Material Name	Typical Description	Elevation (mCD)		Depth (mBGL)			Thickness approx. (m)		
		Min.	Max.	Min.	Max.	Ave.	Min.	Max.	Ave.
Overburden Deposits									
Made Ground	Medium dense to very dense greenish grey to dark brown sandy slightly silty GRAVEL of various lithologies with medium cobble content.	-2.7	6.1	0	7.5	0	3.3	7.5	5.1
Glacial Till (Granular)	Medium to very dense sandy slightly clayey fine to coarse GRAVEL of various lithologies.	-3.31	2.8	3.3	9	5.1	0.6	5.7	2.9
Solid Geology									
Metamorphic Rock	Weak to very strong greenish grey AMPHIBOLITE with occasional feldspar or quartz veins.	-7.56	-2.06	7	9	8	unproven		

Table 5-2: Ground Summary for Marine locations.

Material Name	Typical Description	Elevation (mCD)		Depth (mBGL)			Thickness approx. (m)		
		Min.	Max.	Min.	Max.	Ave.	Min.	Max.	Ave.
Overburden Deposits									
Marine sandy silt/clay	Very soft to firm grey sandy gravelly SILT/CLAY or gravelly SAND with frequent shell fragments	-14.8	0.9	0	8.2	0	0.2	8.2	2
Glacial Till (Cohesive)	Stiff to very stiff brown to grey sandy gravelly CLAY with occasional cobbles.	-17.7	-2.2	0	11.5	5.1	0.5	9	2.8
Solid Geology									
Sedimentary Rock	Weathered MUDSTONE: recovered as grey sandy silty clayey angular fine to medium GRAVEL	-19.5	-3.4	0.2	14.9	5.5	0.3	10.3	2.1
	Extremely weak to medium strong thinly laminated light to dark grey MUDSTONE.	-30.6	-8.25	3.6	-	8.4	unproven		
	Moderately weak to medium strong light grey fine to coarse-grained SANDSTONE, sometimes interbedded with MUDSTONE*.	-22.0	-15.5	10.9	-	13.7	unproven		
Metamorphic Rock	Weak to very strong greenish-grey AMPHIBOLITE or GNEISS with occasional feldspar or quartz veins.	-19.0	-3.2	1.7	-	3.8	unproven		
	Extremely weak to very weak narrowly foliated light grey to reddish brown weathered MYLONITE.	-14.7	-13.7	8.8	-	9.3	unproven		
Igneous Intrusions	Weak to moderately weak reddish brown and grey DOLERITE with rare white inclusions of plagioclase feldspar**.	-22.9	-14.4	7.9	-	9.7	unproven		

*Sandstone is encountered in 2 no. boreholes only, in association with mudstone. This is likely a blocky sandstone formation within the mudstone stratum.

** Dolerite is encountered in 2 no. boreholes only, and is likely locally intrusive within the mudstone.

5.5 GEOLOGICAL CROSS-SECTIONS

A series of geological cross-sections based on the ground model outlined in Section 5.4 are illustrated in Figure 5-12 to Figure 5-17. A location plan is shown in Figure 5-11.

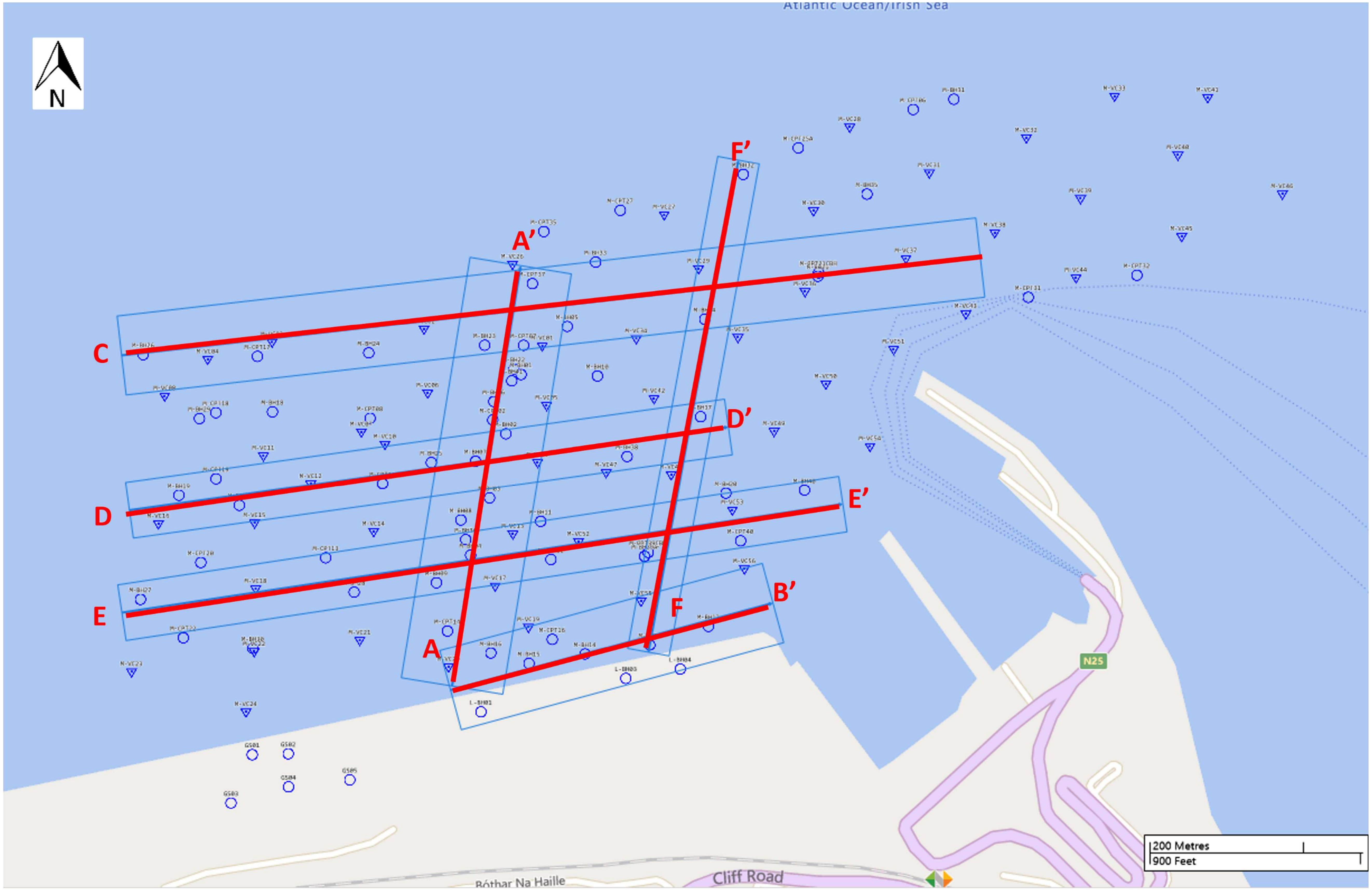
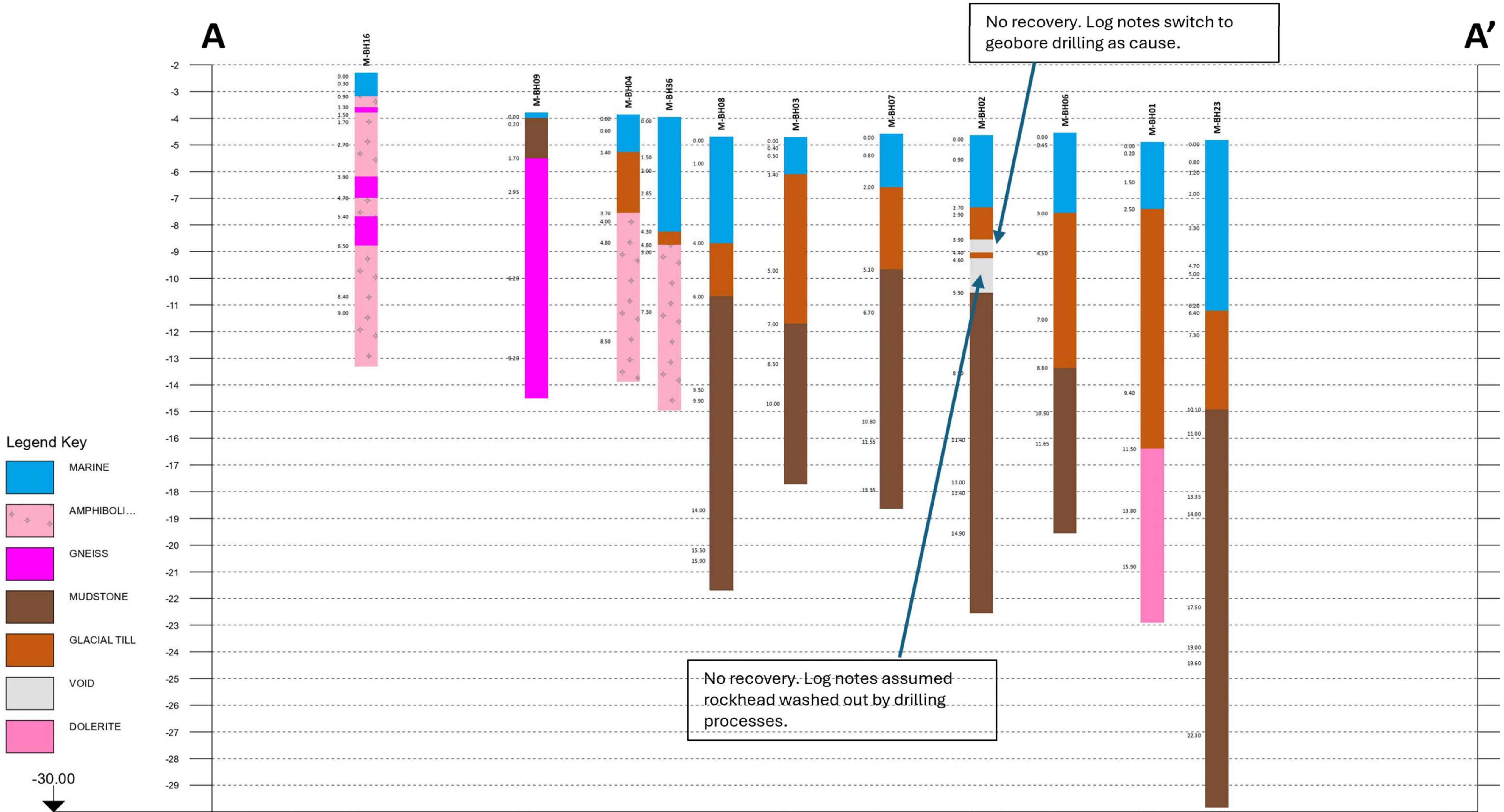


Figure 5-11: Cross-section location plan.



Chainage (m)	Elevation (mCD)	Offset (m)
0.00		
21.87		7.96
50.46	-2.29	43.45
69.85	-7.91	17.25
129.77	-3.80	41.64
136.39	-2.95	35.15
172.46	-3.87	3.35
191.49	-3.96	12.87
215.78	-4.69	23.34
250.37	-4.71	9.09
294.88	-4.58	16.79
336.62	-4.65	16.70
351.61	-4.53	3.19
375.64	-4.56	5.72
406.05	-4.85	13.11
416.19	-4.90	23.72
421.78	-4.63	13.45
446.23	-4.83	29.38
[illegible]	[illegible]	[illegible]
535.79	-5.23	19.15
[illegible]	-5.16	10.29

Figure 5-12: Cross Section based on the marine boreholes at the proposed quay wall.

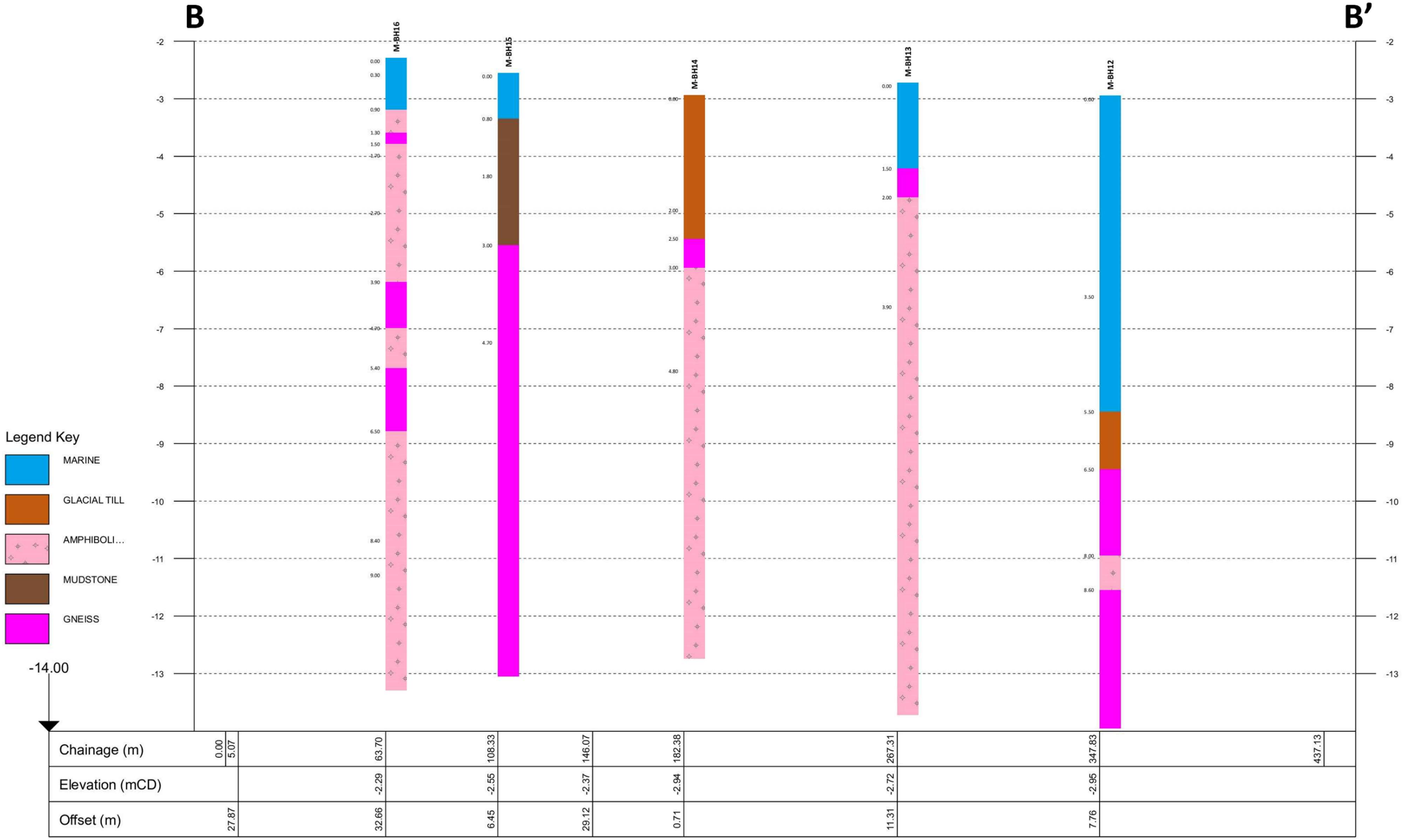


Figure 5-13 Cross Section based on the marine boreholes at the proposed roll-on-roll-off berth.

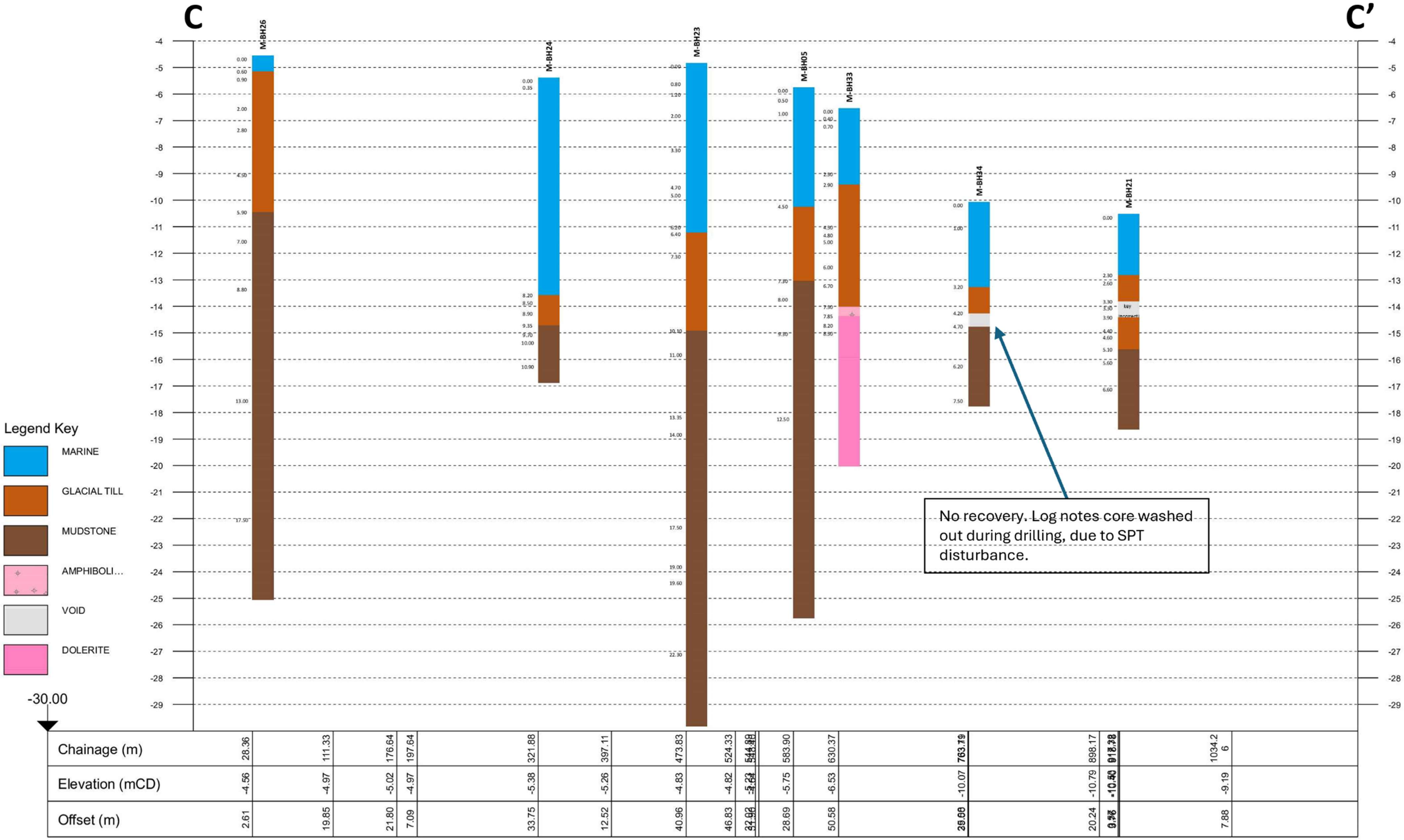


Figure 5-14: Cross-section based on the marine boreholes (far offshore).

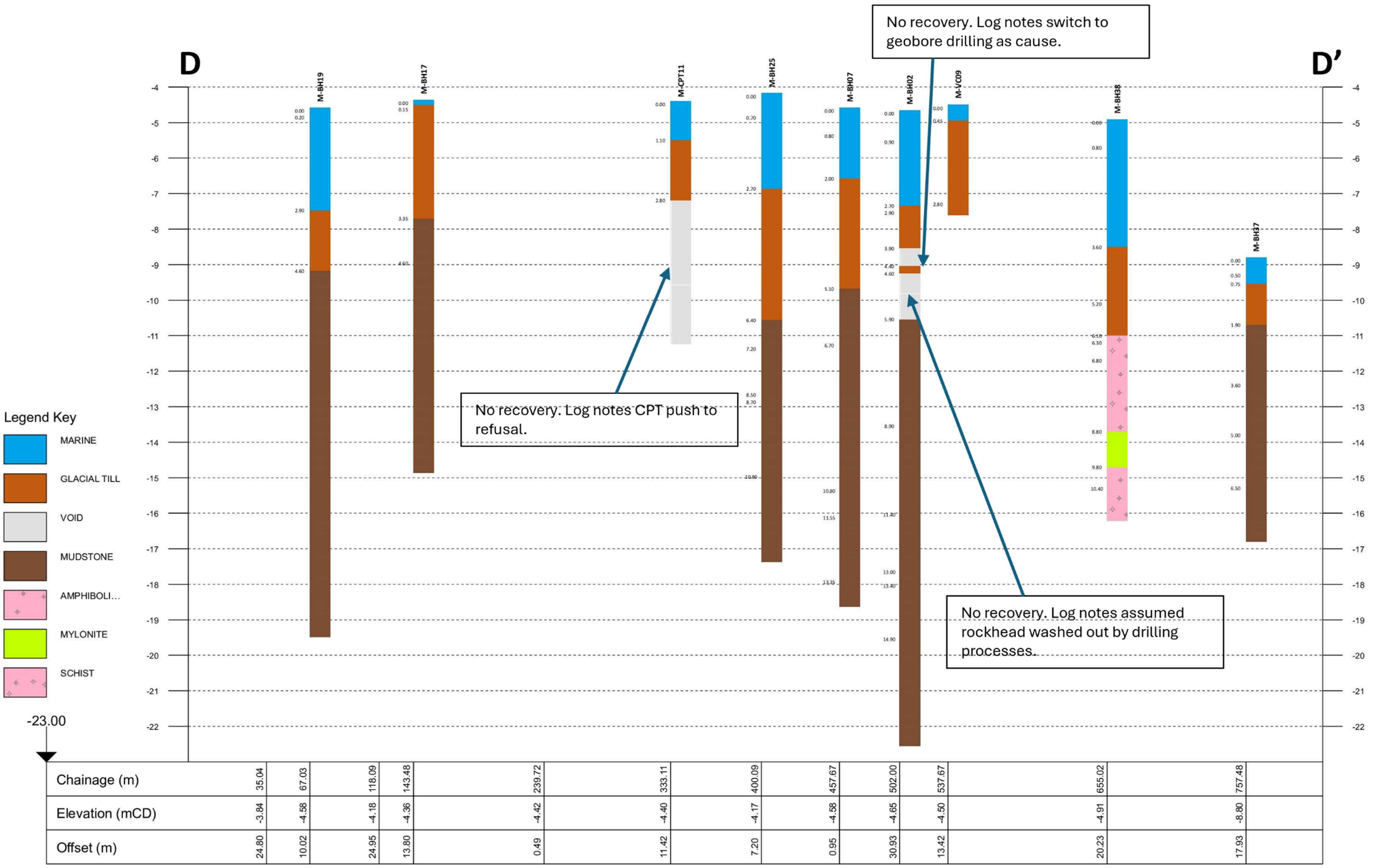


Figure 5-15: Cross-section based on the marine boreholes (mid offshore).

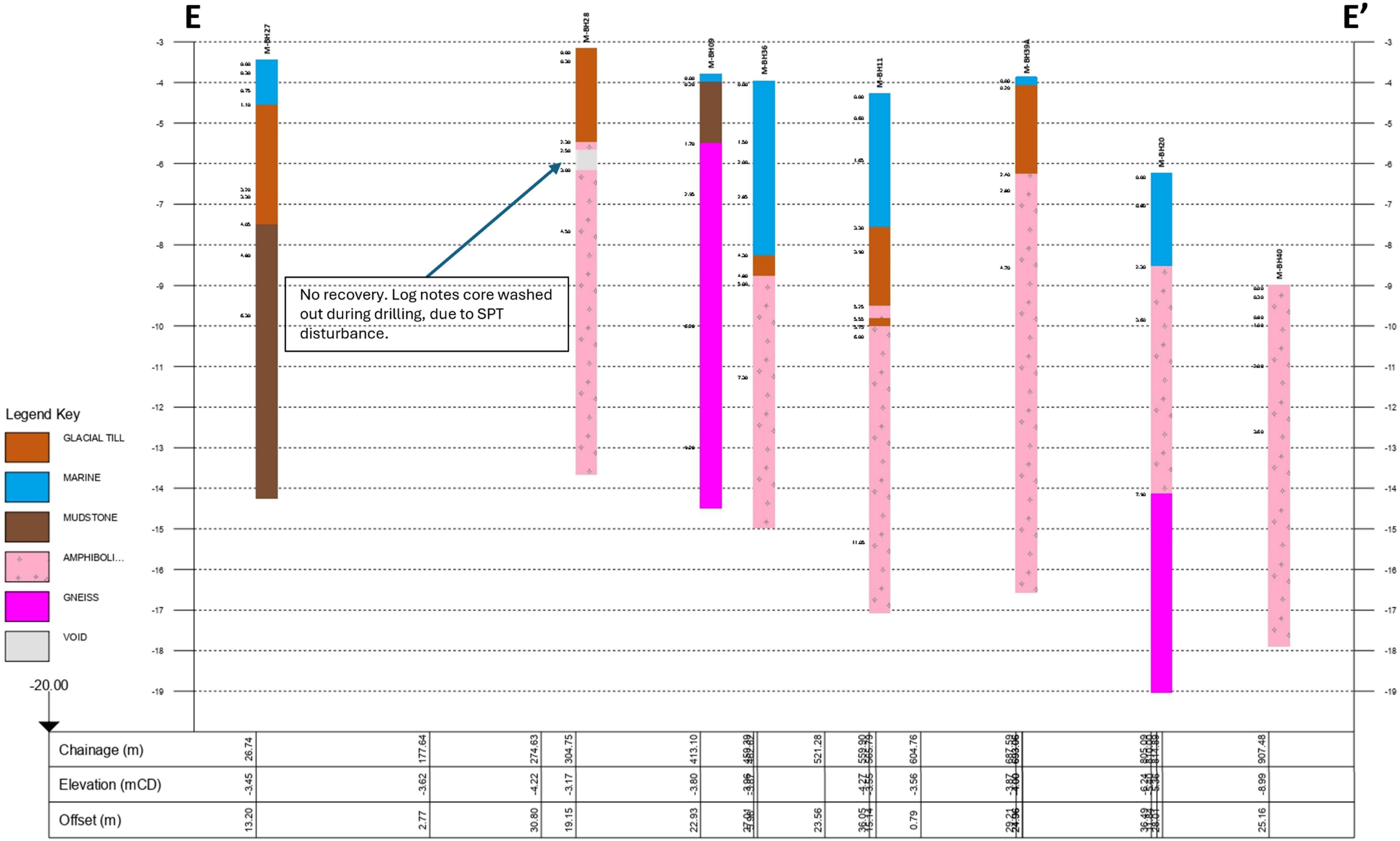


Figure 5-16: Cross-section based on the marine boreholes (near offshore).

