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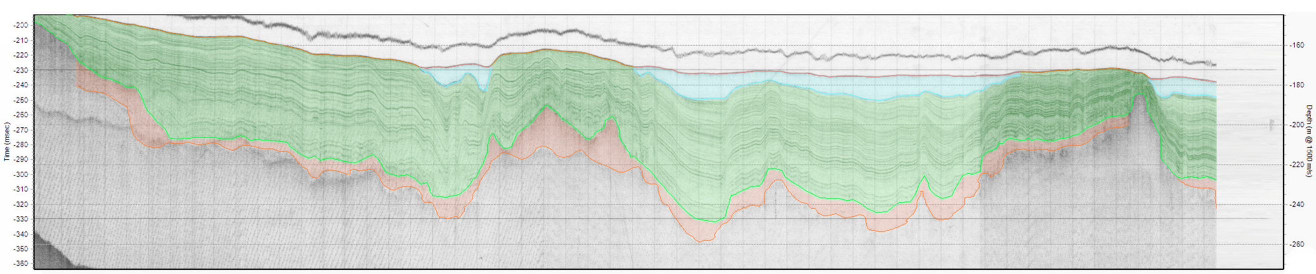
Geotechnical parameters important for offshore wind energy in Atlantic Canada



J.B.R. Eamer, Z. Levisky, and K. MacKillop

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Permanent link: https://doi.org/10.4095/329688

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Recommended citation
Eamer, J.B.R., Levisky, Z., and MacKillop, K., 2022. Geotechnical parameters important for offshore wind energy in Atlantic Canada; Geological Survey of Canada, Open File 8873, 41 p. https://doi.org/10.4095/329688

Publications in this series have not been edited; they are released as submitted by the author.

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The authors acknowledge a helpful and constructive review from Dr. David Mosher and funding from the Renewable Energy and Electricity Division of Natural Resources Canada.

Cover illustration: Interpreted Huntec DTS seismic profile from the Eastern Scotian Shelf. Orange, green and blue represent till, stratified glaciomarine mud, and Holocene mud, respectively.

1.0 Introduction

Wind power holds a high potential for supporting energy demands globally: seven times the world's electricity needs could theoretically be provided by just 20% of estimated available global wind power (Archer and Jacobson, 2005). Offshore wind resources, while currently contributing a small proportion of total wind energy generation (5% in 2020, IRENA 2021), are estimated to be ~90% greater than over land on average due to higher average wind speeds and reduced turbulence (Archer and Jacobson, 2005).

Offshore wind energy installations also benefit from geography: single leases can host large numbers of increasingly massive, high capacity turbines and can be located in close proximity to high-demand coastal cities, a feat not possible for onshore development due to the limited area for development around urban centers. Electricity produced from offshore wind energy converters has been an established component of the energy mix in Europe for decades, where a total of more than 5000 turbines with a capacity of more than 25 GW are in operation as of 2020 (Ramírez et. al, 2021). In contrast, as of 2021, there are only seven completed offshore wind turbines in North America, producing a total of 42 MW of electricity (Ørsted, 2020). However