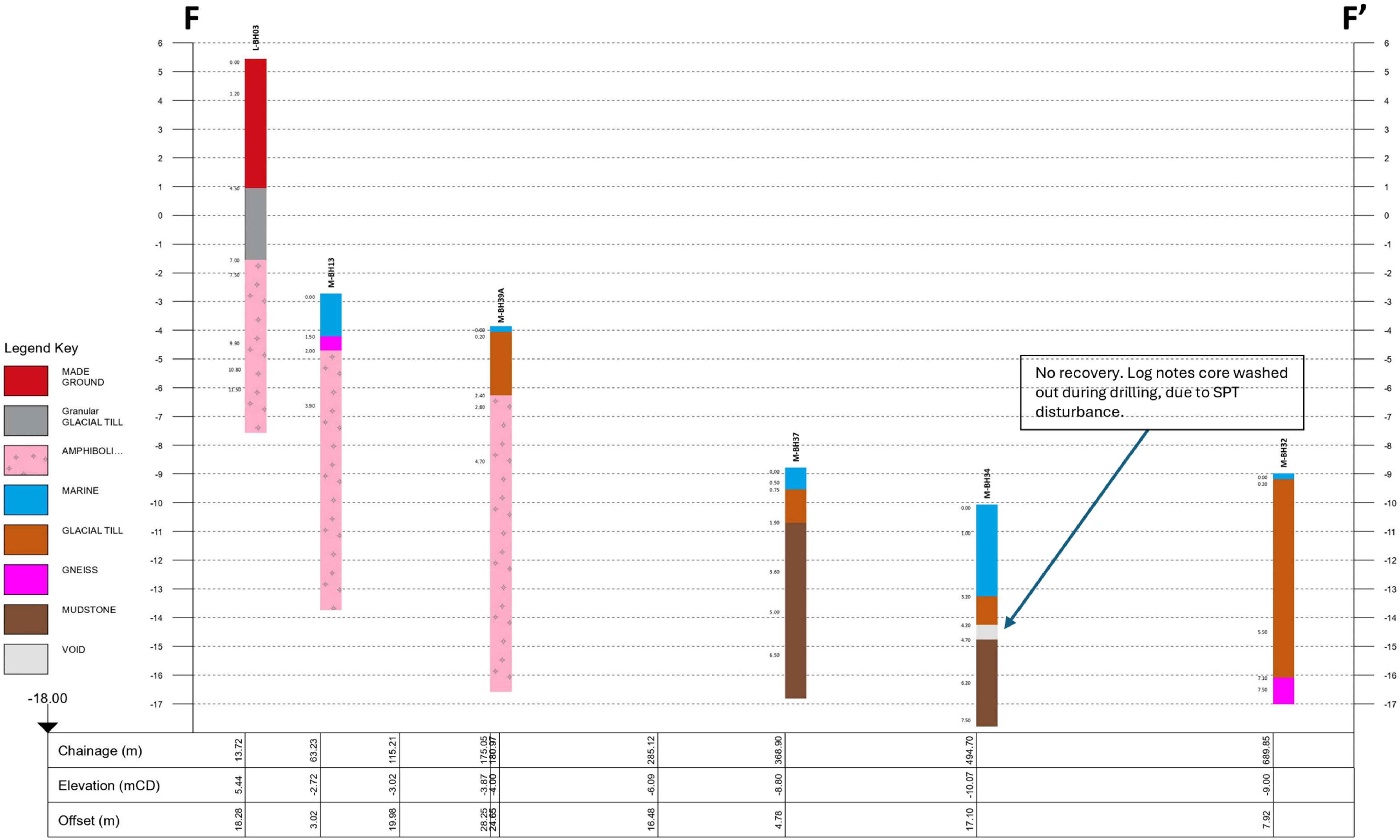


Figure 5-17: Cross-section based on the land and marine boreholes S-N near the existing harbour

5.6 STANDARD PENETRATION TESTING

SPT tests were carried out in 39 boreholes at standard depth intervals throughout the overburden using the split spoon sampler (SPT(s)) or solid cone attachment(SPT(c)). The test results are deduced through the number of blows needed to drive the cone each 450mm into the ground, with 150mm of seating blows, and 300mm of test blows. The “standard penetration resistance” or “SPT N value” is calculated by the sum of the number of blows required for the last four 75 mm increments of penetration, to a maximum of 50 blows following seating penetration, or 100 total. The measured number of blows, (SPT N), presented in Figure 5-18 is uncorrected, and no allowance has been made for energy ratio corrections. SPT N values above 50 blows per 300mm are considered to be refusals. The SPT-N profile suggests that the density of the material increases with depth.

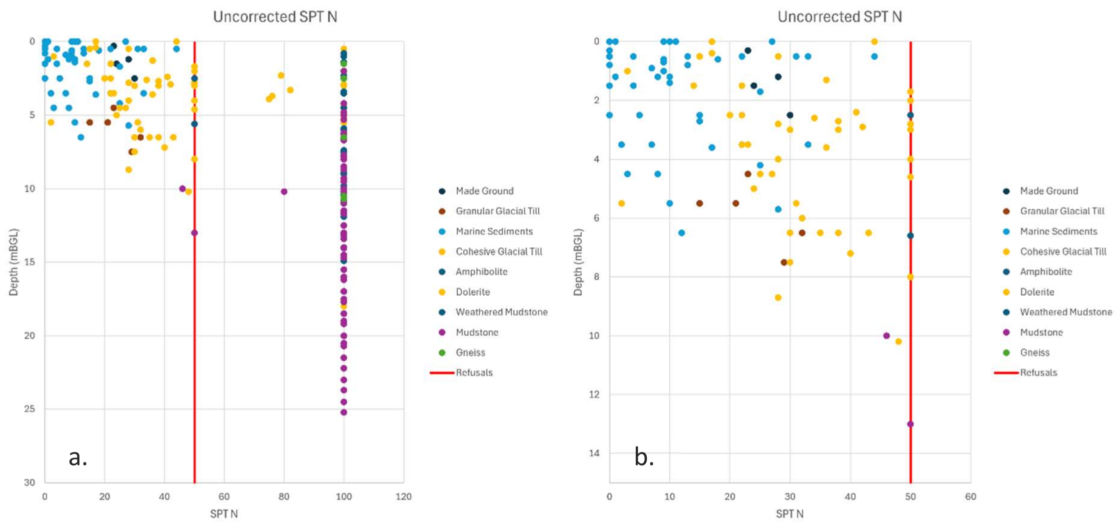


Figure 5-18 SPT N Values vs Elevation based on soil types and boreholes (a) including refusals), (b) excluding refusals.

Table 5-3 Summary of SPT test results

Land Uncorrected SPT						
	Count	**min**	**max**	**Ave.***	**10th Percentile**	**20th Percentile**
Made Ground	10	22	100	63	23	24
Granular Glacial Till	8	15	100	53	19	22
Amphibolite	1	100	100	100	100	100
Marine Uncorrected SPT						
	Count	**min**	**max**	**Ave.***	**10th Percentile**	**20th Percentile**
Cohesive Marine Deposits	43	0	44	12	0	1
Cohesive Glacial Till	57	2	100	45	19	24
Weathered Mudstone	21	50	100	98	100	100
Mudstone	97	46	100	98	100	100
Amphibolite	12	100	100	100	100	100
Dolerite	1	100	100	100	100	100
Gneiss	5	100	100	100	100	100
Mylonite	1	100	100	100	100	100
Sandstone	1	100	100	100	100	100

*Average values are highly influenced by the high number of refusal (N=100) values recorded. These are therefore not considered to be representative.

Based on Figure 5-18 and Table 5-3, a characteristic uncorrected SPT-N blow count of 12 is suggested for the cohesive marine sediments, and 35 is suggested for the cohesive glacial till. The average values for the cohesive glacial till are considered to be unrepresentative due to the high number of N=100 (refusal) values.

5.7 Classification Testing

5.7.1 Atterberg Limits and Moisture Contents

Atterberg limit testing was carried out on 65 samples from the marine borehole locations across the study area. The selected samples were a combination of disturbed tub samples and bulk bag samples collected from the sonic drilling methods. Samples were collected from the cohesive marine deposits, and from the cohesive glacial till. Moisture content (MC) tests were carried out on a further 85 samples taken from the marine boreholes, collected from the cohesive marine deposits and the cohesive glacial till, on disturbed tub samples and bulk bag samples.

The results of the Atterberg limit and moisture content tests are shown in Figure 5-19. The liquid limit (LL) values of the cohesive marine material varied between 22% and 66% and the plastic limit (PL) results varied between 13% and 31%. The plasticity index (Ip) results for the cohesive marine material range between 7% and 35%. In the cohesive glacial till material, the LL range between 20% and 46%, while the PL results vary between 13% and 23%, with a Ip ranging between 6% and 26%. The plasticity index results are shown in Figure 5-20.

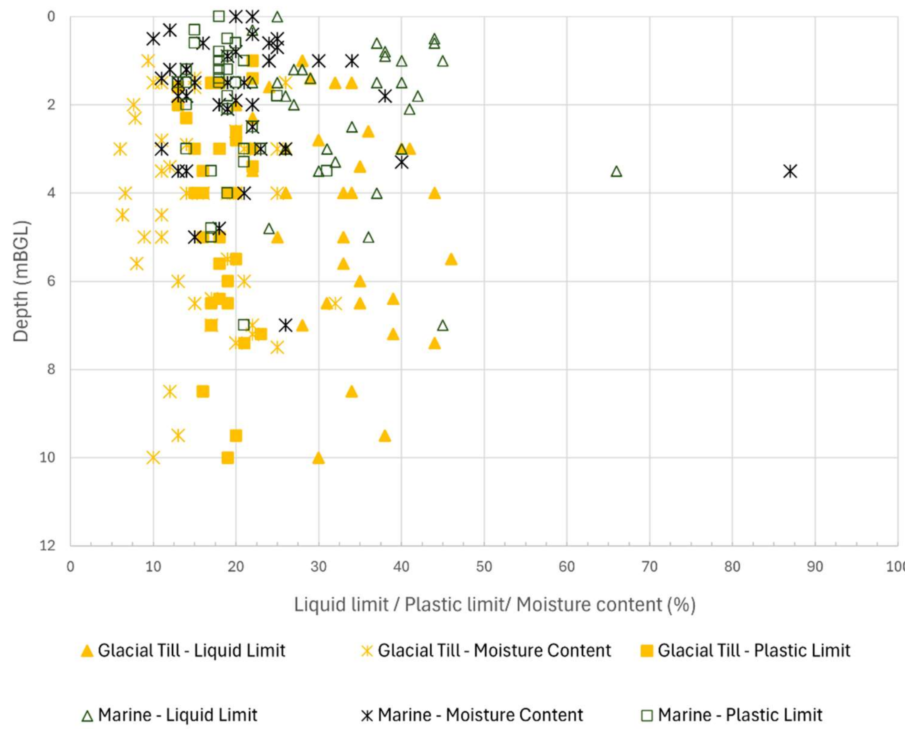


Figure 5-19: Atterberg limit results vs depth (liquid limit, plastic limit and moisture content).

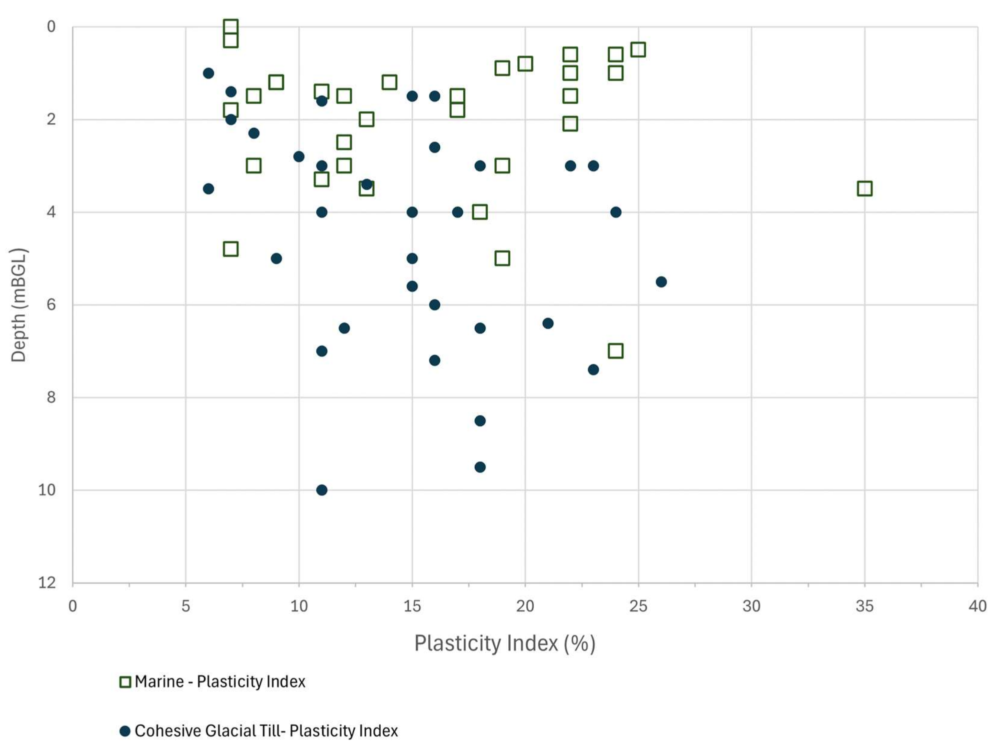


Figure 5-20: Plasticity index results vs depth (m bgl).

The moisture content results in the cohesive marine material vary between 10% and 85% with no clear trend in the results. Moisture content results in the glacial till vary between 6% and 32%, with no clear trend observed. In addition to moisture content testing, moisture condition value (MCV) tests were carried out on 3 samples- 1 within the cohesive marine stratum, and 2 within the cohesive glacial till. The MCV values are shown in Figure 5-21.

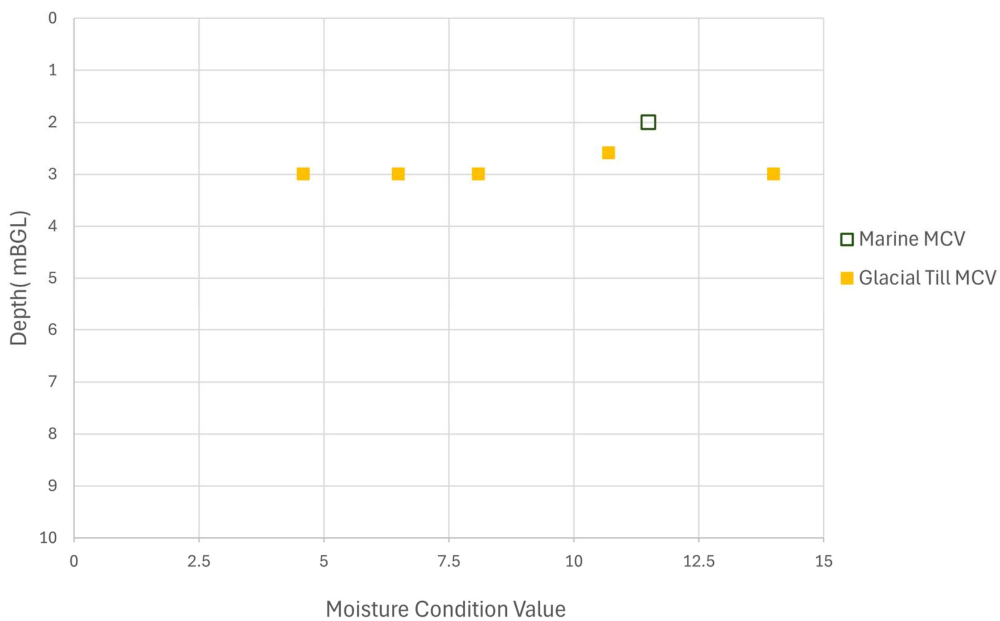


Figure 5-21: MCV vs depth.

A characteristic plasticity index (Ip) for the cohesive Marine material of 16% is suggested, and 14.7% for the cohesive Glacial Till.

The Casagrande (1948) A-line plasticity chart is shown in Figure 5-22. The combination of LL and PI results in both of the cohesive marine material and the cohesive glacial till vary between low plasticity CLAY and intermediate plasticity CLAY material.

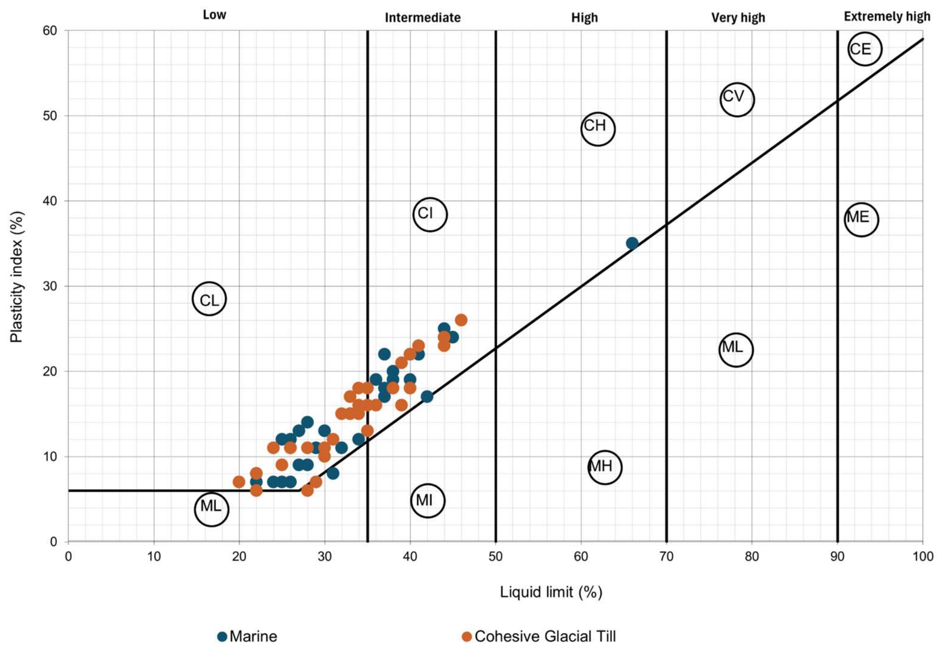


Figure 5-22: Plasticity index vs liquid limit, after Casagrande (1948): C = clay. M = silt. L = low plasticity. I = intermediate plasticity. H = high plasticity

5.7.2 Particle Size Distribution

Particle size distribution (PSD) testing was completed on 2 samples taken from the made ground stratum, 2 taken from the granular glacial till stratum, 56 taken from the cohesive marine deposits stratum, and 44 taken from the cohesive glacial till stratum. The results of the PSD testing are illustrated in Figure 5-23 to Figure 5-26. A breakdown of the sample constituents is illustrated in Figure 5-27 to Figure 5-30.

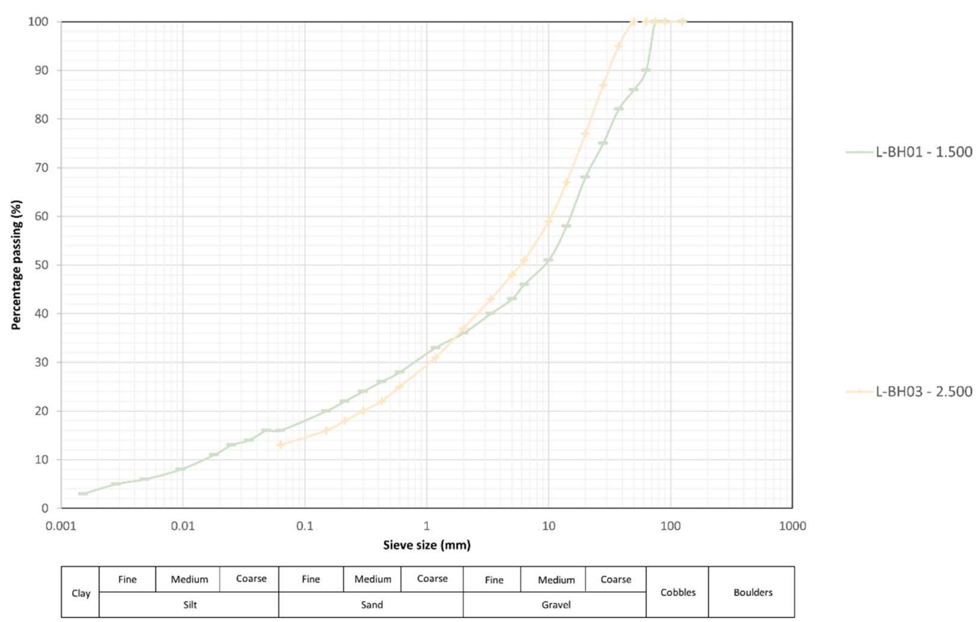


Figure 5-23: Particle size distribution (PSD) results in the made ground stratum.

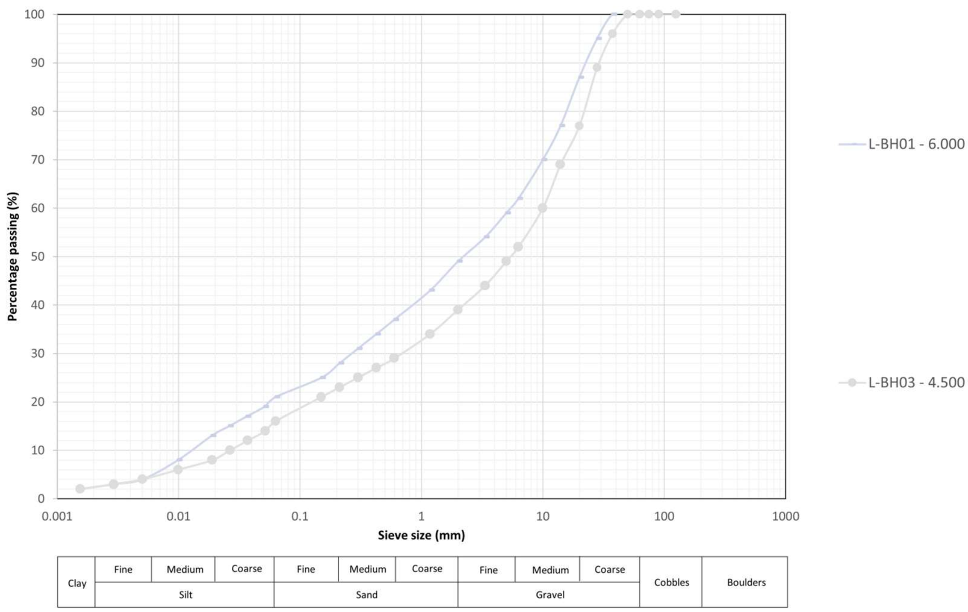


Figure 5-24: Particle size distribution (PSD) results in the granular glacial till stratum.

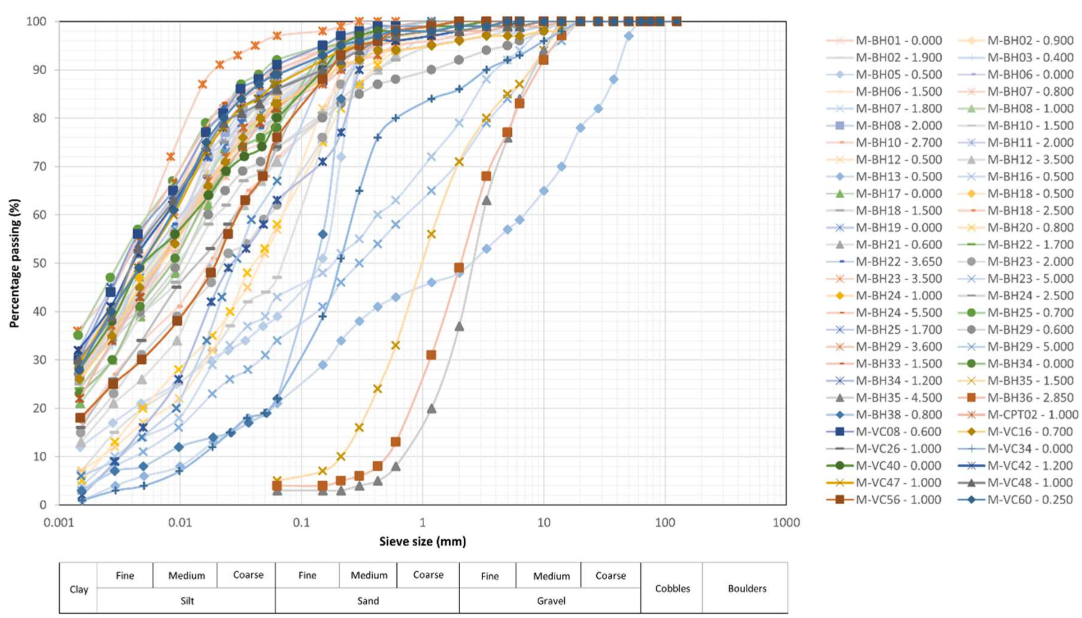


Figure 5-25: Particle size distribution (PSD) results in the cohesive marine stratum.

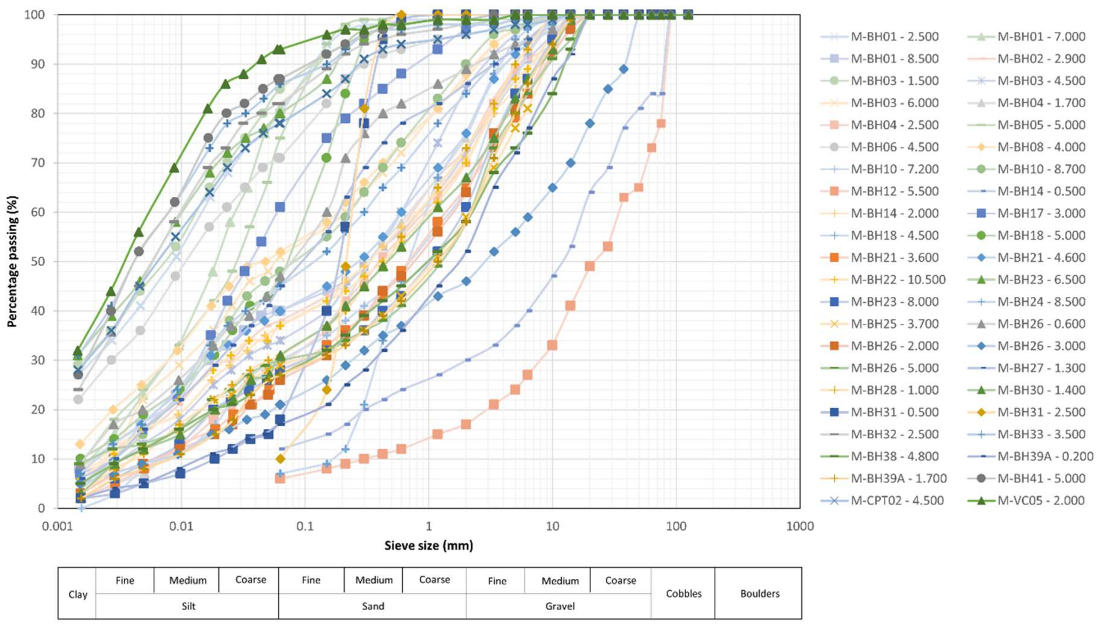


Figure 5-26: Particle size distribution (PSD) results in the cohesive glacial till stratum.

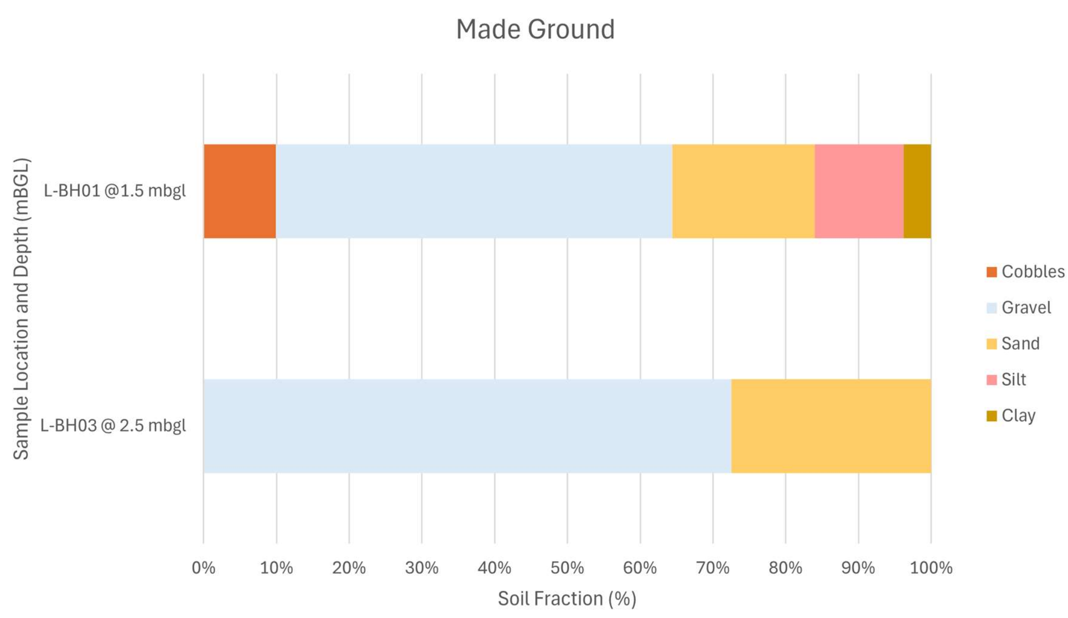


Figure 5-27: Soil fraction results for the Made Ground.

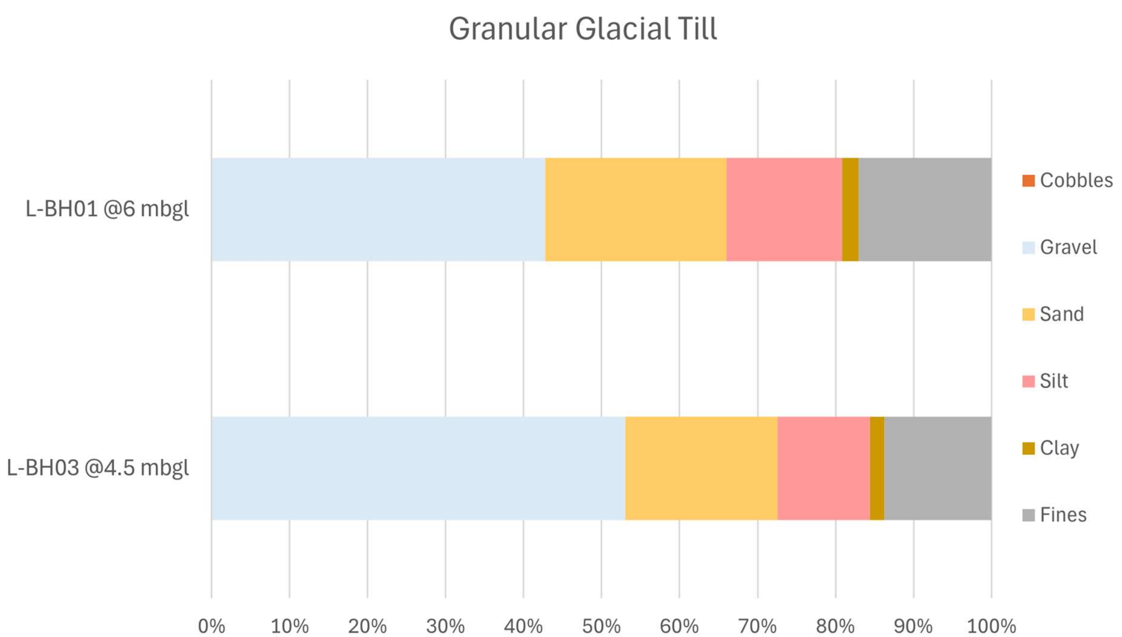


Figure 5-28: Soil fraction results for the granular glacial till.

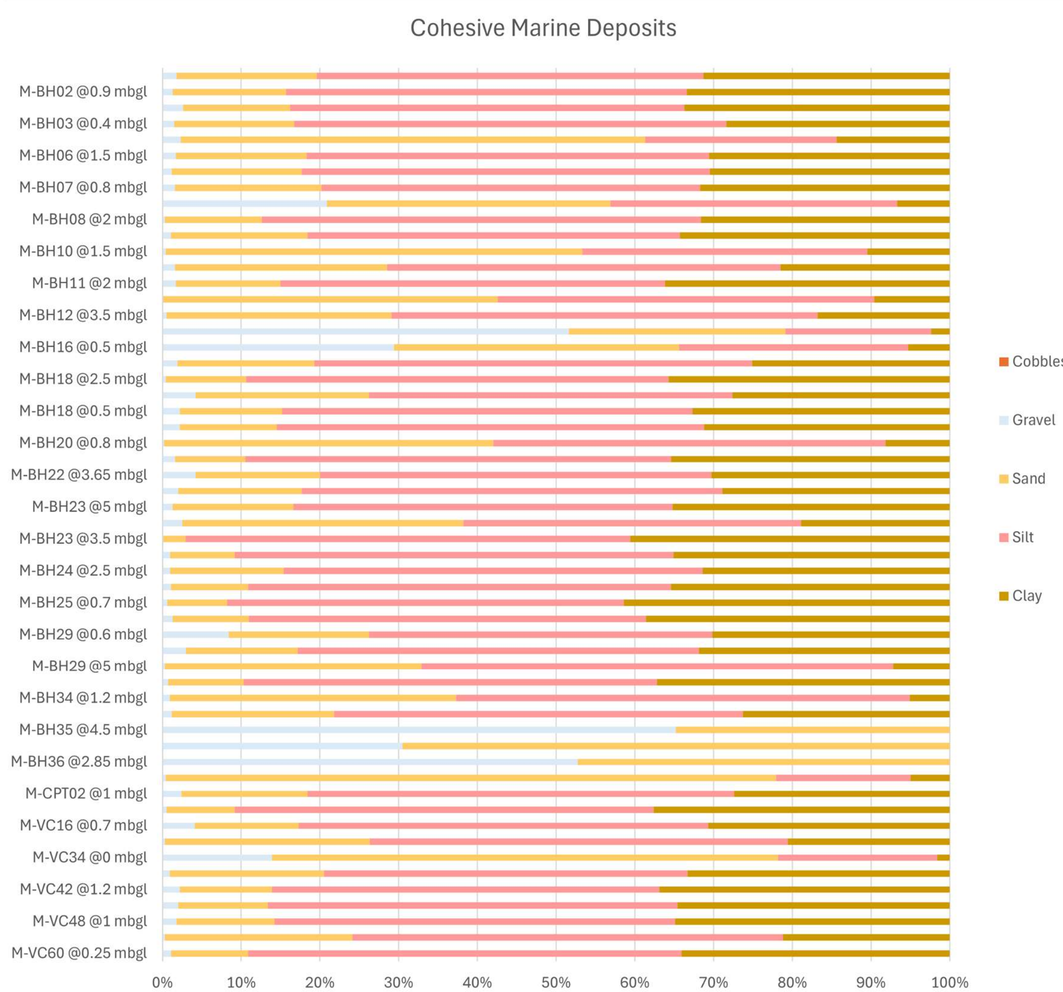


Figure 5-29: Soil fraction results for the cohesive marine deposits.

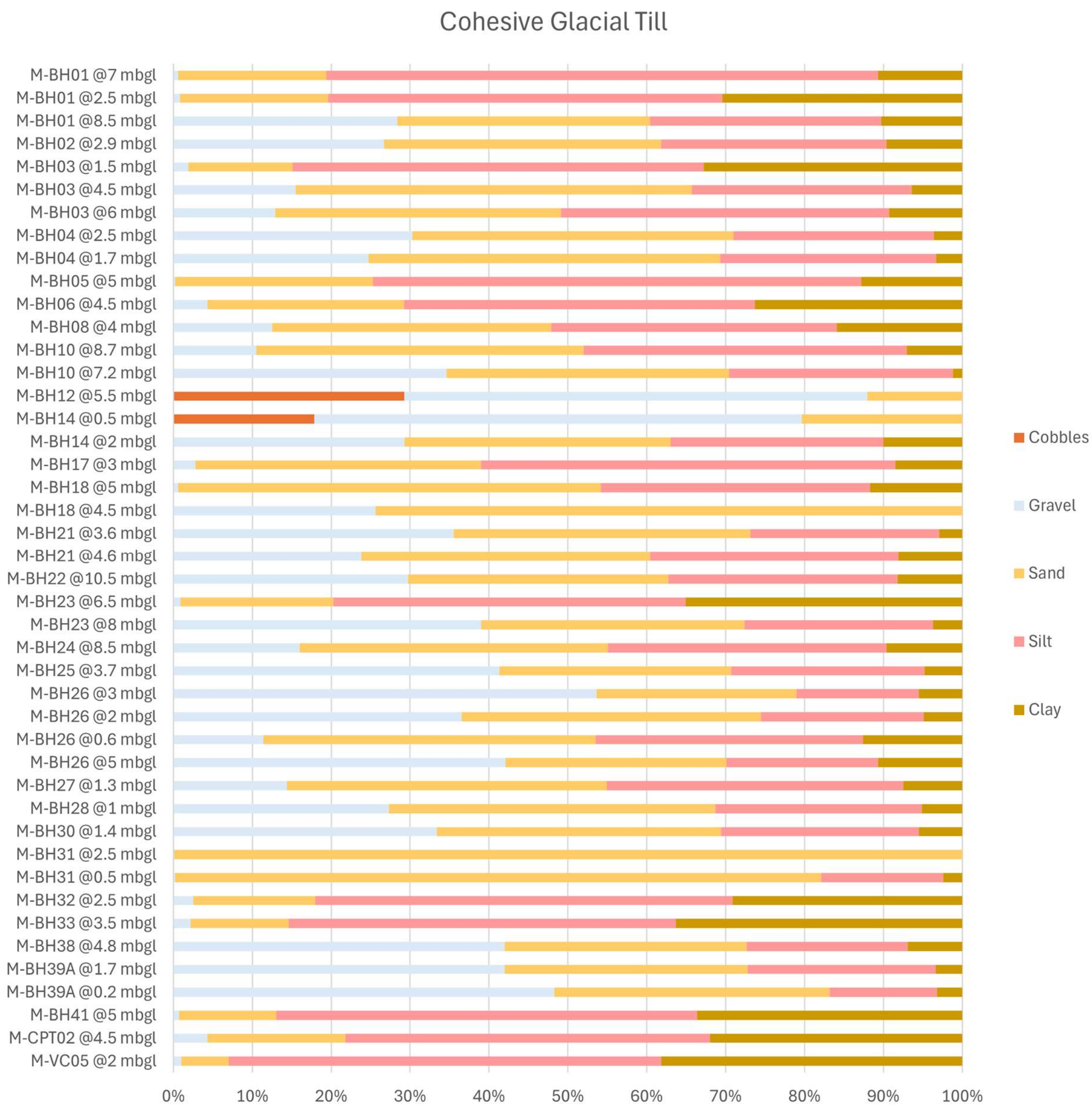


Figure 5-30: Soil fraction results for the cohesive glacial till.

The PSD tests completed on the made ground suggest that this material consists of 0-10% cobbles, 55-63% gravel, 20-24% sand, 0-12% silt, and 0-4% clay. The made ground can therefore be described as a slightly silty very sandy GRAVEL.

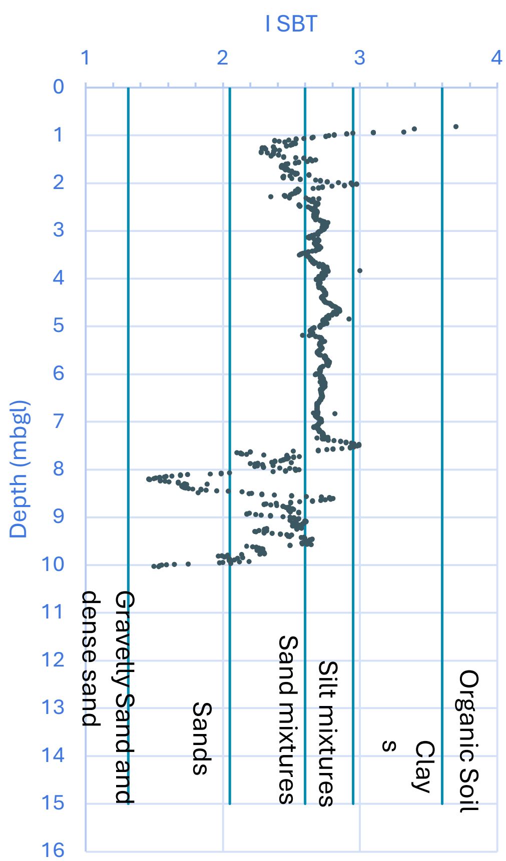
The PSD tests completed on the granular glacial till suggest that this material consists of 0% cobbles, 52-62% gravel, 23-28% sand, 14-18% silt, and 2-3% clay. The granular glacial till can therefore be described as a silty very sandy GRAVEL.

The cohesive marine deposits consist of 0% cobbles, 0-63% gravel (average of 6%), 3-78% sand (average of 23%), 0-60% silt (average of 46%), and 0-41% clay (average of 25%). The Atterberg limit testing outlined in Section 5.7.1 indicate that the soil behaviour is dominated by the CLAY fraction. The cohesive marine deposits can therefore be described as a slightly sandy silty CLAY in accordance with BS 5930:2015+A1:2020.

The cohesive glacial till consists of 0-28% cobbles, 0.1-63% gravel (average of 21%), 6-90% sand (average of 34%), 0-70% silt (average of 32%), and 0-38% clay (average of 12%). The Atterberg limit testing outlined in Section 5.7.1 indicate that the soil behaviour is dominated by the CLAY fraction. The cohesive glacial till can therefore be described as a slightly gravelly slightly sandy silty CLAY in accordance with BS 5930:2015+A1:2020.

5.7.3 CPT CLASSIFICATION TESTING

Using CPT01 as an example of a CPT location with drill-out sampling and laboratory classification testing, including Particle Size Distribution tests, a comparison can be made between the ‘Soil Behaviour types’ as derived from the CPTs and the materials sampled.



Zone	Soil Behavior Type	I_c
1	Sensitive, fine grained	N/A
2	Organic soils – clay	> 3.6
3	Clays – silty clay to clay	2.95 – 3.6
4	Silt mixtures – clayey silt to silty clay	2.60 – 2.95
5	Sand mixtures – silty sand to sandy silt	2.05 – 2.6
6	Sands – clean sand to silty sand	1.31 – 2.05
7	Gravelly sand to dense sand	< 1.31
8	Very stiff sand to clayey sand*	N/A
9	Very stiff, fine grained*	N/A

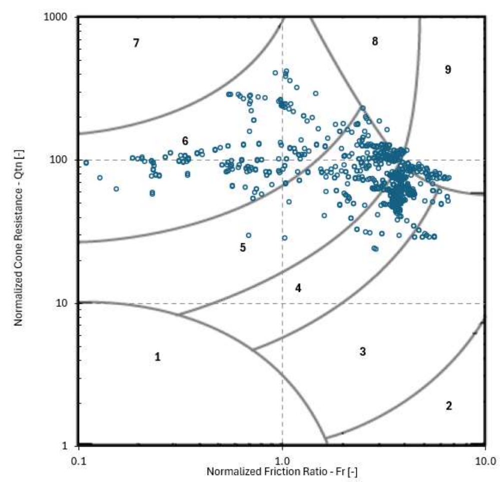


Figure 5-32: M-BH01 CPT classification information

As outlined in Figure 5-31, The CPT test undertaken at M_BH01 indicates a soil behaviour type of generally ‘silt mixtures: clayey silt to silty clays’ to a depth of approximately 7.5mbgl, where it changes to a mixture of sands and sandy mixtures. Likely indicative of the change in the strata between the marine deposits and the glacial till deposits and the CPT encountering an increase in particle size, likely sands and gravels.

Four PSD tests were carried out in M_BH01 at depths of 0.0mbgl, 2.5mbgl, 7mbgl and 8.5mbgl, the results are outlined in Figure 5-33

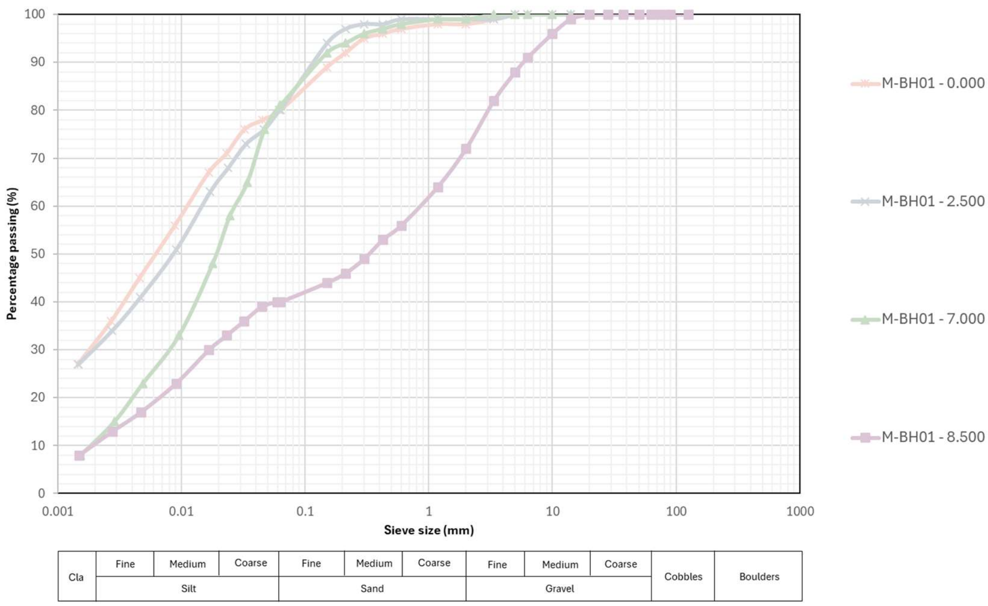


Figure 5-33: PSD results in M_BH01

The results of the PSD tests indicate the following:

- 0.0mbgl: Slightly sandy silty CLAY
- 2.5mbgl: Slightly sandy silty CLAY
- 7mbgl: Slightly sandy silty CLAY
- 8.5mbgl: sandy gravelly silty CLAY

Atterberg limit testing carried out at 4mbgl and 7mbgl indicates that the material is a CLAY.

In the hierarchy of characterisation, the PSD and Atterburg testing is considered to be a more representative indicator of the materials characteristics, rather than the derived 'soil behaviour types as indicated by the CPT information. For this reason the material composition interpreted in the ground model are predominantly taken from the sample testing and verified against the CPT information.

5.7.4 Optimum Moisture Content

Optimum moisture content (OMC) was determined on 3nr. samples using the BS1377:Part 4:1990, clause 3.5, 4.5kg rammer method. The OMC results are shown in Table 5-4. A characteristic OMC of 12% is recommended for the cohesive marine material, and 14% is recommended for the cohesive glacial till.

Table 5-4: Optimum Moisture content (OMC) results.

BH ID	Depth (m BGL)	Stratum	OMC(%)
M-BH18	3.5	Cohesive Marine Deposits	13
M-BH21	2.6	Cohesive Glacial Till	14
M-BH34	2.7	Cohesive Marine Deposits	11

5.8 SOIL AND ROCK DENSITY

The results for the bulk and dry density testing carried out on samples recovered from the granular glacial till, marine deposits, cohesive glacial till, amphibolite, mudstone, gneiss, and sandstone are shown Figure 5-34. Dry and bulk density values were captured from a combination of direct shear (shear box) testing, triaxial compression (consolidated undrained and unconsolidated undrained), 1-dimensional oedometer tests, optimum moisture content (4.5kg rammer), lab California bearing ratio (CBR) and rock density tests.

Dry density results in the cohesive marine material range between 1.00 and 2.03 Mg/m3 and bulk densities range between 1.59 and 2.43 Mg/m3, with an average bulk density of 2.03Mg/m3. The cohesive glacial till material dry density ranges from 1.54 to 1.98Mg/m3, and the bulk density ranges from 1.80 to 2.19Mg/m3, with and average bulk density of 2.06Mg/m3. One sample within the granular glacial till material was tested, recording a dry density of 2.05Mg/m3 and a bulk density of 2.25Mg/m3.

In the weathered mudstone, dry densities range between 1.80 and 2.00Mg/m3, and bulk densities range between 1.95 and 2.242.74Mg/m3.

Results from the samples recovered in the mudstone bedrock record dry densities ranging between 1.79 and 2.68 Mg/m3 and bulk densities between 2.14 and 2.71 Mg/m3, with an average bulk density of 2.27Mg/m3. The seven samples in the Amphibolite bedrock record dry densities between 2.66 and 2.85Mg/m3 and bulk densities between 2.67 and 2.87Mg/m3, while two samples taken within the Gneiss bedrock record dry densities between 2.69 and 2.70Mg/m3 and bulk densities between 2.70 and 2.71Mg/m3. One sample was tested within the sandstone, recording a dry density of 2.71Mg/m3 and a bulk density of 2.74Mg/m3.

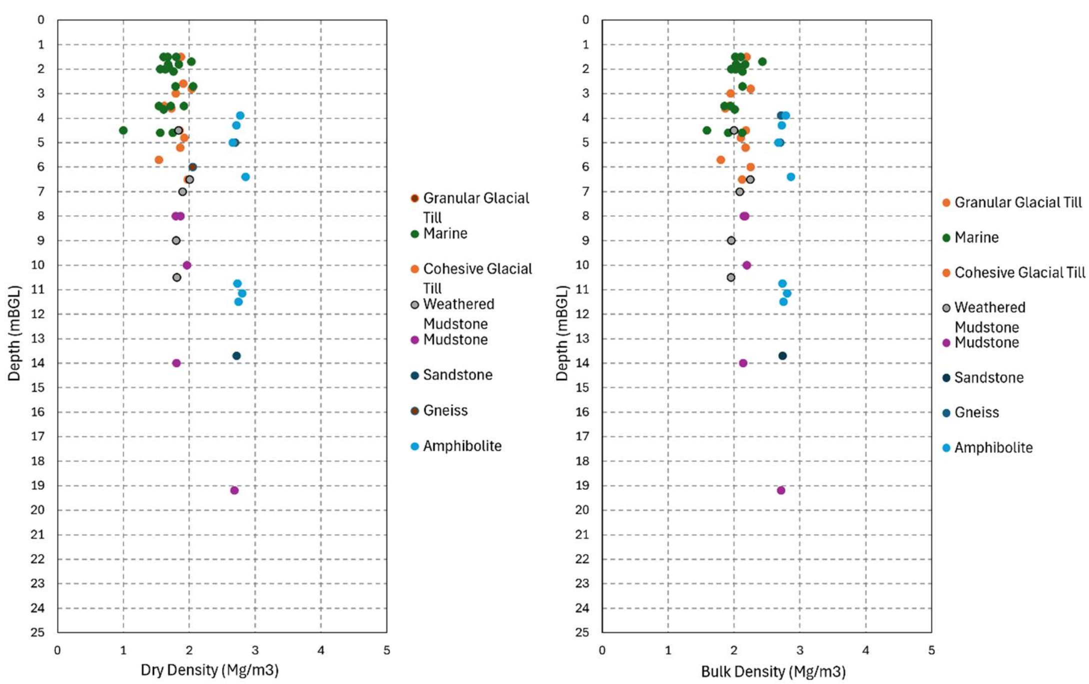


Figure 5-34: Dry and bulk density results.

5.9 CPTs

5.9.1 Unit weight

The derived unit weight of the overburden material was examined using CPT data. The calculation outlined in Equation 5-1 was used to calculate the soil unit weight:

Equation 5-1: Soil unit weight from CPT (Robertson, 2010)

$$\gamma/\gamma_w = 0.27(\log R_f) + 0.36[\log(q_t/P_a)] + 1.236$$

Where:

g = Unit Weight

g_w = Unit weight of water

R_f = Friction ratio

P_a = Atmospheric Pressure

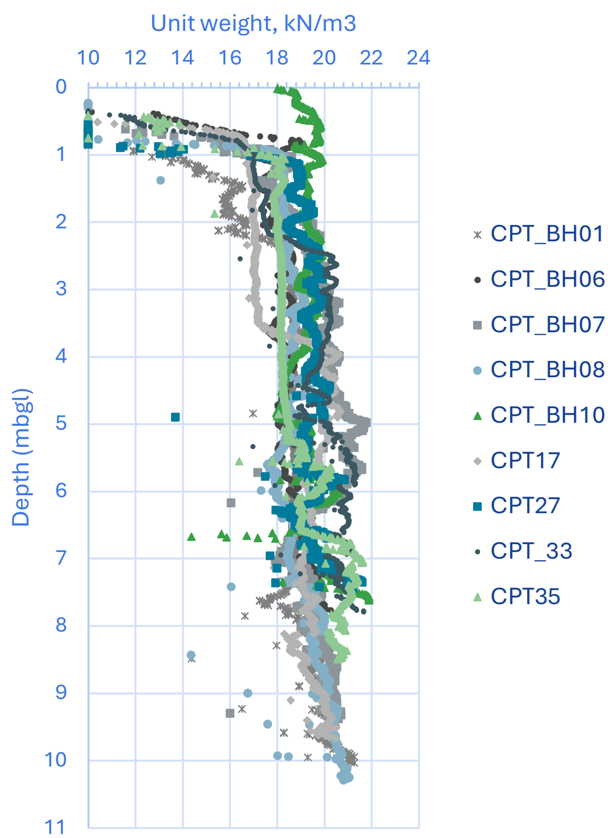


Figure 5-35: Unit weight by CPT

The results indicate the material is variable with unit weight between 18 kN/m^3 and 20 kN/m^3. The material is relatively consistent, with some lower results at shallow depths, likely in soft marine sediments, seafloor organic materials, or as a result of soil disturbance from the casing flush at the beginning of the test.

5.9.2 UNDRAINED SHEAR STRENGTH

The undrained shear strength of the cohesive materials was calculated using the method outlined in Equation 5-2 (Roberson, 2010).

Equation 5-2: Undrained shear strength by CPT (Robertson, 2010)

$$c_u = \frac{q_t - \sigma_v}{N_{kt}}$$

Where:

Q_t = Cone resistance

$_{sv}$ = Insitu effective stress

N_{kt} = Factor related to eh material consolidation ratio. A N_{kt} value of 15 was assumed for this assessment, this is commonly used in over consolidated cohesive materials.

The undrained shear strength values derived for cohesive materials in CPT01, CPT06, CPT07, CPT17, CPT27, CPT33 and CPT35 are outlined in Figure 5-36. Where cohesive materials were not encountered or the CPT soil behaviour type results are indicative of a granular material the undrained shear strength value was not calculated, for example in all of CPT06.

The results are indicative of high strength silts and clay layers within granular strata. The results suggest an undrained shear strength value for the marine and glacial till materials of between 75 and 375kPa. However, the presence of the granular layers or more coarse material within the silts and clay would have influenced these results possible increasing the derived results due to increase cone resistance results and creating some of the spikes and peak results we see in Figure 5-36. A more realistic result range for the CPT undrain shear strength value would be in the range of 60 to 120kPa.

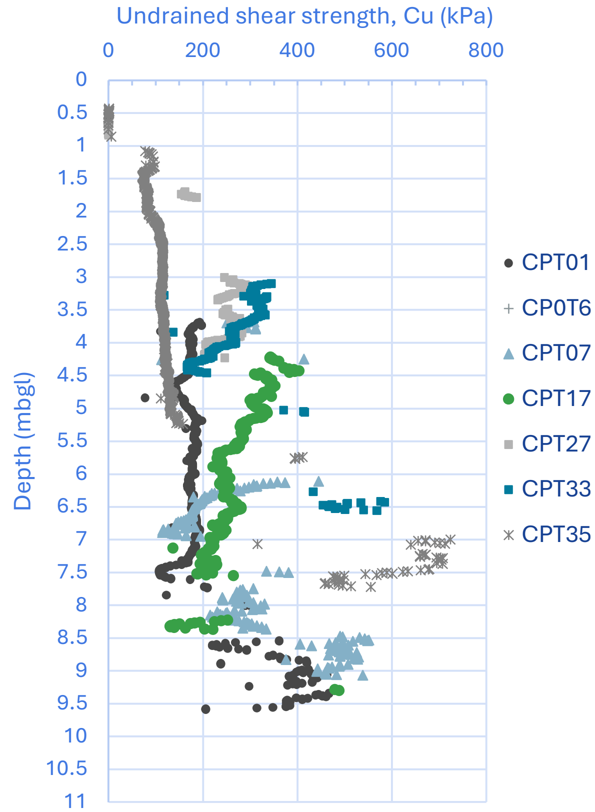


Figure 5-36: Undrained shear strength by CPT

5.9.3 EFFECTIVE ANGLE OF FRICTION

The effective angle of friction of the materials encountered in the CPTs is calculated using Bolton (1986).

Equation 5-3: Effective angle of friction by CPT (Bolton, 1986)

$$\phi = \phi_c + \psi(p, e)$$

Where:

Φ = Peak Effective angle of friction

Φ_c = critical state angle of friction

Ψ = Dilatancy term dependant on void ratio, e and effective mean pressure, p.

The derived results are outlined in Figure 5-37. The results are highly variable and are suggestive of an effective friction angle in the outlined test materials ranging between 32 and 45 degrees with the majority of the results between 37 and 45 degrees.

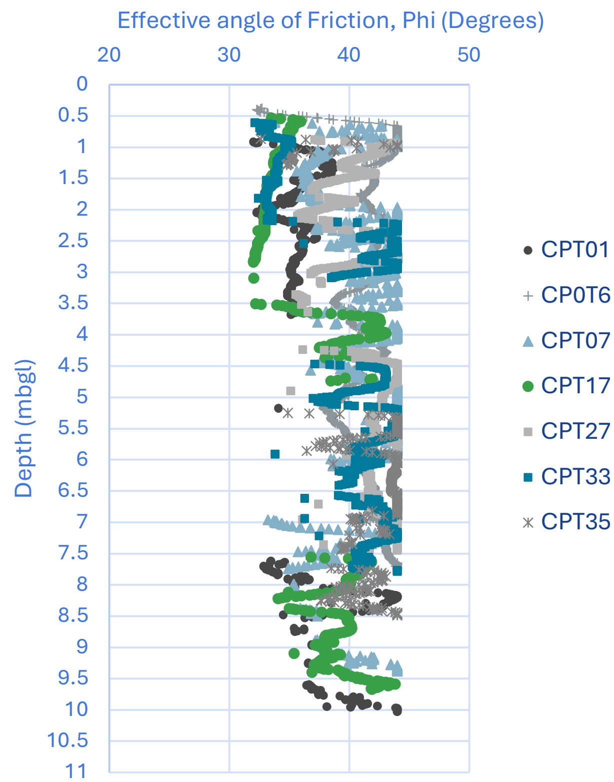


Figure 5-37: Effective friction angle by CPT (Bolton, 1986)

5.9.4 COMPRESSIBILITY CHARACTERISTICS

The compressibility characteristics such as the constraint modulus (M) and coefficient of consolidation(c_h) are derived from the dissipation test results carried out during the CPT tests. A total of 40nr. dissipation tests were carried out by the Causeway (2024) ground investigation. The summary of the parameters collected during these test is outlined in Table 5-5.

Parameters derived from dissipation testing include:

- 1-D Constraint modulus (M),
- Coefficient of consolidation in the horizontal direction (c_h),
- Soil permeability (K_h)
- Volume Compressibility Coefficient, Mv

Table 5-5: Summary of dissipation test results

Location ID	Depth (mbgl)	C_h (m^2/s)	C_h (m^2/year)	M (MPa)	M_v	K_h (m/s)
M-CPT01_BH	7.91	1.21E-05	382	39.28	0.025458	3.10E-09
M-CPT02	3.41	8.13E-07	26	19.65	0.050891	4.15E-10
M-CPT03_BH	4.58	9.13E-07	29	53.24	0.018783	1.72E-10
M-CPT04_BH	1.54	2.00E-05	629	70.78	0.014128	2.83E-09
M-CPT05_BH	3.25	1.53E-05	481	23.59	0.042391	6.49E-09
M-CPT05_BH	5.47	4.46E-05	1405	71.36	0.014013	6.26E-09
M-CPT07_BH	1.65	2.02E-06	64	20.17	0.049579	1.00E-09
M-CPT07_BH	4.56	7.56E-08	2	67.97	0.014712	1.12E-11
M-CPT08	3.35	1.23E-06	39	19.19	0.05211	6.43E-10
M-CPT10_BH	3.13	3.68E-05	1161	35.6	0.02809	1.04E-08
M-CPT11	3.24	6.28E-07	20	55.2	0.018116	1.14E-10
M-CPT12	2.06	2.20E-06	69	44.66	0.022391	4.94E-10
M-CPT17	4.1	2.41E-05	759	63.36	0.015783	3.81E-09
M-CPT17	5.1	6.05E-07	19	39.64	0.025227	1.53E-10
M-CPT18	3.69	9.90E-07	31	6.13	0.163132	1.62E-09
M-CPT19	3.43	1.44E-06	46	57.13	0.017504	2.53E-10
M-CPT19	5.51	7.88E-07	25	157.38	0.006354	5.02E-11
M-CPT20	2.79	4.80E-06	151	84.81	0.011791	5.68E-10
M-CPT20	4.46	4.09E-09	0	60.58	0.016507	6.76E-13
M-CPT22	1.11	1.41E-06	45	42.64	0.023452	3.32E-10
M-CPT23_BH	6.18	3.36E-06	106	19.25	0.051948	1.75E-09
M-CPT23_BH	6.42	1.06E-05	336	25.64	0.039002	4.16E-09
M-CPT23_BH	7.49	6.81E-05	2147	16.22	0.061652	4.21E-08
M-CPT24_BH	3.47	2.48E-07	8	22.39	0.044663	1.11E-10
M-CPT27	1.73	1.45E-05	458	19.41	0.05152	7.51E-09
M-CPT27	3.07	4.37E-06	138	31.35	0.031898	1.40E-09
M-CPT27	7.37	2.02E-06	64	207.63	0.004816	9.77E-11
M-CPT33_BH	1.53	2.93E-07	9	11.75	0.085106	2.50E-10
M-CPT33_BH	3.27	6.01E-07	19	37.21	0.026874	1.62E-10
M-CPT35	1.29	2.47E-06	78	11.44	0.087413	2.16E-09
M-CPT35	7.75	5.52E-06	174	92.83	0.010772	5.96E-10
M-CPT37	2.19	6.62E-04	20891	24.35	0.041068	2.73E-07
M-CPT37	4.46	9.04E-06	285	40.31	0.024808	2.25E-09
M-CPT37	6.72	1.46E-05	461	154.91	0.006455	9.47E-10
M-CPT38_BH	4.62	0.00E+00	0	87.48	0.011431	-1.00E+04
M-CPT39_BH	0.86	6.07E-05	1914	59.48	0.016812	1.02E-08
M-CPT41_BH	1.53	0.00E+00	0	33.48	0.029869	-1.00E+04
M-CPT41_BH	5.7	1.03E-06	33	26.47	0.037779	3.92E-10
M-CPT41_BH	7.55	1.30E-06	41	27.75	0.036036	4.71E-10
M-CPT41_BH	9.78	6.18E-06	195	53.33	0.018751	1.16E-09

5.10 Soil Chemistry

Limited geochemical testing was undertaken, involving the collection of seven samples from four boreholes. The sampling depth ranged between 0.6m – 8m below ground level. The sample from M-BH26, at a depth of 2.6m bgl in a Glacial Till layer, measured the water-soluble (S04) concentration as 69.07mg/l. The pH value and Chloride at this depth were determined to be 8.7 and 982 mg/kg.

Organic matter testing was performed on samples from three boreholes, with sampling depths ranging between 0.6m – 8 m below ground level. The measured organic matter content ranged from <0.1 % to 0.95 % with no clear trend with depth or borehole location.

The aggressivity of the soil to the concrete structure was assessed following the guidance of the BRE guidance for concrete in aggressive soil (BRE Special Digest 1:2005). Based on the natural ground conditions and measured values of water-soluble sulphate and Ph values for mobile water regime, the Design Sulphate Class is considered as DS-1 and the ACEC class as – AC- 1[d].

Table 5-6: Summary of Geochemical Testing

Strata	Location ID	Depth (mbgl)	Chemical Name	Result Value
Cohesive Glacial Till	M-BH26	2.6	Sulphate Aqueous Extract as SO4 (2:1)	69.07 mg/l
Cohesive Glacial Till	M-BH26	2.6	Chloride	982.04 mg/kg
Marine	M-BH05	2.5	Organic matter	0.56 %
Marine	M-BH26	0.6	Organic matter	0.06 %
Cohesive Glacial Till	M-BH23	7.4	Organic matter	0.95 %
Cohesive Glacial Till	M-BH23	8	Organic matter	0.5%
Cohesive Glacial Till	M-BH26	2.6	pH	8.71 Ph units

5.11 Rock Quality

Causeway encountered bedrock in 46 exploratory hole locations, including all of the sonic and rotary cored boreholes with the exception of M-BH31. Rock cores were recovered from 39 of the boreholes, with recovered core ranging from highly weathered angular GRAVELS to strong, competent bedrock cores. Plots of the total core recovery (TCR), solid core recovery (SCR) and the rock quality designation (RQD) are shown in Figure 5-38

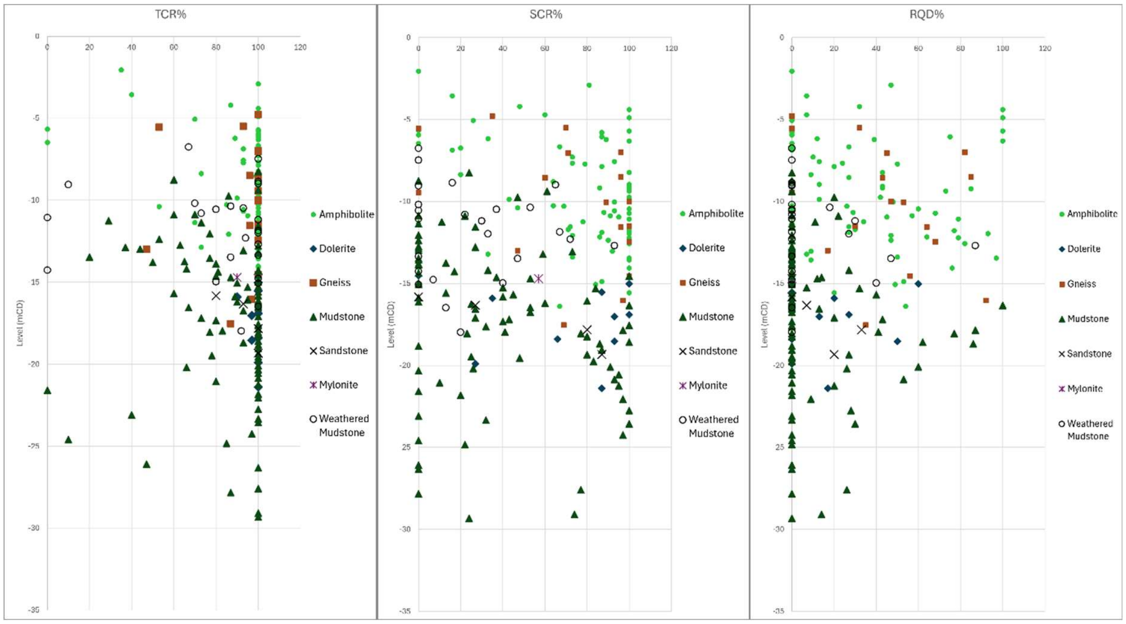


Figure 5-38: TCR, SCR and RQD profiles with elevation (mCD).

Total Core Recovery (TCR) at the site was generally high for the rock encountered presents the TCR results showing TCR ranging from 0% to 100% for all of the cores, with an average TCR of 88%. Solid Core Recovery (SCR) results show the values vary between 0% and 100% with an average of 52.7% recorded. Rock Quality Designation (RQD) results values are generally between 0% to 100% with an average of 22.8% recorded.

Core was generally highly fractured, particularly in the mudstone, but also in the metamorphic strata in some places. Mudstone core recovery was often very poor and became extremely friable when recovering from the core liner. This led to difficulties in obtaining samples for point load and uniaxial compression testing. The high degree of discontinuities and poor core recovery in the mudstone, and in parts of the metamorphic strata may be linked to the high degree of faulting indicated in the GSI mapping, and to the presence of mylonites and possible fault gouge material encountered in some boreholes. The mudstone appears to be very finely laminated in places, contributing to the highly friable nature of the core samples.

5.11.1 Rock Strength

Causeway completed standardised unconfined compressive strength (UCS) tests and point load tests ($I_{s(50)}$) on rock samples recovered from a wide set of the rotary core boreholes completed across the site. UCS tests were completed on 20 samples and point load tests were completed on 124 samples. The UCS and corrected point load test values (UCS=16.3 *$I_{s(50)}$) are shown in Figure 5-42. The majority of point load tests were carried out as axial tests (114nr.), with a small number carried out on irregular lumps (3nr.) and 1 diametral test carried out. Additional UCS and point load tests have been scheduled on samples taken from within the metamorphic strata. This report will be updated to reflect the results of this testing.

5.11.1.1 Uniaxial Compressive Strength

The 20nr UCS test results are shown in Figure 5-39, and are highly variable, particularly within the amphibolite strata. This may be due to a number of factors, particularly the high degree of discontinuities noted in some locations. It is also noted that the UCS values may be influenced by anisotropy within the core samples, with banding and foliation noted within the rock core potentially influencing the variability of the results.

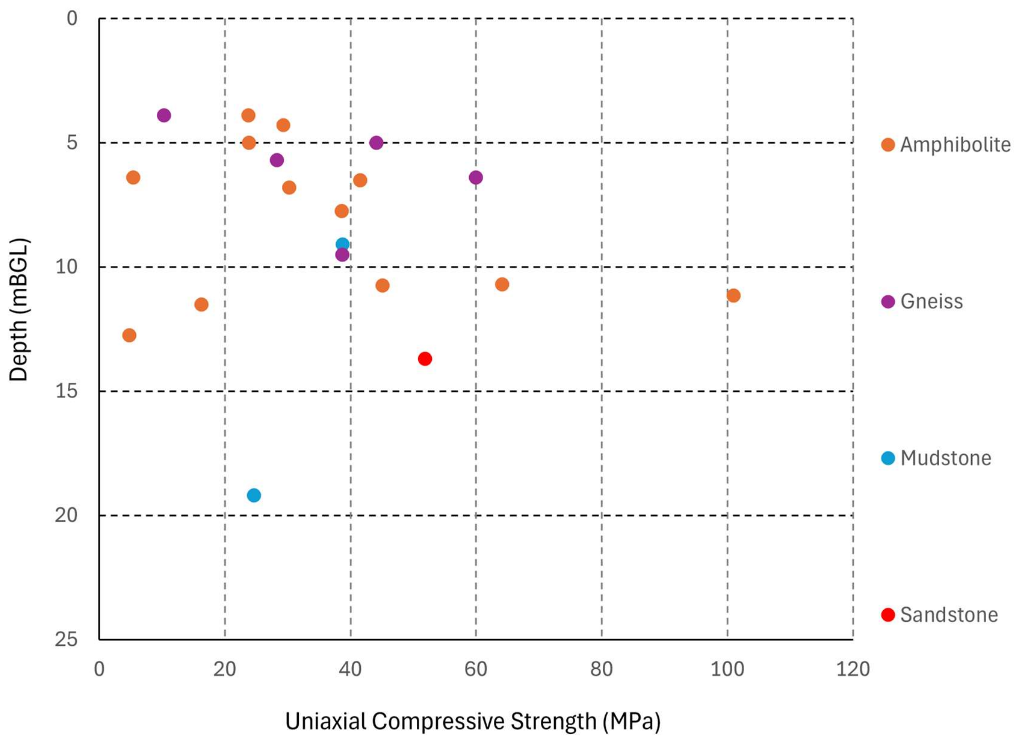


Figure 5-39: Uniaxial Compressive Strength test results vs depth.

5.11.1.2 POINT LOAD TESTS

The 124nr. point load tests determined $I_{s(50)}$ values ranging between 0 and 8.5MPa. A number of the point load tests carried out on core samples taken within the mudstone reported low Is(50) values, typically around 0.1MPa in the upper 7-10m in which they were encountered. This is an indication of the weak and fractured nature of the weathered mudstone bedrock.

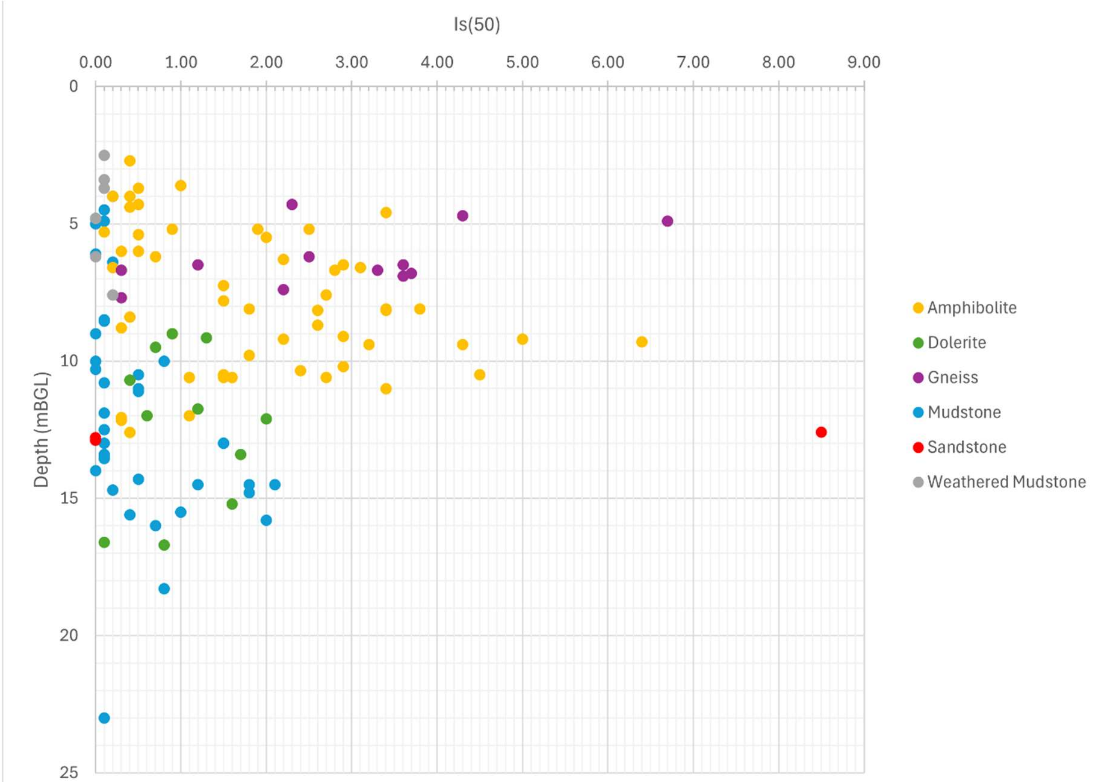


Figure 5-40: Average Point Load Index, $I_{s(50)}$ values vs depth.

5.11.1.3 Schmidt Hammer Tests

Two Schmidt hammer tests were carried out on samples taken from within the mudstone strata. The lab noted that the core samples were highly fractured when preparing them for testing. Both samples reported Schmidt Rebound Hardness values of <10N/mm^2. These values have been correlated with UCS values using the correlation proposed by Kidybinski (1980), from Goudie (2006):

$$UCS = 0.447e(0.045(N+3.5)+ \gamma)$$

Where N= Schmidt Rebound Hardness, and γ= bulk density.

5.11.1.4 Fracture Spacing Index

The fracture spacing index identified on core samples retrieved from the rotary boreholes is shown in Figure 5-41. This illustrates the wide variety of fracture spacings identified. It might be expected that fracture spacing would increase with depth if the high fracture spacing was primarily weathering-related, however no depth relationship is identified. This suggests that the wide variation in fracture spacing, and very high fracture spacing in some units is a depositional and structural feature. Fractures are extremely closely spaced throughout the mudstone unit.

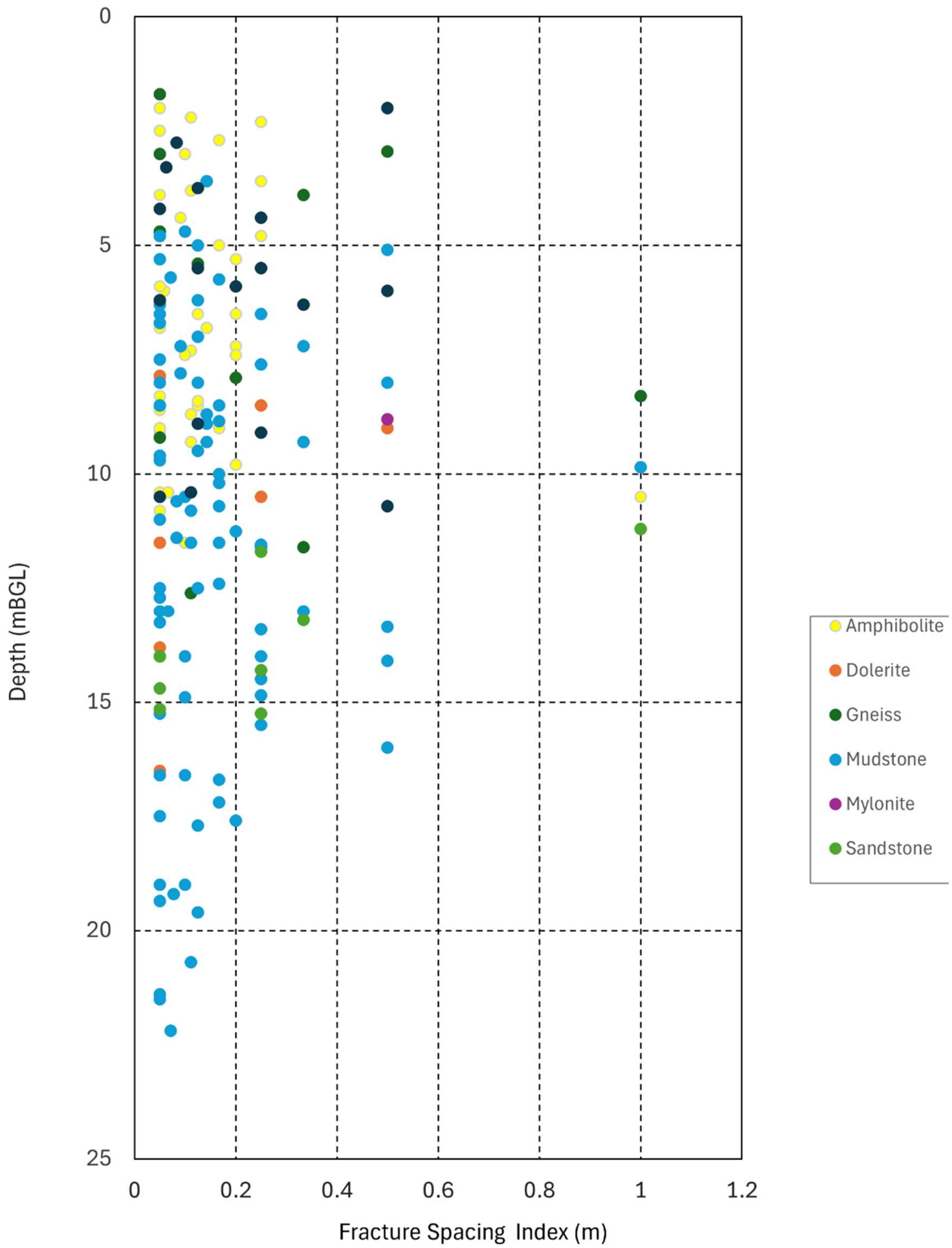


Figure 5-41: Fracture spacing index vs depth.

5.11.1.5 CHARACTERISTIC PARAMETERS

The point load test $I_{s(50)}$ values can be converted to equivalent UCS values using the formula UCS = $K.I_{s(50)}$. UCS and point load tests collected at similar locations are used to derive the K factor. An average UCS/ $I_{s(50)}$ ratio of 16.3 has been applied, based on the UCS and point load data available.

The results are shown in combination with the results of the UCS testing in Figure 5-42, and are highly variable. The mudstone remains consistently below 10MPa until a depth of 12-14m BGL, below which the UCS appear to gradually increase. The areas of bedrock sampled have likely been weathered and some samples may have been affected by sample disturbance and drying out during the coring, sampling and testing process. These values below 10MPa are derived from the Point Load Test $I_{s(50)}$ values. These are considered somewhat less reliable than the UCS testing.

A characteristic UCS value of 4MPa is recommended for the mudstone, and 18MPa for the dolerite. The amphibolite material is highly variable, ranging between 3.3MPa and 104MPa. The majority of the outlying values are derived from the Point Load Test data, therefore the characteristic USC value has been based largely on the UCS testing. A characteristic UCS range of 30-50MPa for the amphibolite is recommended. No clear trend can be established for the sandstone material, for which only two data points are available, therefore a characteristic range has not been defined. The gneiss is also variable, with a characteristic range of 35-60MPa recommended.

Table 5-7: Characteristic UCS values.

Stratum	UCS (MPa)
Weathered Mudstone	2
Mudstone	4
Amphibolite	30-50
Gneiss	35-60
Dolerite	18

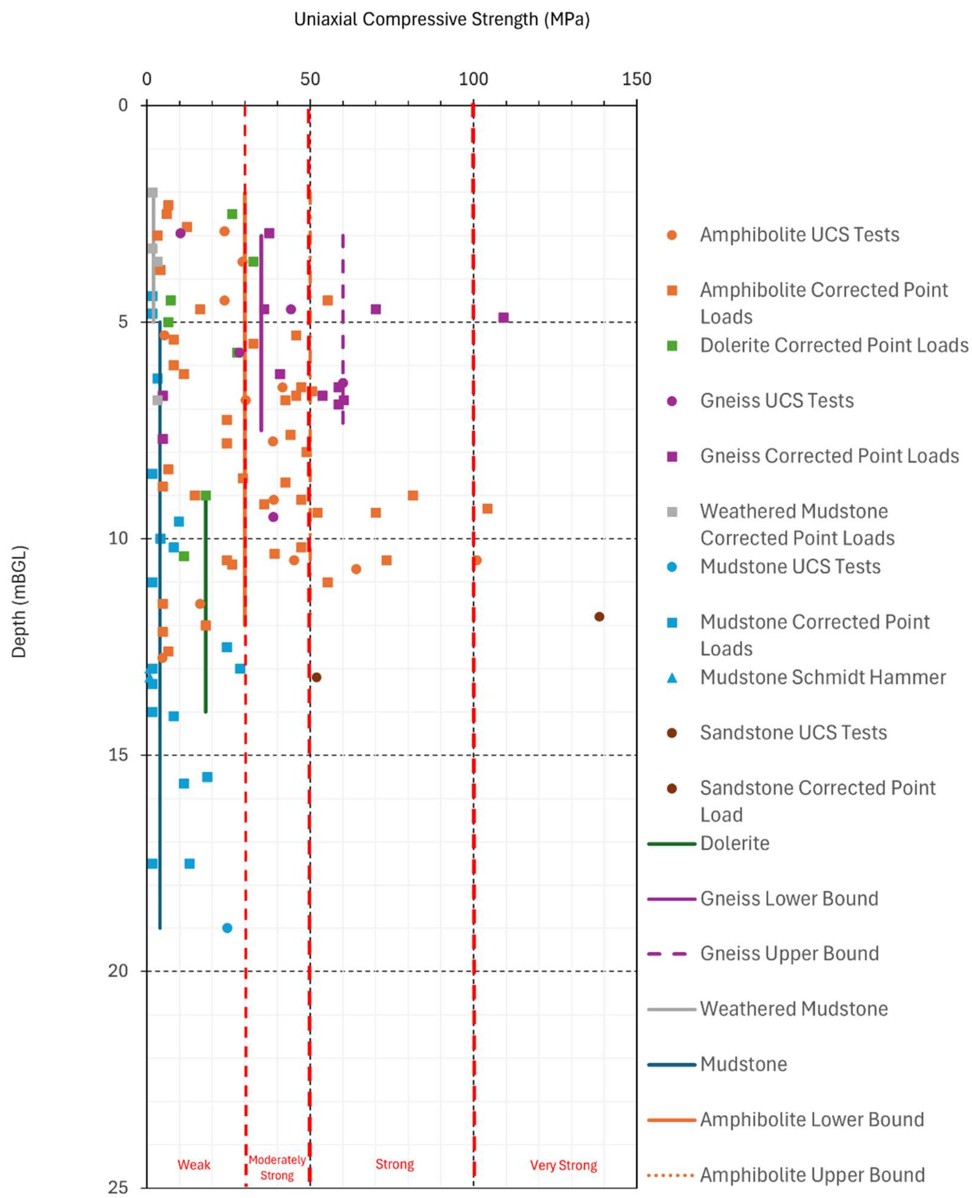


Figure 5-42: Uniaxial Compressive Strength results from UCS testing and derived from Point Load Test $I_{s(50)}$ values and Schmidt hammer results.

6 CHARACTERISTIC GEOTECHNICAL PARAMETERS

The geotechnical parameters of the overburden and rock geology units are derived from the land and marine-based boreholes based on the GI carried out by Causeway in 2024 within the proposed development area. All 2024 Causeway GI locations have been considered. The characteristic geotechnical parameters are based on the geotechnical lab test values as outlined in Section 5, along with relevant correlations or published values, as described in the following sections.

6.1 SELECTION OF CHARACTERISTIC GEOTECHNICAL PARAMETERS

6.1.1 BULK UNIT WEIGHT

Bulk unit weight has been assessed from measured values on samples collected during the Causeway (2024) SI, with bulk density values recorded from direct shear (small shear box) tests, consolidated undrained triaxial compression tests, unconsolidated undrained triaxial compression tests, 1-dimensional oedometer tests, optimum moisture content (4.5kg rammer) tests, and rock density (ISRM:2007) tests. The unit weight of the overburden materials was also derived from the CPT results and is outlined in Section 5.9.1.

The unit weight (γ) of strata have been assessed primarily from the measurements of bulk density, then were converted to unit weights, the density values were multiplied by the acceleration due to gravity, taken as 9.81 m/s2. The average bulk densities, and calculated bulk unit weight of the materials across the site is presented in Table 6-1. The calculated bulk unit weight values are presented in Figure 6-1.

Table 6-1 Summary of Bulk Density and Bulk Unit Weight.

Stratum	Bulk Density (Mg/m^3)	Calculated Bulk Unit Weight (kN/m^3)
Granular Glacial Till	2.25	22.0
Cohesive Marine Deposits	1.59-2.43	15.6-23.8
Cohesive Glacial Till	1.80-2.18	17.7-21.5
Weathered Mudstone	1.95-2.24	19.2-22.0
Bedrock - Mudstone	2.14-2.71	21.0-26.6
Bedrock - Sandstone	2.74	27.7
Bedrock - Amphibolite	2.67-2.87	26.2-28.1
Bedrock - Gneiss	2.70-2.71	26.5-26.6

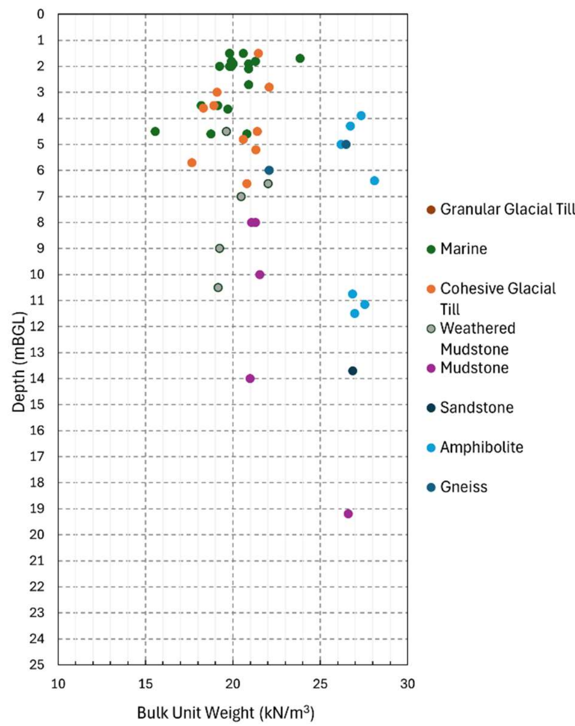


Figure 6-1: Calculated Bulk Unit Weight Values

The unit weights collected from the lab tests were used to verify against the CPT data. However, the high level of variation within the collected CPT data made the confirmation of the unit weight parameter by CPT difficult to characterise and the lab test results were used to verify this.

Bulk density was measured on one sample from within the granular glacial till, with a value of 2.25Mg/m^3 recorded. A bulk unit weight of 22kN/m^3 has been calculated. When compared to Figure 1 and Figure 2 of BS 8004:2015 (reproduced in Figure 6-2), a more conservative value of 19kN/m^3 has been selected, within the medium dense to dense GRAVEL unit weight range for material above the groundwater table.

The bulk density results recorded in the cohesive marine material range between 1.59 and 2.43 Mg/m^3, with an average bulk density of 2.03 Mg/m^3. An average bulk unit weight of 19 kN/m^3 has been calculated. When compared to the Figure 1 and Figure 2 of BS 8004:2015 (reproduced in Figure 6-2), the results suggest the cohesive marine material is within the SILT, and within the low strength to high strength CLAY unit weight range for material below the groundwater table. For design purposes, a conservative characteristic bulk density value of 18kN/m^3 is suggested.

The bulk density values recorded in the cohesive glacial till range between 1.80 and 2.18 Mg/m^3, with an average bulk density of 2.03 Mg/m^3. An average bulk unit weight of 20kN/m^3 has been calculated. When compared to the Figure 1 and Figure 2 of BS 8004:2015 (reproduced in Figure 6-2), the results suggest the cohesive glacial till sits within the SILT, and within the medium strength to very high strength CLAY unit weight range for material below the groundwater table. For design purposes, a conservative characteristic bulk density value of 19kN/m^3 is suggested.

No bulk density values were recorded within the MADE GROUND, however experience with similar material on past projects, and comparison with the Figure 1 and Figure 2 of BS 8004:2015, a conservative characteristic bulk density value of 20kN/m^3 is suggested.

A characteristic bulk unit weight of 19kN/m³ has been proposed for the weathered mudstone. A characteristic bulk unit weight of 21kN/m³ has been proposed for the mudstone bedrock, and of 27kN/m³ for the sandstone and metamorphic bedrock (amphibolite and gneiss).

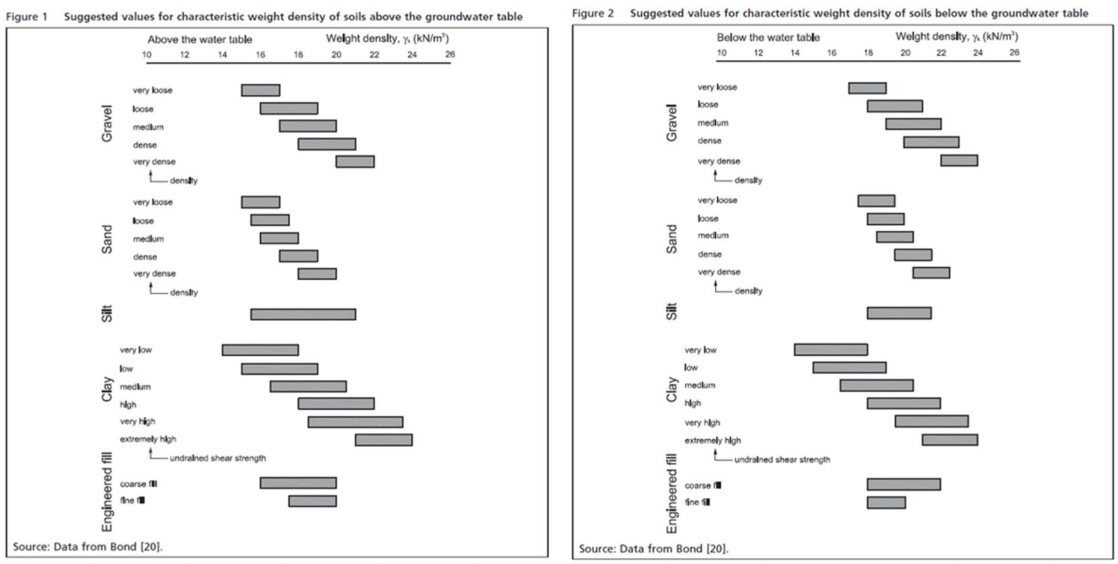


Figure 6-2: Suggested bulk density ranges as outlined in BS:8004.

Table 6-2 Summary of characteristic bulk unit weight.

Stratum	Characteristic Bulk Unit Weight (kN/m³)
Made Ground	20
Granular Glacial Till	19
Cohesive Marine Deposits	18
Cohesive Glacial Till	19
Weathered Mudstone	19
Bedrock - Mudstone	21
Bedrock - Sandstone	27
Bedrock - Amphibolite	27
Bedrock - Gneiss	27

6.1.2 UNDRAINED SHEAR STRENGTH

Undrained shear strength (c_u) of the cohesive marine deposits and of the cohesive glacial till strata has been assessed using the following datasets:

- Results of unconsolidated-undrained triaxial tests on core samples recovered during the Causeway (2024) SI,
- Laboratory shear vane testing on samples recovered during the Causeway (2024) SI,
- Correlation with Standard Penetration Test N-values collected during the Causeway (2024) SI, correlated used Stroud (1989),
- Derived from CPT data using Robertson (2010) as outlined in Section 5.9.2.

The c_u of the material has been assessed from correlations with Standard Penetration Test N-value after Stroud (1989) using the formula $c_u = f_1.N$, where f_1 is based on the accordance with the Stroud (1989) plot reproduced in Figure 6-3.
Based on the plasticity index (I_p) results outlined in Section 5.7.1, The characteristic I_p for the cohesive marine material of 16% was selected and 14.7% for cohesive glacial till material, which indicate an f_1 correlation factor of 6 for the marine stratum, and of 6.2 for the cohesive glacial till stratum in accordance with the Stroud (1989) plot reproduced in Figure 6-3.

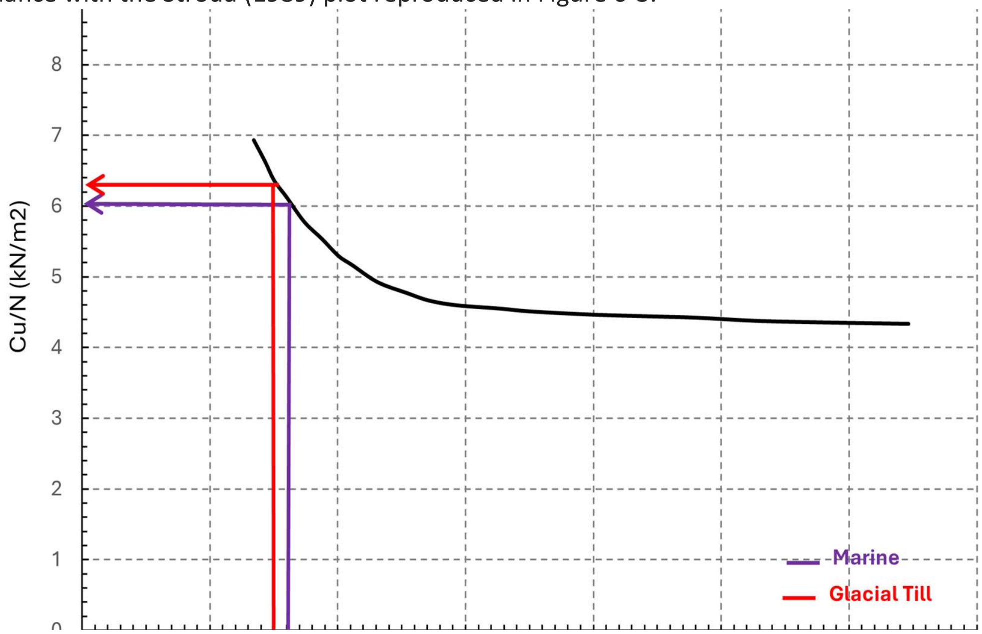


Figure 6-3: Stroud correlation between plasticity index, SPT N and undrained shear strength

The resulting values for undrained shear strength (c_u) of the glacial till and mudstone bedrock are summarised in Figure 6-4.

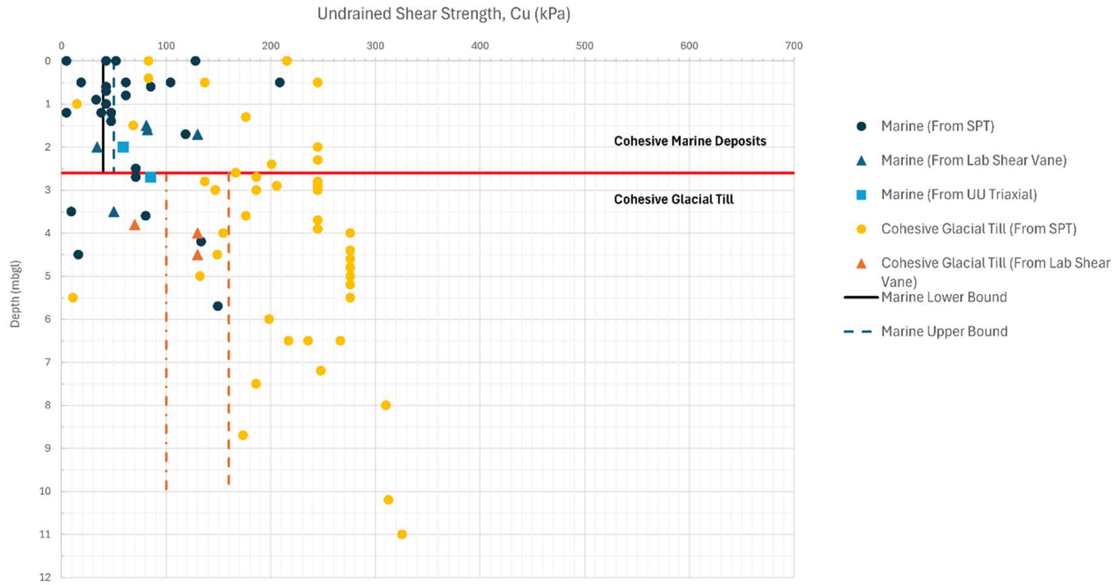


Figure 6-4: Undrained shear strength summary.

The correlated SPTs produce the minimum and maximum results for both the marine deposits and cohesive glacial till material. The lab shear vane results correlate well with the triaxial tests. The SPT derived results display a high degree of variability- generally producing lower C_u values than the lab tests in the cohesive marine deposits, and higher values in the cohesive glacial till. The outlying SPT results, particularly within the cohesive glacial till, often reflect SPT N values of >N=50, which are regarded as refusal and may not be a representation of the true ground conditions. For the purposes of this undrained shear strength assessment, SPT 'N' values have been set at a maximum value of N=50. Higher SPT 'N' values may also reflect granular material encountered within the boreholes. Based on this variability, a greater weight has been placed on the direct values derived from the lab testing for defining the characteristic C_u values.

The C_u results derived from CPT testing are outlined in Section 5.9.2. They are suggesting of undrained shear strengths between 75 and 375kPa within the cohesive marine and glacial till materials. However, it is interpreted that the granular material could have influenced these test results giving a higher than realistic strength parameter.

The recorded undrained shear strength (C_u) results in the cohesive marine material range between 4.74kPa and 208.56kPa. Based on experience with similar materials, a characteristic value range of 40-50kPa is recommended, with a characteristic value of 45kPa suggested.

The cohesive glacial till material ranges between 11kPa and 375kPa, showing a possible trend of increasing with depth. For design, a range of values should be used to carry out a sensitivity check of the proposed structure to these values. A characteristic value range for C_u of 100-160 kPa, is recommended, with a characteristic value of 135kPa suggested.

6.1.3 Effective Stress Parameters

Effective stress parameters, cohesion c' and angle of shearing resistance ϕ' have been assessed from:

- Direct shear (small shear box) tests, carried out as part of the Causeway (2024) SI
- Consolidated undrained multistage triaxials carried-out as part of the Causeway (2024) SI
- Correlations between from plasticity index after Santamarina and Díaz-Rodriguez (2003) and BS8004 (2015), based on values recorded from the Causeway (2024) SI.

- Correlations from SPT N Values after Peck et al (1974) taken from Tomlinson (2001)
- Derived from CPT data using Bolton (1986) as outlined in Section 5.9.3.

For the cohesive glacial till and mudstone materials, the characteristic critical state shear resistance ($\phi'_{cv,k}$) was estimated from the characteristic I_p derived in Section 5.7.1 using the expression proposed by Santamarina and Díaz-Rodriguez (2003) and BS8004 (2015):

$\phi'_{cv,k} = (42° - 12.5\log_{10}I_P)$ for $5\% \leq I_P \leq 100\%$

With characteristic I_p values for the cohesive marine material of 16% and 14.7% for the cohesive glacial till material, the $\phi'_{cv,k}$ for the cohesive marine material was calculated at 27°, and cohesive glacial till at 27.4°. The above formula is for the calculation of the critical state or constant volume shear resistance ($\phi'_{cv,k}$) of fine-soil only. From experience in similar materials, the peak angle of shear resistance (ϕ'_{pk}) is commonly estimated as 2° higher than $\phi'_{cv,k}$, suggestive of ϕ'_{pk} values of 29° in the cohesive marine material and 29.4° in the cohesive glacial till.

Direct shear (small shear box) tests carried out as part of the Causeway (2024) can be used to determine the angle of shearing resistance and effective cohesion of the material. The results of the direct shear tests are summarised in Table 6-3. Effective friction angle values range from 26.5° to 35° for the cohesive marine deposits, from 23° to 30.5° for the cohesive glacial till, and from 26.5° to 35.5° for the mudstone bedrock. Effective cohesion ranges from 0-5.5kPa for the cohesive marine material, 13-35kPa for the cohesive glacial till, and 1-29kPa for the mudstone bedrock.

The direct shear (small shear box test), while considered more representative than SPT 'N' correlation, has some limitations. Remoulding of samples into the small shear box required a reduction in the larger/ coarse particles from the sample. In the case of the mudstone, testing was possibly carried out on more clay-like samples, or on samples which had become friable during removal from the core liner. Large shear box tests were not available as adequate samples could not be collected from a rotary borehole.

Table 6-3: Direct shear (small shear box) test results.

BH ID	Depth (mbgl)	Stratum	Effective cohesion (c') (kPa)	Effective friction angle (ϕ') (°)
L-BH01	6	Granular Glacial Till	21	37
M-BH12	4.5	Cohesive Marine Deposits	3	31.5
M-BH22	3.65	Cohesive Marine Deposits	5.5	26.5
M-BH35	3.5	Cohesive Marine Deposits	0	35
M-BH07	4.8	Cohesive Glacial Till	35	28
M-BH10	5.7	Cohesive Glacial Till	13	23
M-BH14	1.5	Cohesive Glacial Till	21	28
M-BH17	3	Cohesive Glacial Till	22	28.5
M-BH21	3.6	Cohesive Glacial Till	17	34
M-BH25	5.2	Cohesive Glacial Till	26	26.5
M-BH29	6.5	Cohesive Glacial Till	34	27.5
M-BH31	3.5	Cohesive Glacial Till	30	30.5
M-BH03	7	Bedrock - Mudstone	16	30
M-BH05	8	Bedrock - Mudstone	20	29
M-BH06	9	Bedrock - Mudstone	28	26.5
M-BH06	10.5	Bedrock - Mudstone	19	28.5
M-BH08	6.5	Bedrock - Mudstone	19	30.5
M-BH17	4.5	Bedrock - Mudstone	25	31.5
M-BH18	10	Bedrock - Mudstone	29	29.5

BH ID	Depth (mbgl)	Stratum	Effective cohesion (c') (kPa)	Effective friction angle (ϕ’) (°)
M-BH23	14	Bedrock - Mudstone	8	30.5
M-BH29	8	Bedrock - Mudstone	1	35.5

Consolidated undrained (CU) multistage triaxial tests carried out as part of the Causeway (2024) SI has been used to determine the angle of shearing resistance and effective cohesion of the cohesive marine materials. The results of the CU triaxials are summarised in Table 6-4. Effective friction angle values range from 22° to 27.7°, while effective cohesion ranges from 0-19kPa.

Table 6-4: Consolidated undrained multistage triaxial results (Causeway2024).

BH ID	Depth (mbgl)	Stratum	Effective cohesion (c') (kPa)	Effective friction angle (ϕ’) (°)
M-BH02	1.9	Cohesive Marine Deposits	0	27.7
M-BH06	1.5	Cohesive Marine Deposits	19	24.9
M-BH07	1.8	Cohesive Marine Deposits	0	26.3
M-BH18	3.5	Cohesive Marine Deposits	10	22.4
M-BH29	4.6	Cohesive Marine Deposits	11	22

In addition to these tests, the ϕ’ value for the granular deposits can be determined using the SPT correlation proposed by Peck et al (1974), taken from Tomlinson (2001) which is presented in Figure 6-5. This methodology, while useful, has limitations, arising from the variability of materials encountered.

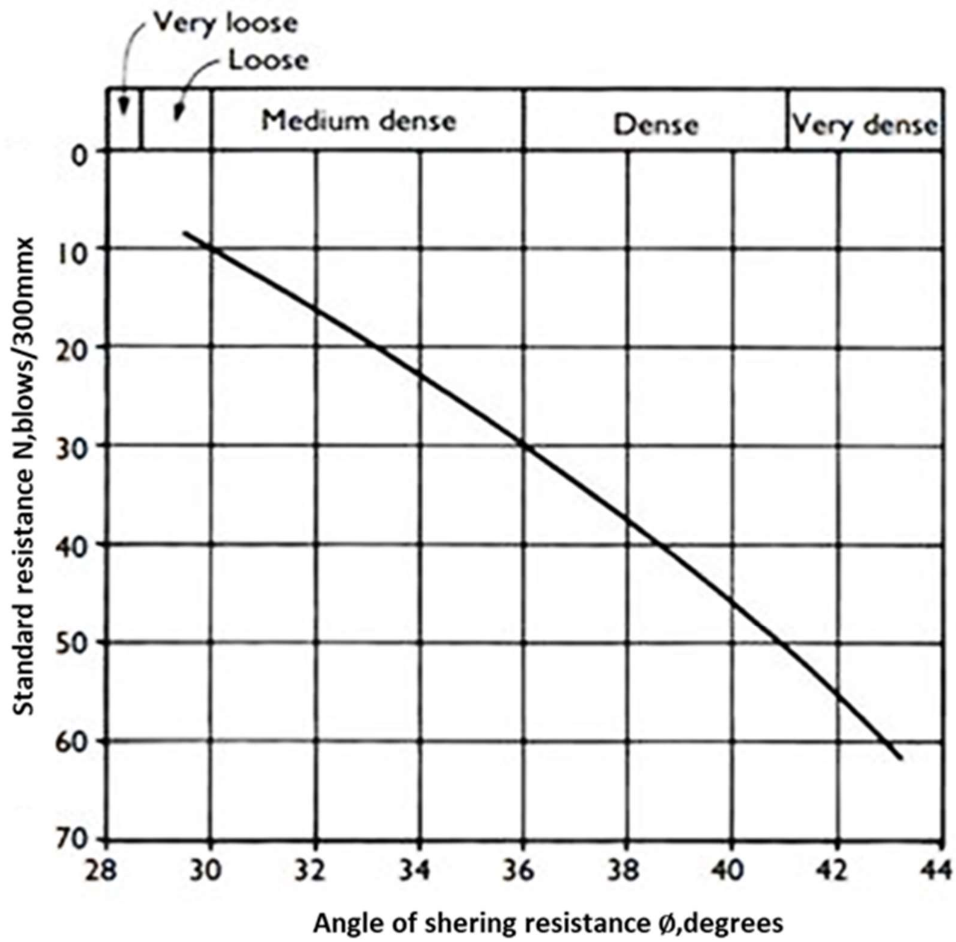


Figure 6-5: SPT vs angle of shearing resistance correlation chart for granular deposits (Peck et al., 1974)

The ϕ’ values calculated from the above testing and correlations are presented in Figure 6-6.

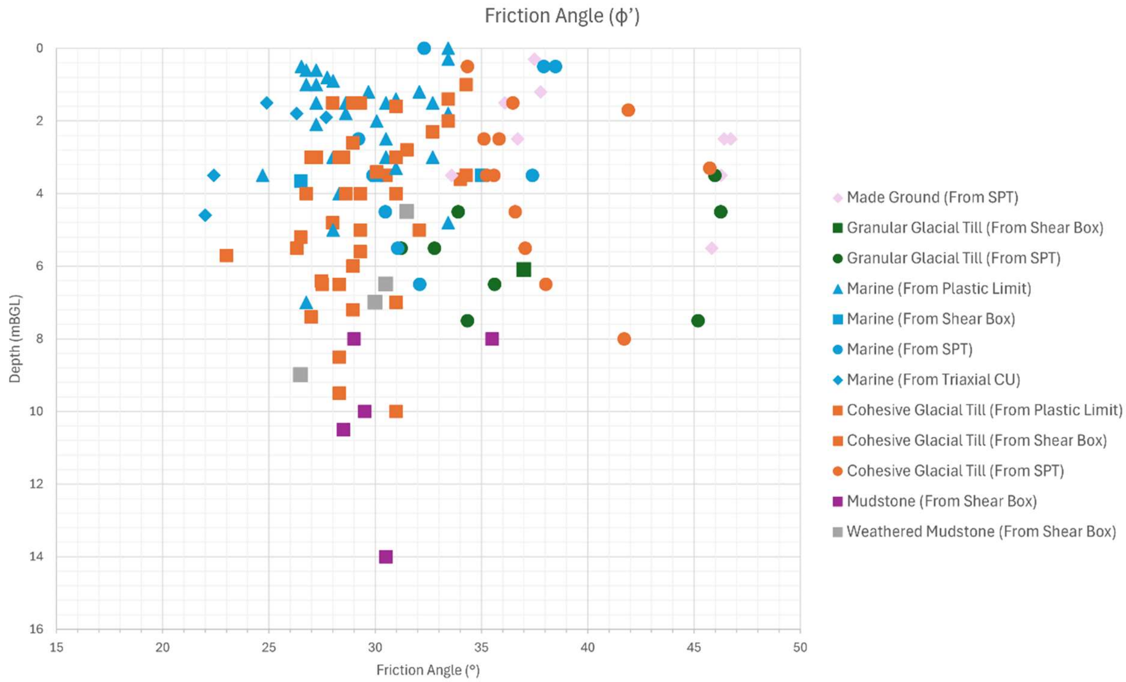


Figure 6-6: Effective friction angle vs depth, derived from SPTs, Plastic Limits, Shear Box and Consolidated undrained triaxial tests.

The derived results for the effective friction angle from the CPT locations is outlined in Section 5.9.3. These are suggestive of a variety of results between 32 and 45 degrees, with most results between 37 to 45 degrees. These results are significantly higher than those encountered in the other test methods. This may be due to the coarse material encountered in the CPT test locations influencing the total resistance of the cone.

Based on these results, and experience in similar materials, a characteristic effective friction angle, ϕ', of 31° is proposed for the made ground and granular glacial till, 27° is recommended for the cohesive marine deposits, 30° is recommended for the cohesive glacial tills, and 30 is recommended for the weathered and competent mudstone bedrock.

Based on the test results, and experience in similar materials, a characteristic effective cohesion, c', value of 0kPa for the cohesive marine deposits, 1kPa for the cohesive glacial till, 0kPa for the weathered mudstone, and 5kPa for the mudstone is recommended.

Table 6-5: Summary of effective shear stress parameters.

Stratum	Effective Friction Angle, ϕ' (°)	Effective cohesion (c') (kPa)
Made Ground	31	0
Granular Glacial Till	31	0
Cohesive Marine Deposits	27	0
Cohesive Glacial Till	30	1
Weathered Mudstone	30	0
Bedrock- Mudstone	30	5

6.1.4 COMPRESSIBILITY

The Volume Compressibility Coefficient, Mv (m^2/MN) for the material has been assessed using the following datasets:

- 1 Dimensional Oedometer Consolidation testing to BS1377: Part 5:1990c Clause 3 (10 samples).
- Correlation with Standard Penetration Test N-values collected during the Causeway (2024) SI, correlated used Stroud (1989), as shown in Figure 6-7,
- Derived from CPT dissipation testing, as outlined in Section 5.9.4.

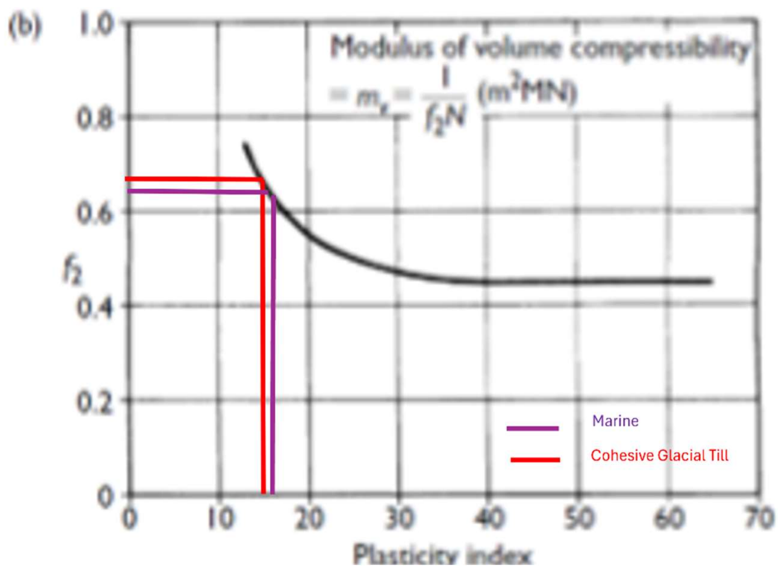


Figure 6-7: Correlation between SPT'N' and the coefficient of volume compressibility (Stroud, 1989)

Figure 6-7 illustrates which f2 factor values should be taken for each geological unit. The coefficient of volume compressibility, mv, can be calculated using the expression proposed by Stroud and Butler (1975) and Stroud (1989):

$$mv = 1/(f2*N)$$

Based on the characteristic Ip values of 16% and 14.7% for the marine and cohesive glacial till material outlined in 5.7.1, the characteristic f2 values are identified in Table 6-6:

Table 6-6: Characteristic F2 values.

Stratum	f2 Value
Cohesive Marine Deposits	0.64
Cohesive Glacial Till	0.67

The Mv results calculated from CPT dissipation testing are outlined in Section 5.9.4. These results are outlined in Figure 6-8 and Figure 6-9. The results show good correlation with the SPTs and the 1-D oedometer test results, also carried out by Causeway (2024)

The Mv values calculated from the SPT tests, the CPT dissipation test and the results from the the 1-D Oedometer tests carried out as part of the Causeway (2024) GI are illustrated in Figure 6-8 and Figure 6-9. Characteristic Mv values are outlined in Table 6-7.

Table 6-7: Characteristic Mv values.

Stratum	Mv (m^2/MN)
Cohesive Marine Deposits	0.13
Cohesive Glacial Till	0.04

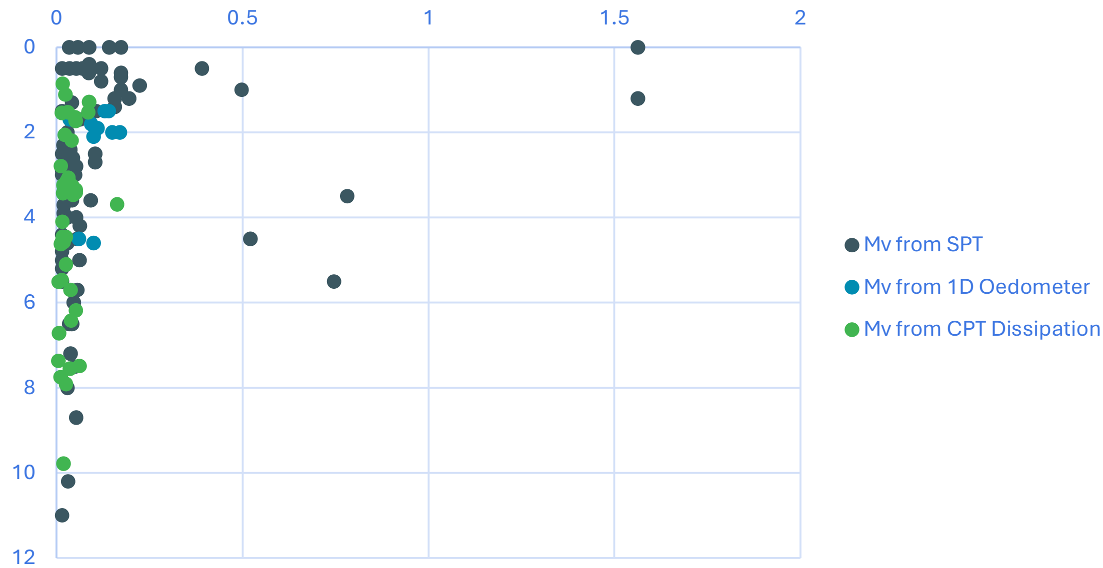


Figure 6-8: Mv vs depth (mBGL).

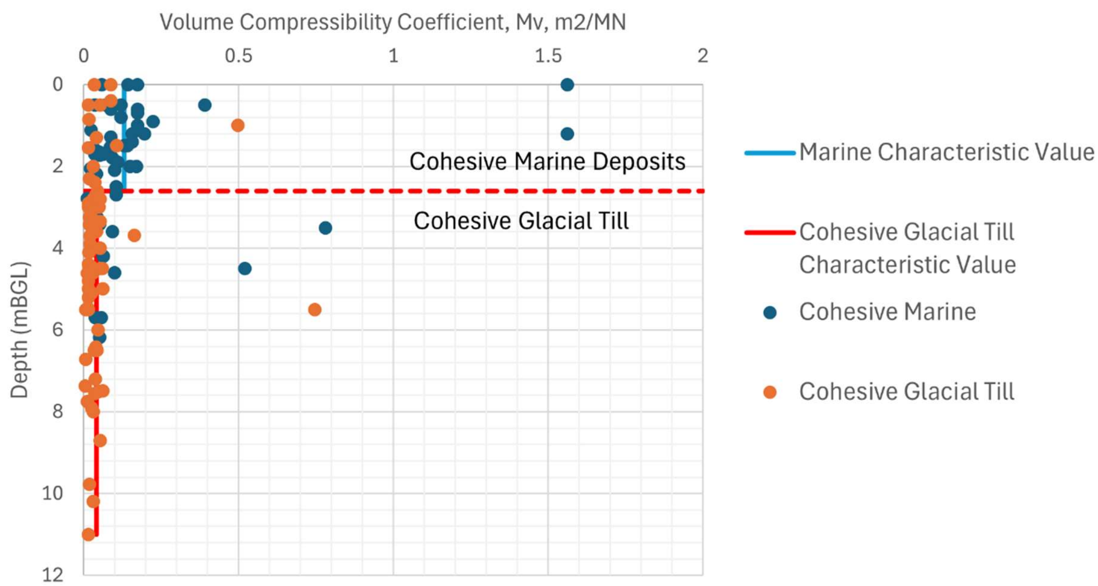


Figure 6-9: Mv vs depth indicating material type

6.1.5 Young's Modulus

6.1.5.1 Young's Modulus from SPT

A wide range of equipment is used to undertake SPT testing which influences the amount of energy transferred to the sampler with each blow of the drop hammer. The constitutive properties of a given soil deposit should not vary with the equipment used, and so *N* is conventionally corrected to a value of N_{60}, representing a standardised energy ratio of 60%. The blow count also needs to be corrected for the size of the borehole and tests done at shallow depths (<10m). These corrections are achieved using the following:

$$N_{60} = N.\zeta.(ER/60)$$

Where ζ is the correction factor for rod length (i.e. test depth) and borehole size, and *ER* is the Energy Ratio of the equipment used.

In sands and gravels, the corrected blow counts are further normalised to account for overburden pressure at the test depth. The normalised bow count ($(N_1$)60 is obtained from

$$(N_1)_{60} = C_N N_{60}$$

where C_N is the overburden correction factor and is given by:

$$C_N = \frac{A}{B + \sigma'_{v0}}$$

A and *B* vary with density, coarseness and OCR. For dense to very dense sands and gravels A=300 and B=200, otherwise, *A* and *B* are assumed equal to 200 and 100 respectively.

6.1.5.2 Undrained

The undrained Young's Modulus in the cohesive strata may be evaluated using the following relationship:

$$E_u = 1 * N_{60}(MPa)$$

A plot of the correlated undrained Young's Modulus values against depth is shown in Figure 6-10.

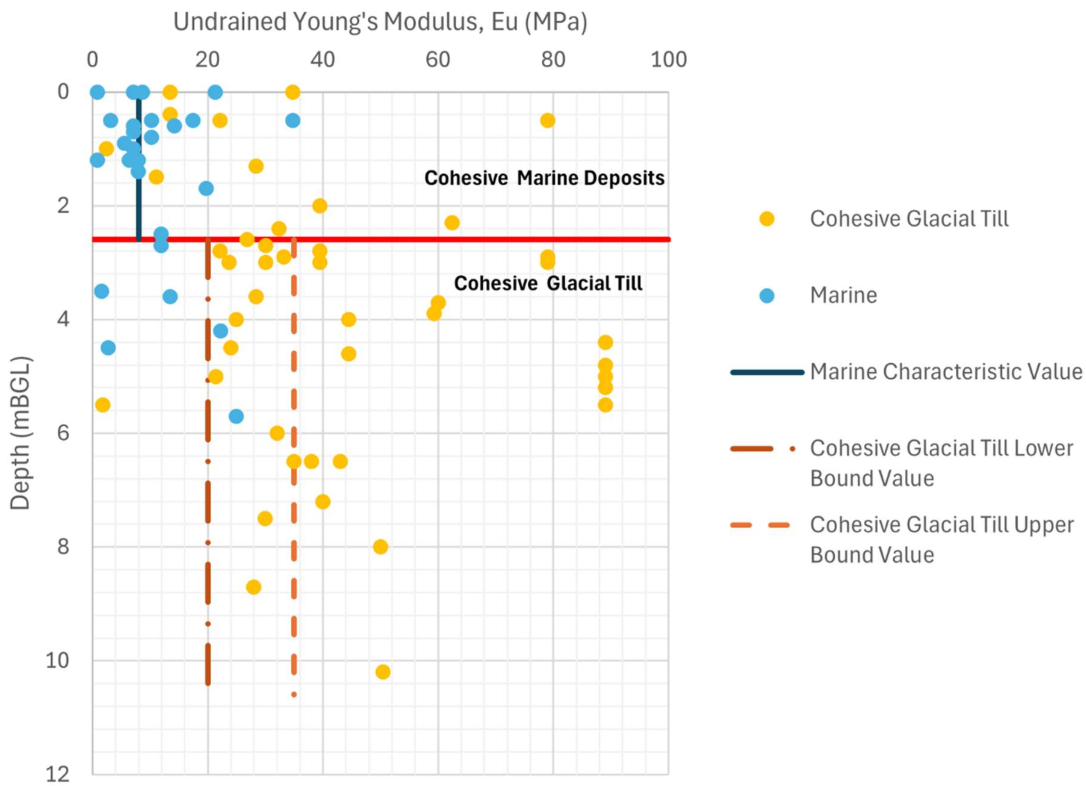


Figure 6-10: Undrained Young's Modulus values vs depth.

Based on these values, the following characteristic Undrained Young's Modulus, Eu values have been recommended in Table 6-8:

Table 6-8: Characteristic Undrained Young's Modulus, Eu values.

Stratum	Eu (MPa)
Cohesive Marine Deposits	8
Cohesive Glacial Till	20-35

6.1.5.3 DRAINED

Stroud (1989) suggested a correlation between corrected SPT N values and the drained Young's Modulus for sand and gravels:

$$E' = 2 * (N_1)_{60}\ (MPa)$$

The drained Young's Modulus in the cohesive strata may be estimated by 0.8 times the undrained Young's Modulus. A plot of the correlated drained Young's Modulus values against depth is included in Figure 6-11. Assessment of the calculated Young's Modulus based on the characteristic SPT value chosen for each stratum is presented in Table 6-9.

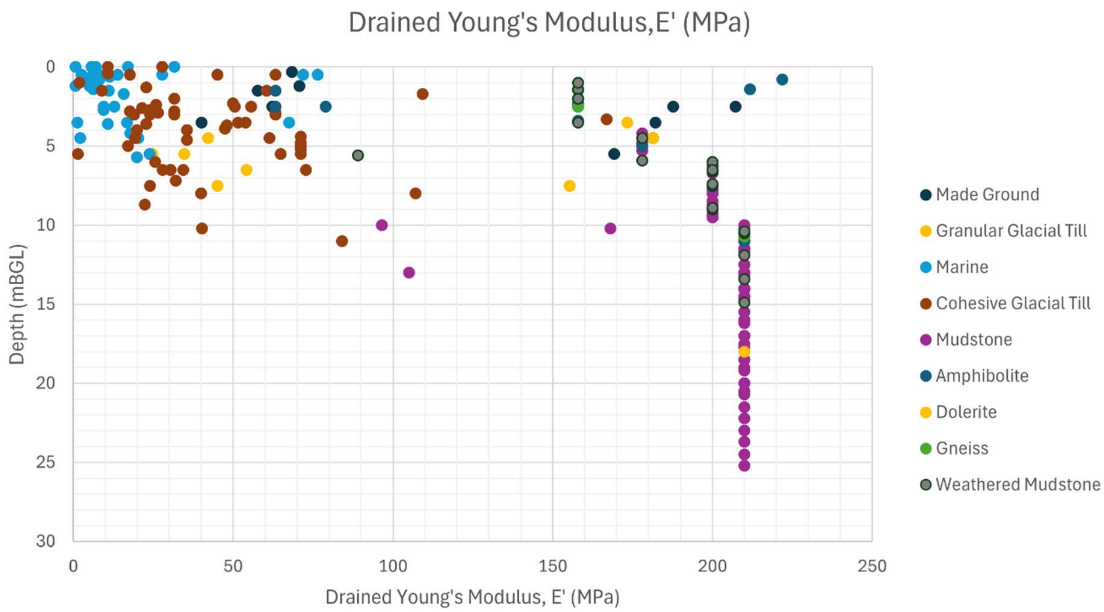


Figure 6-11: Drained Young's Modulus values vs depth.

Table 6-9: Characteristic Drained Young's Modulus, E' values.

Stratum	E' (MPa)
Made Ground	50
Granular Glacial Till	30
Cohesive Marine Deposits	5
Cohesive Glacial Till	25-30
Weathered Mudstone	80
Mudstone	100-175 (120)
Amphibolite	200
Dolerite	200
Gneiss	200

6.1.6 California Bearing Ratio

One laboratory California Bearing Ratio (CBR) test was performed, on one sample of cohesive glacial till material from M-BH07.The test was carried out on a bulk sample, which was remoulded in laboratory conditions. This test gives an indication of the suitability of the material for re-use following disturbance and recompaction. It does not give an indication of the in-situ properties of the material. A CBR of 1.8% was reported.

6.2 SUMMARY OF CHARACTERISTIC GEOTECHNICAL PARAMETERS

The characteristic soil parameters recommended for use in the geotechnical design of the various works for the proposed Rosslare ORE hub were developed based on the available GI, relevant published design standards and GDG experience with similar sites. A summary of the recommended geotechnical parameters is presented in Table 6-10 and Table 6-11. The majority of the characteristic parameters are typically based on low estimates, with a discrete number of characteristic parameters based on the best estimates (e.g., unit weight). Variations from this table may be required for other limit states, temporary works designs and constructability-related assessments (e.g., pile driving). This table may be subject to change in later revisions of the GIR should further information become available and justify such alterations.

Table 6-10 Proposed Characteristic Parameter ranges for the soil units and (recommended characteristic value).

Classification	Parameters	Made Ground	Marine	Glacial Till	
Geological unit		Sandy slightly silty gravel	Sandy silt/clay	Granular- Sandy slightly clayey gravel	Cohesive- Sandy gravelly clay
Soil Classification	Optimum Moisture Content (%)		12		14
	γ (kN/m^3)	20	18	19	20
Soil Strength	Undrained shear strength C_u(kPa)		40-50 (45)		100-160 (135)
	Effective Peak Friction Angle ϕ' (°)	31	27	31	30
	Effective Cohesion c' (kPa)	0	0	0	1
	Volume Compressibility Coefficient, Mv (m2/MN)	-	0.13	-	0.04
	Undrained Youngs Modulus Eu (MPa)	--	8	-	20-35 (30)
	Drained Youngs Modulus E’ (MPa)	50	5	30	25-30
	SPT-N		5	12	25

Table 6-11 Proposed Characteristic Parameter ranges for the bedrock units and (recommended characteristic value).

Classification		Bedrock				
Geological unit	Parameter	Weathered Mudstone	Mudstone	Amphibolite	Dolerite	Gneiss
Rock Properties	γ (kN/m^3)	19	21	27	-	27
	Effective Peak Friction Angle ϕ' (°)	30	30	-	-	-
	Effective Cohesion c' (kN/m^2)	0	5	-	-	-
	Drained Youngs Modulus E' (MPa)	80	100-170 (120)	200	200	200
	Unconfined compressive strength (MPa)	2	4	30-50 (40)	18	35-60 (45)

7 MATERIAL REUSABILITY

7.1 SOILS

The characteristic testing available for the marine soil units indicates that the reusability of materials for earthworks varies. Optimum moisture content testing (OMC, 4.5kg rammer) outlined in Section 5.7.4 allows an acceptable moisture content range to be identified for reuse as cohesive fill for both the marine and cohesive glacial till strata. This range is typically +-3% of OMC and is shown in Figure 7-1 and Figure 7-2. This indicates that the majority of samples collected in both marine and cohesive glacial till strata have moisture contents wet of optimum, with a small number of cohesive glacial till samples having moisture contents dry of optimum. No clear trend was visible with depth.

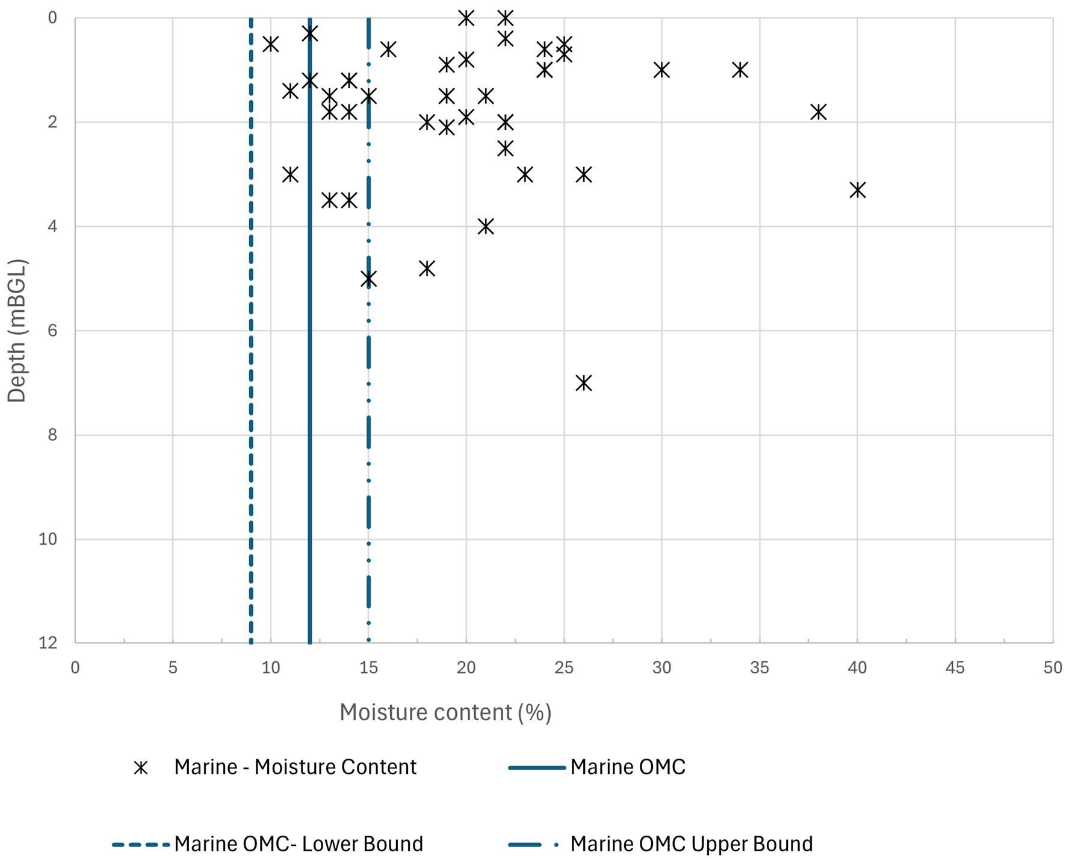


Figure 7-1: Moisture content results and optimum moisture content ranges vs depth for cohesive marine material.

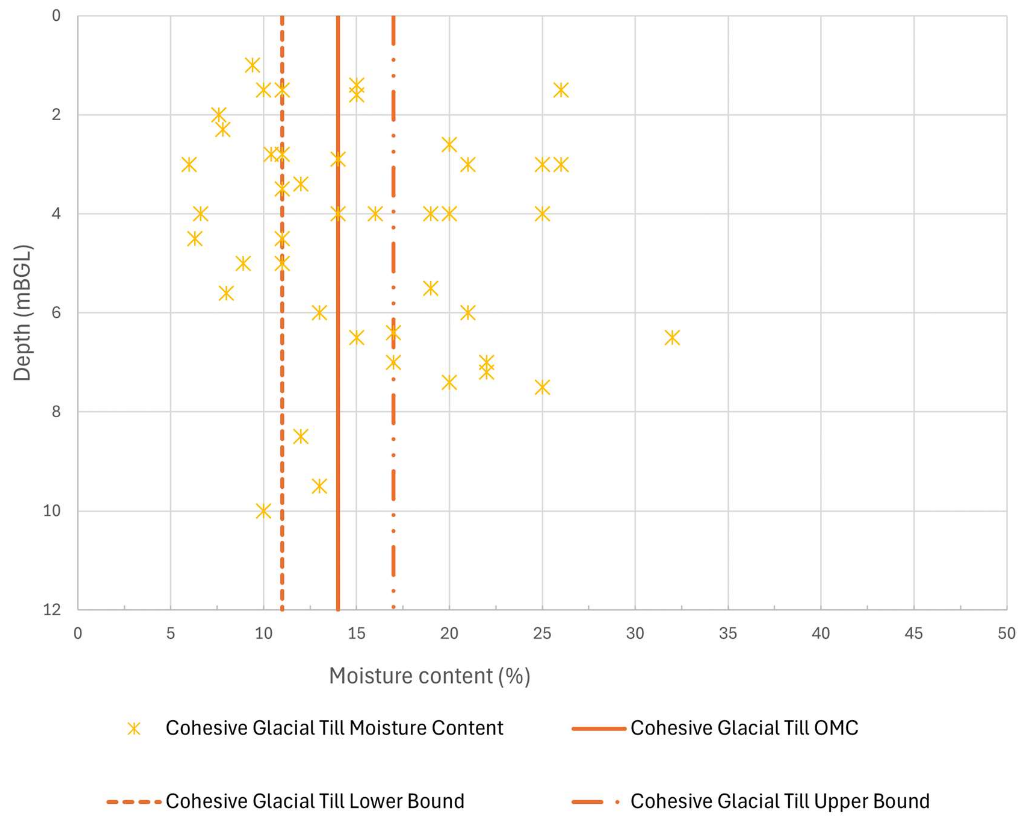


Figure 7-2: Moisture content results and optimum moisture content ranges vs depth for cohesive glacial till.

Moisture condition value (MCV) tests were carried out on three samples (outlined in Section 5.7.1). Typical ranges for acceptability of cohesive materials for use as cohesive fill to earthworks are typically MCV values of 8-15. Figure 7-3 illustrates that all but two of the tests fall within the acceptable range, with two tests on a sample of glacial till from M-BH03 falling below the acceptable range.

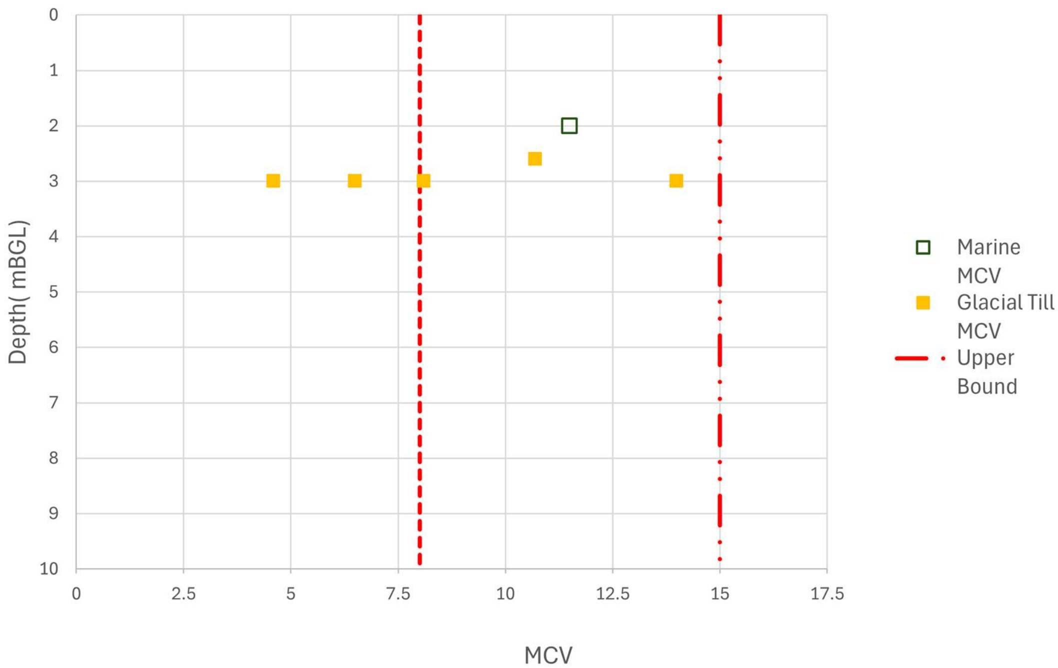


Figure 7-3: MCV vs depth, with suggested upper and lower limits.

Particle size distribution and Atterberg limit testing undertaken within the soil units indicates that the cohesive marine deposits are dominated by fine-grained constituents, behaving largely as CLAY. Grading curves are variable, but suggest that a large number of samples would meet grading requirements for use as Class 2 cohesive fill. The cohesive glacial till is more highly varied, with frequent granular layers, but is in general dominated by CLAY constituents. Grading curves vary considerably, with a lower proportion of samples meeting Class 2 grading requirements.

One lab California Bearing Ratio (CBR test) was carried out on a remoulded sample of cohesive glacial till taken from M-BH07 (Section 6.1.6). This test achieved a CBR of 1.8%. This result is indicative of how this material may perform when disturbed and recompacted.

It is likely, based on the soil strength parameters derived, and the CBR testing, that ground improvement will be required within the soil strata. Short and long term settlements will need to be considered in any designs. Where appropriate, ground improvement strategies such as vertical drains may need to be implemented to accelerate settlements, or stabilisation or soil mixing techniques to improve strength of the reused materials. However, further assessments will need to be carried out to assess the suitability of these methods.

7.2 ROCK RE-USE

A rippability assessment was carried out based on the excavatability classification system proposed by Pettifer and Fookes (1994). The chart is based on the fracture spacing index (FSI), point load (PLT) and UCS for the site, which is presented in Figure 7-4. Variability in rock strength observed in strength tests including UCS and PLT reflects on this plot when combined with the fracture spacing index. The results of this assessment indicate that the mudstone will be largely suitable for extraction methodologies ranging from easy digging to easy ripping, with a couple of samples indicating hard ripping. The two sandstone samples indicate that extraction by hard ripping will be required where these are encountered, and the dolerite samples range between hard digging and hard ripping. . Assessment of the metamorphic strata indicates that the amphibolite strata is highly variable, ranging from hard digging to blasting (one sample). The majority of amphibolite and gneiss samples indicate that easy to very hard ripping will be required.

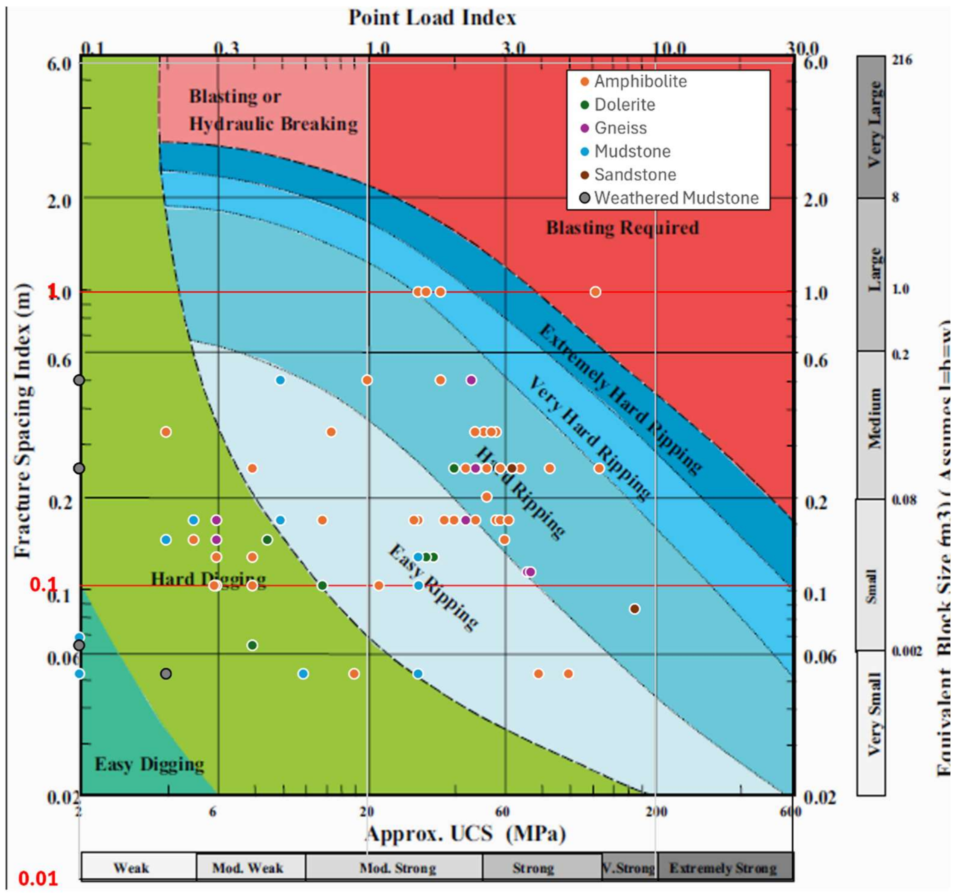


Figure 7-4: Rock rippability assessment, after Pettifer and Fookes (1994).

Variation in rock strength from that considered within this interpretation is highlighted as a key geotechnical risk and outlined in more detail within Section 9.

Fracture Spacing Index (FSI) has been plotted against elevation and is presented in Section 5.11.1.4. The plot doesn't show any significant trend with depth, where it may have been anticipated that FSI would decrease with depth as the degree of weathering should probably be reducing. It is noted that a wide range of Fracture Spacing Indices were recorded within the borehole logs.

7.3 CHEMICAL SUITABILITY

The chemical suitability of the soils with respect to risks to human health and the wider environment is important when considering the reusability of the soils. Section 8, Geoenvironmental Review, presents the findings from the geochemical soils analysis of soil samples obtained during the recent ground investigation. The results of the chemical testing are provided in Appendix F.

8 GEOENVIRONMENTAL REVIEW

8.1 INTRODUCTION

The following section provides a preliminary assessment of the investigation data with respect to potential risks to human health and the wider environment, including the water environment.

Soil samples were selected for chemical analysis to evaluate potential contamination risks to human health, infrastructure, and the water environment. A total of 23 soil samples were analysed, with 3 obtained from onshore boreholes and the remaining 20 from offshore boreholes. The analysis suite and chemical analysis results are detailed in Appendix F.

While the assessment includes a preliminary indication of potential health risks and environmental impacts to inform the design of the proposed development and its construction, it does not constitute a detailed contamination risk assessment suitable for a full assessment of potential contamination liabilities.

8.2 RISKS TO HUMAN HEALTH

Construction workers and future maintenance staff are considered receptors in this context. To assess the potential health risks to these receptors, soil chemical analysis results were screened against Tier 1 values protective of a commercial/industrial end-use scenario. For criteria sensitive to organic matter content, the most conservative Soil Organic Matter Content of 1% was utilised. The results of this analysis are provided in Appendix F.

None of the assessed determinands exceeded the Tier 1 commercial/industrial screening criteria, and no asbestos was identified in the soil samples. Given the chemical analysis results, the absence of identified contamination sources during the investigation, and the low sensitivity of the proposed development, the risk to human health from the site soils as part of the construction and final development is considered low.

8.3 RISKS TO THE WATER ENVIRONMENT

The proposed development is situated within a current port, with the potential for contamination sources that present a potential risk to the water environment. To assess these risks, geochemical testing has been undertaken on soil leachate samples collected from across the Site. The results have been screened against relevant inland water surface water screening values from the Water Framework Directive 2015, including Environmental Quality Standards (EQS) to assess risks to surface water. Although the Site is in close proximity to a large surface water body that is likely to be a major discharge zone, for completeness the results have also been compared with Drinking Water Standards to assess risks to the groundwater resource.

8.3.1 SOIL LEACHATE PROTECTIVE OF GROUNDWATER

A summary of the Tier 1 exceedances recorded in the soil leachate results is provided in Table 8-1, which included three metal and three polycyclic aromatic hydrocarbons (PAHs) compounds. The samples which exceeded their respective screening criteria for cadmium, copper, naphthalene, fluoranthene, benzo(a)pyrene are considered to be low risk to groundwater as they are within an order of magnitude of the screening criteria or only marginally above the limit of detection (LOD). Boron was observed to have frequently elevated concentrations compared to the screening criteria values, although in the absence of a known source of boron, these concentrations are likely to be natural occurring. This combined with the very low screening criteria means that there are a large number of exceedances, however the soil results do not indicate a significant volume of potential contaminant source and consequently the risk to groundwater from boron within the site soils is considered to be low.

Table 8-1 Soil Leachate Vs. Groundwater Screening Criteria Summary

Determinand	Maximum (ug/l)	Average (ug/l)	Groundwater Screening Criteria	Source	No. Exceedances
Metals					
Boron	230	70	0.001	E & W DWS	23(23)
Cadmium	1.0	0.09	0.005	E & W DWS	4(23)
Copper	2.5	1.2	2.0	E & W DWS	2(23)
PAHs					
Naphthalene	0.09	0.05	0.075	GQW	1(22)*
Fluoranthene	0.18	0.043	0.075	GQW	4(22)*
Benzo(a)pyrene	0.16	0.042	0.01	GQW	10(22)*

* *sample M-BH21 at 3.6m, is not included in the screening analysis of the polycyclic aromatic hydrocarbons (PAHs). This exclusion was due to the sample being run with a x100 dilution, resulting in a much higher limit of detection compared to the other samples.*

8.3.2 Soil Leachate Protective of Surface Water

A summary of the Tier 1 exceedances recorded in the soil leachate results is provided in Table 8-2, which included two metals and two PAH compounds. The samples which exceeded their respective screening criteria for copper, cadmium and benzo(a)pyrene are considered to be low risk to surface water as they are within an order of magnitude of the screening criteria or only marginally above LOD. Fluoranthene is observed to have frequently elevated concentrations when compared to the screening values, although the screening value is extremely low and the soil results do not indicate a significant volume of potential contaminant source and consequently the risk to groundwater from fluoranthene within the site soils is also considered to be low.

Table 8-2 Soil Leachate Vs. Surface Water Screening Criteria Summary

Determinand	Maximum (ug/l)	Average (mg/l)	Surface Water Screening Criteria	Source	No. Exceedances
Metals					
Cadmium	1.0	0.09	0.08	WFD 2015 Inland Surface Waters	4(23)
Copper	2.5	1.2	1.0	WFD 2015 Inland Surface Waters	11(23)
PAHs					
Fluoranthene	0.18	0.043	0.0063	WFD 2015 Inland Surface Waters	16(22)*
Benzo(a)pyrene	0.16	0.042	0.00017	WFD 2015 Inland Surface Waters	10(22)*

* *sample M-BH21 at 3.6m, is not included in the screening analysis of the polycyclic aromatic hydrocarbons (PAHs). This exclusion was due to the sample being run with a x100 dilution, resulting in a much higher limit of detection compared to the other samples.*

8.3.3 WATER ENVIRONMENT CONCLUSIONS

The chemical analysis results indicate there are some potentially leachable levels of metal and organic contamination within the natural soils, although these are generally at low levels and a significant source of soil contamination has not been identified. Consequently, the risk to the water environment from the site soils is considered to be low.

The risk to the water environment during construction is also low, assuming that surface water and groundwater protection measures are undertaken in accordance with a suitable Construction Environmental Management Plan (CEMP).

However, considering the possible significant volume of earthworks, and the associated potential for this to affect the chemical quality of the water environment, further assessment should be undertaken as part of the detailed earthworks design, which may need to include detailed quantitative risk assessment (DQRA).

8.4 RISKS FROM GROUND GAS

During the ground investigation no contamination with the potential to generate significant ground gas was observed, and a review of the logs found no materials that would be expected to be a significant source of ground gas. Considering this, and the low sensitivity of the proposed development, the risk to the proposed development from ground gas is currently assessed as low. However, it is noted that this assessment is based on a review of available data and that no gas monitoring was undertaken during the 2024 ground investigation.

Given that the proposed development involves the reuse and movement of a significant volume of materials, further assessment of their potential for ground gas generation should be undertaken as part of the detailed earthworks design. Once the earthworks design is finalised further investigation and assessment of the risk may be required.

8.5 REUSE OF SITE-WON MATERIALS

The chemical analysis screening indicates a low risk to human health and the water environment. The reuse of the materials will likely be within the Site and in the same general environment, so there is no increased risk to receptors. However, it is recommended that contamination be considered as part of the earthworks specification to confirm the material suitability for re-use within the Site. The requirements outlined in Section 8.6.2, on marine environmental testing should also be considered.

8.6 SOILS DISPOSAL

8.6.1 GENERAL WASTE

A preliminary assessment based on observation from the boreholes suggests that if materials are required to be removed from the Site, predominantly inert classifications are likely to be encountered. Before any material is disposed of off-site, specific waste classification testing and possibly waste acceptance criteria (WAC) testing should be undertaken. Disposal of such waste must be undertaken in accordance with all relevant current waste legislation and duty of care regulations

8.6.2 MARINE

As part of the Causeway Geotech (2024) ground investigation campaign, 33nr. Marine Institute Environmental testing suite were carried out on samples table from Vibrocore locations across the proposed dredge area of the site, and grab samples from within the existing small boat harbour. The tests targeted generally shallow depth 0 – 1.5mbgl, with the methodology of conducting further testing should results suggest that any contamination is present at the site.

The assessment of the results of these samples in outlined in Appendix D in line with the classification criteria outlined in the EPA guidance document "Guidelines for the Assessment of Dredge Material for Disposal in Irish Water (Cronin et al., 2006)". A full summary of the results can be seen in Appendix D.

Assessment against the Marine Institute Suite results demonstrated that the majority of test results are below lower level – thus suitable for disposal at sea. In 21 samples, Arsenic was detected marginally within the upper and lower limits (9-70 mg/kg), with a maximum recorded value of 21.2mg/kg in MAR02235.018, taken from 0-0.5m BGL in M-VC42. One sample (MAR02235.024), taken from 0-0.3m BGL in M-VC56 recorded a Copper level of 53.9mg/kg marginally within the upper and lower limits (40-110mg/kg). In 16 samples, Nickel values were detected marginally within the upper and lower limits (21-60mg/kg), with a maximum recorded value of 53.3mg/kg in MAR02235.018, taken from 0-0.5m BGL in M-VC42. Test results from the determinants, gamma - HCH (Lindane) and HCB (Hexachlorobenzene) were recorded as being below upper limit in all samples tested, however it should be noted that the limit of detection for gamma - HCH and HCB was 1.00 µg/kg; with both sample results reported as <1.00 µg/kg. Given the test results from a high proportion of other determinants tested was below the lower limit, it is likely that with improved limits of detection for gamma - HCH and HCB, that test results would also lie below lower limits also. Two samples detected total hydrocarbon content (THC) values above the lower limit of 1,000,000ug/kg, with 3,850,000ug'kg detected in MAR02235.001 (taken from 0-0.15m BGL in GS01), and 6,900,000ug/kg detected in MAR02235.003, taken from 0-0.15m BGL in GS05.

Thus, if these test results are used as an indicator for the more widespread area at depth, then we may conclude that any new dredging works within the area of sampling would be suitable for disposal at sea. It is however likely that further sampling and testing will be needed in order to gauge the opinion of the Marine Institute.

8.7 Unexpected Contamination

If any unforeseen contamination is identified during construction (e.g., hydrocarbon impacted soils, asbestos, etc.), then work in such areas should be halted until a suitably qualified professional has been consulted to assess the situation and provide advice.

8.8 Imported Soils

It is recommended that any imported soils or fill materials required for construction purposes are subject to assessment to demonstrate their suitability for use, which may need to include chemical analysis and assessment against relevant screening values.

9 GEOPHYSICAL SURVEY

The geophysical surveys were carried out at the site on 27th and 28th of October, on the 07th of November 2023 and on the 21st May 2024.

The scope of work of the geophysical survey campaigns included the following:

- Multibeam echosounder (MBES),
- Marine magnetometer,
- the sidescan sonar (SSS),
- Sub-bottom profiler (SBP), and
- Boomer seismic.

Findings and interpretations of the geophysical survey are outlined in the Hydromaster (2024) geophysical report in Appendix C.

The multibeam echosounder (MBES), the marine magnetometer, the sidescan sonar (SSS) and the sub-bottom profiler (SBP) surveys were conducted simultaneously within the area of the purple boundary shown in Figure 9-1. These surveys were carried out ahead of the geotechnical survey to derisk the works related archaeology and/ or obstructions to the placement of the marine jack up platform and drilling works.

The boomer seismic survey was conducted within a reduced area as shown in the cyan boundary in in Figure 9-1.

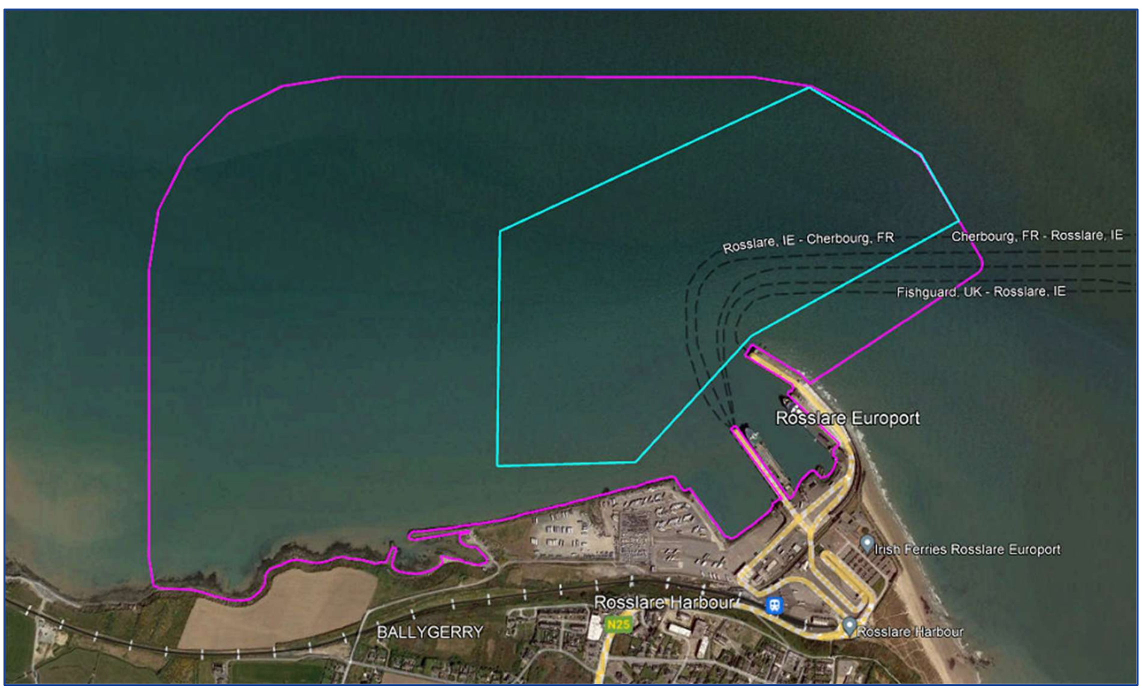


Figure 9-1: Geophysical survey areas

9.1 SURVEY RESULTS AND INTERPRETATIONS

9.1.1 BATHYMETRY AND SEABED TYPE

The results of the MBES surveys outline water depths range from 0 m LAT to 11 m LAT, with deeper water depths within the harbour and on the west part of the site.

A few boulders and outcrops/reefs can be identified. The harbour seabed is dominated by scours created by prop wash from large vessels, and with some sediment deposition.

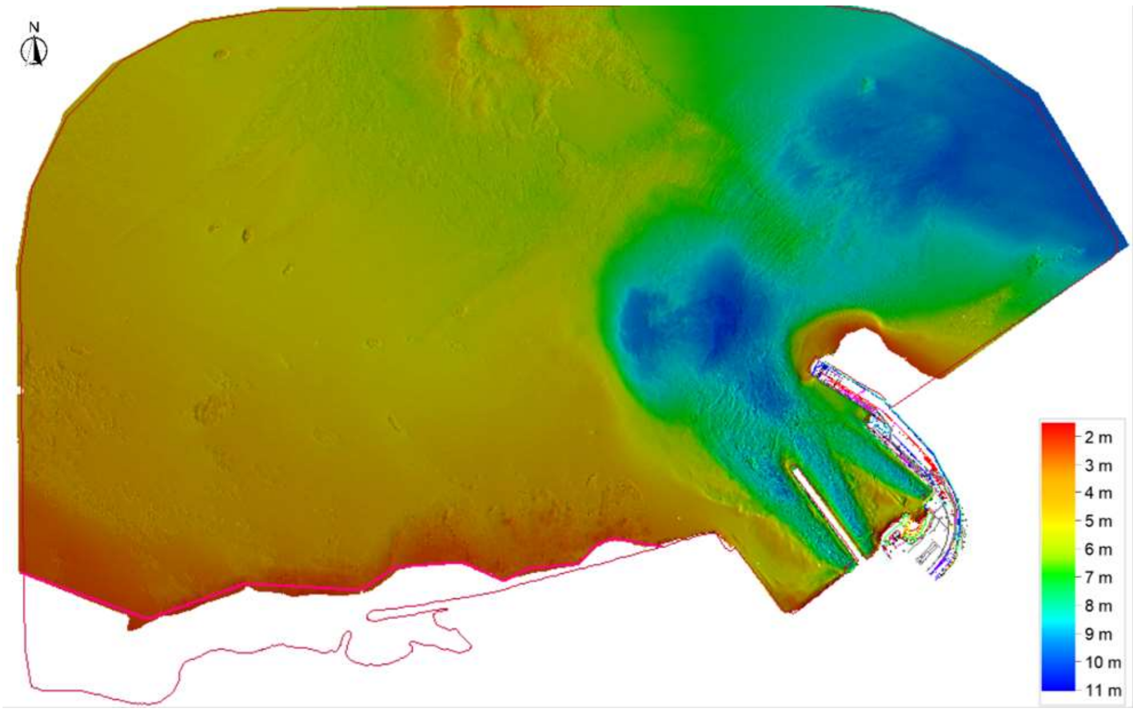


Figure 9-2: Bathymetry data

9.2 SEABED TARGET IDENTIFICATION

Several items or 'targets' were identified on the seabed within the survey area. The MNBES, SSS and MM surveys were used to identify and characterised these items. Many of the items are small to medium sized boulders, blocks and items related to part works and operations at the port such as fish pots and moorings. Two potential small buried wrecks were identified at the site, identified in the magnetometer and MBES surveys. These items are located at AT_05 and AT_06 as outlined in Figure 9-3. These wrecks are not identified in the INFOMAR shipwreck archive.

The full list of these items can be examined in the Hydromaster (2024) report in Appendix C.

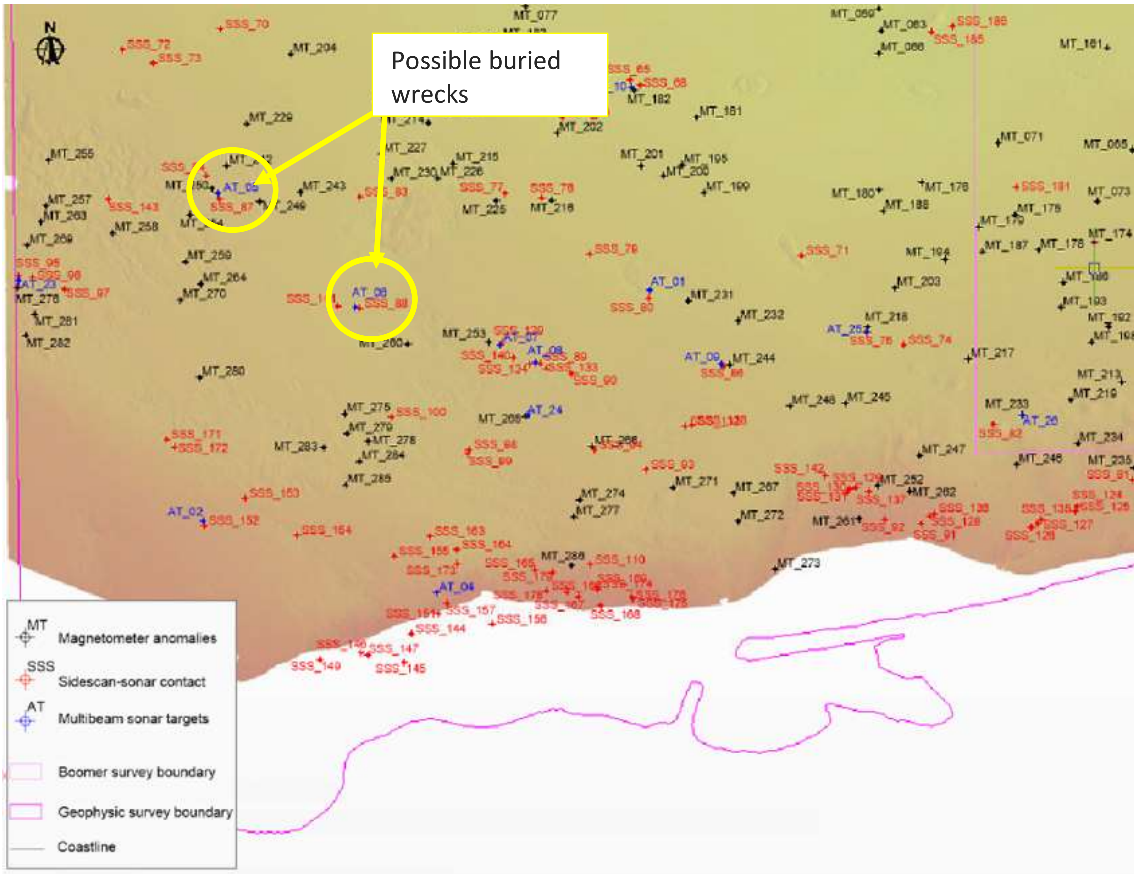


Figure 9-3: Seabed target identification

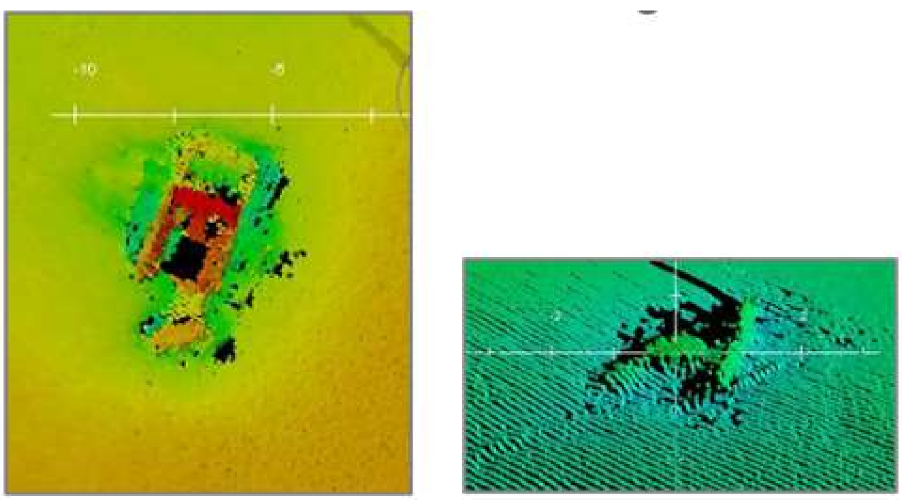


Figure 9-4: Identified possible shipwreck targets

9.3 BEDROCK DEPTH INTERPRETATIONS

High frequency sub-bottom profiler surveys were carried out across the full site survey area with a more details, higherpenetration medium frequency boomer survey carried out within the reduced survey area at the proposed dredge area as identified in Figure 9-1.

The results of the high frequency sub-bottom profiler suggest an overall good quality , however, signal penetration was limited by sediment type below the seabed but overall achieving good penetration on most areas. The results give an indication of depth to bedrock level below sea level. The results correlate relatively well with the borehole locations indicating a level +/- 1-2m from the borehole identified bedrock levels. The outlined bedrock levels struggle in areas where the bedrock level was deeper than ca. 8m.

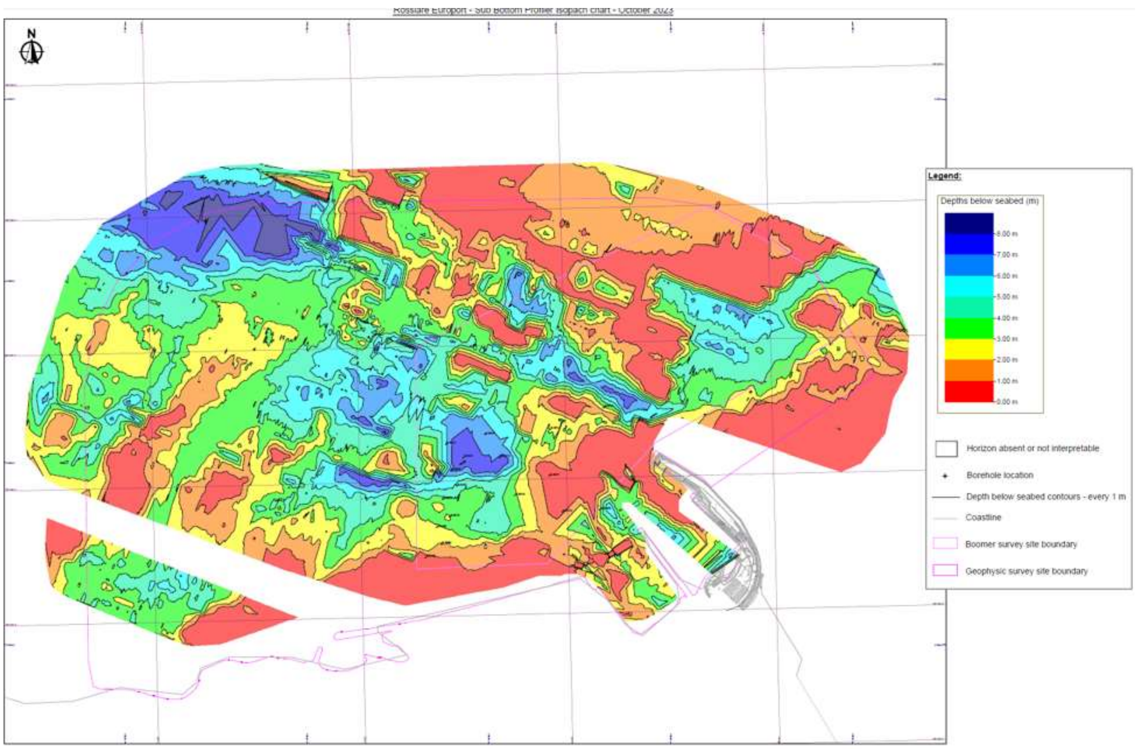


Figure 9-5: Results of high frequency sub-bottom profiler survey

Medium sub-bottom profiler survey information was gathered in a reduced area to gather information on materials at greater depths. The medium frequency system will sacrifice detailed resolution for depth penetration and so as a result the information collected by this system for shallow materials is limited. These surveys were predominantly used to identify a more detailed picture of the bedrock profile and identify paleo channels within the overburden soils.
The interpreted results are shown in Figure 9-6 and Figure 9-7.
The surveys correlate relatively well with the borehole data, indicting depth +/- 1-2m of the borehole and CPT information and do not indicate any major or sudden elevation changes in the bedrock profile. The area to the southwest of the survey area, adjacent to BH01 reflect the increase in depth to bedrock as outlined in the borehole and CPT logs.

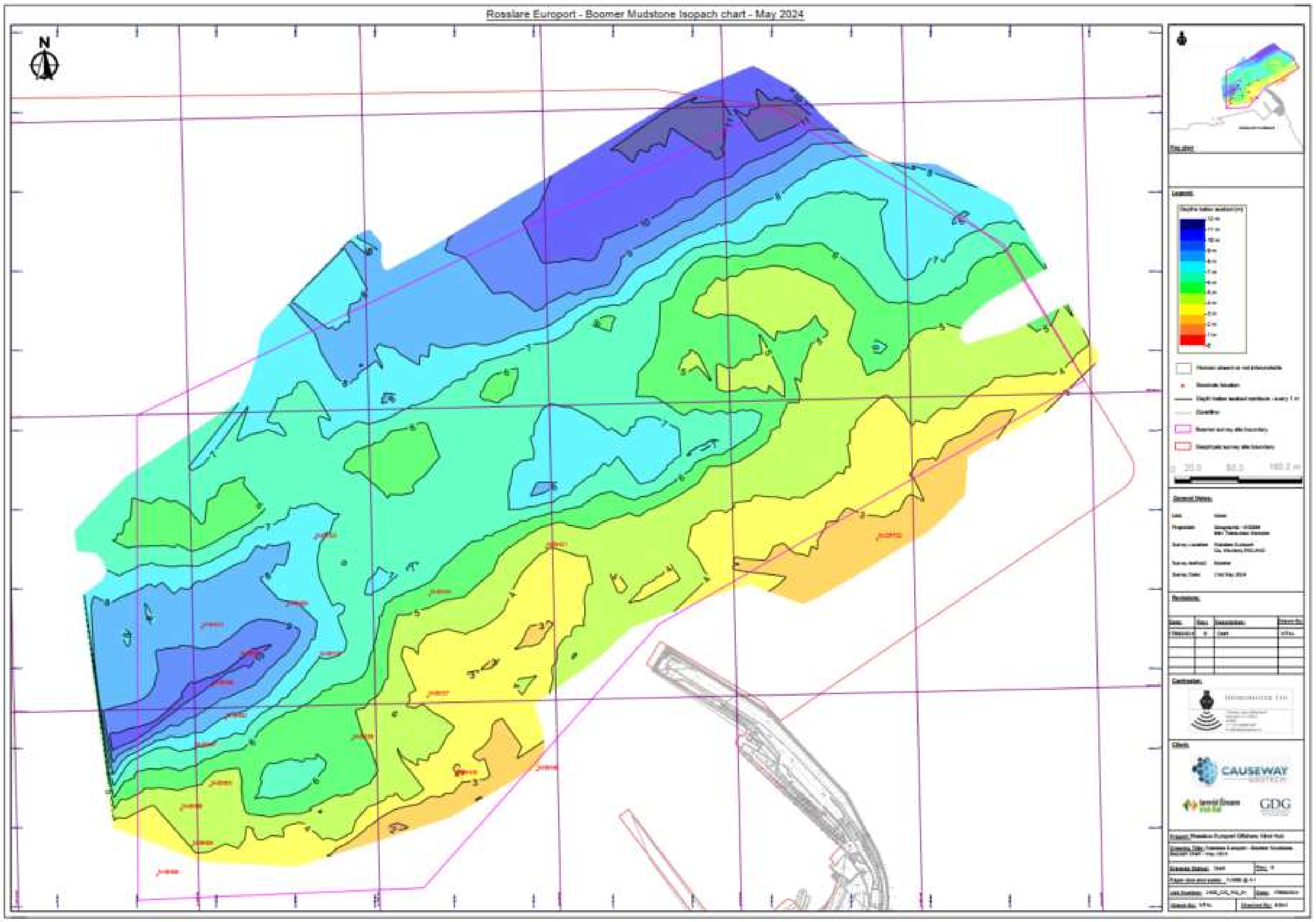


Figure 9-6: Boomer survey bedrock level isopach

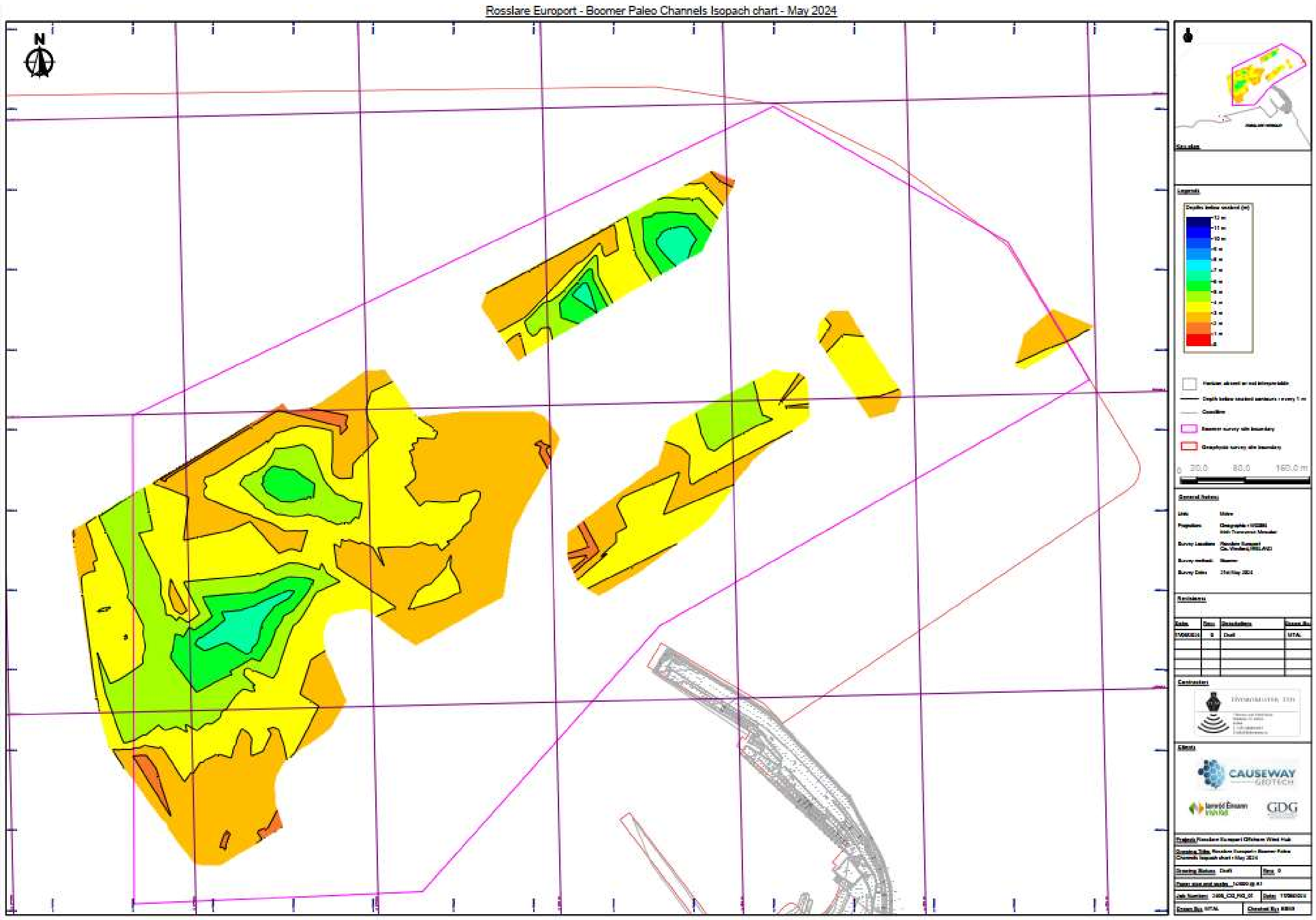


Figure 9-7: Paleochannel interpretation from boomer survey

10 GEOTECHNICAL RISK REGISTER

The following items have been identified as plausible geotechnical risks and should be incorporated into any risk registers or assessments for the project as a whole. The items included within the Geotechnical Risk Register (GRR) 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 do they 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 shall satisfy themselves that all hazards have been eliminated or mitigate any remaining risks as far as is 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 10-1: Geotechnical risk register

No.	Risk description	Mitigation
1	Variability of chosen engineering parameters. Risk for conceptual design.	The interpretation has derived parameters to be used within the design which have been based on highly variable information within the area of interest. Whilst consideration of the parameters has allowed for the anticipated range, there is a risk that the parameters have over/underestimated the strength characteristics of the soil and rock profiles. Within the design work appropriate sensitivity analyses should be completed to ascertain which parameters most heavily influence the design outputs.
2	Variation in rock strength across the site. Rock at the proposed location more difficult to remove than samples from within the harbour suggest.	The rippability assessment considered a relatively small number of rock samples within the boundary of the proposed development. From review of the fracture spacing index and point load test values, the samples generally indicated that hard digging to hard ripping would be suitable. Should samples within the proposed development area be of higher strength, blasting may be required as a method of rock dredging. This may have implications for consenting, project budgets,

No.	Risk description	Mitigation
		methodologies, and programme. It is noted that blasting was previous required within the harbour. This is highlighted as a high-risk item.
3	Underestimation of rock strength in amphibolite and mudstone strata.	Core recovery within the mudstone stratum in particular, but also within the amphibolite stratum in places, was poor. The mudstone often became extremely friable when being extracted from the core lining, meaning that samples were often of very poor quality when subjected to testing. Amphibolite samples were additionally often of poor quality when subjected to testing. This may lead to an underestimation of strength parameters for the in-situ bedrock. Further
4	Presence of organic material. Risk that organic matter content may have been underestimated.	Borehole logs have indicated the presence of organic material within the cohesive stratum. This will be relevant to the degree of settlement which may be possible within this layer, with organic material decaying and leading to additional reduction of volume. One sample taken from this stratum has been tested for organic matter content, reporting a result of 0.06%. Two samples of cohesive glacial till were also tested, reporting results of <1%. Due to the small number of samples tested, it is possible that the organic matter contents could be higher than reported in some areas.
5	Obstructions (e.g boulders,) within the underlying stratigraphy.	Several SPT tests within the cohesive glacial till strata recorded 'N' values of N=100. These are considered refusal values, and very likely in many places record interaction with large cobbles or boulders which may not have been recovered by the drill runs. These obstructions may cause higher SPT 'N' values to be recorded than are representative of the entire soil mass, and may lead to overestimation of soil strength parameters based on SPT correlations. To mitigate against this, higher weight has been placed on lab testing, and conservative values have been selected based where SPT correlations have been

No.	Risk description	Mitigation
		used. These boulders and obstructions may also cause difficulty during pile installation – leading to early refusal.
5	Presence of buried services and hazards.	As outlined in Section 2.9, no indication of submarine cable or pipelines is identified from national level mapping. This GIR has not assessed this in further detail. A detailed assessment will be required before works commence.

11 CONCLUSIONS

This report presents the findings of an intrusive geotechnical and geophysical Site Investigation carried out by Causeway in 2024 at the proposed ORE Hub development at Rosslare Port. The Causeway (2024) SI provided full coverage to the ORE Hub development site and the geotechnical parameters are defined based on the findings of the in-situ and laboratory testing within the borehole, CPT, and vibrocore sampling locations as outlined in the factual report Causeway (2024). The intrusive geotechnical campaign was accompanied by a geophysical survey derisking the seabed environment and aiding in creating a extensive plan if material and bedrock depths across the site. A ground model has been defined in Section 5, and is summarised in 5.4.

The ground conditions encountered at the site has been divided into land-based and marine-based ground models, summarised in Table 5-1 and Table 5-2 respectively. The land ground conditions consist of granular MADE GROUND overlying granular glacial till, overlying metamorphic bedrock, described as AMPHIBOLITE. The marine ground conditions consist of cohesive marine deposits overlying cohesive glacial tills, overlying bedrock, which varies between metamorphic units (amphibolites and gneiss) in the near-shore locations, and mudstones, sandstones, and rare dolerite in the more distal locations.

The ground model for the site has been evaluated. Characteristic soil and parameters are proposed for the materials which include:

- The bulk unit weight of the soil and rock materials,
- The SPT-N value of the soil materials,
- The unconfined compressive strength of the rock,
- The undrained shear strength of the cohesive soil materials,
- The effective friction angle and cohesion of the soil and rock materials,
- The static and dynamic Young's Moduli of the soil materials,
- Compressibility characteristics of soil materials, and
- California bearing ratio of soil material.

Geotechnical parameters have been derived from the results of lab testing and correlations from SPT and CPT data based on correlations from published datasets. A summary of parameters is outlined in Section 6.2. Variations from this table may be required for other limit states, temporary works designs and constructability-related assessments. These tables may be subject to change in later revisions of the GIR should further information become available and justify such alterations. The ground model and characteristic parameters have been developed on a site-wide basis, with a significant degree of variability identified within the data. GDG further recommends that each designer give consideration to the local ground conditions and testing results in the relevant area of their proposed design, creating geological sections as required for their relevant design locations.

This interpretation has used all available intrusive investigations, geophysical surveys and information available from GSI (Geological Survey Ireland). The interpretation has provided a ground model for the proposed site, in addition to characteristic parameters. GDG has also identified several geotechnical risks and provided recommendations for mitigation measures in a geotechnical risk register in Section 9. A Design Risk Assessment (DRA) has also been presented in Appendix E.

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Appendix A SITE LAYOUT AND SI LOCATIONS

Appendix B BATHYMETRY

Appendix C Causeway Factual Report

Appendix D ENVIRONMENTAL CONTAMINANT TESTING

Appendix E Design Risk Assessment

Appendix F GEOENVIRONMENTAL SOIL RESULTS

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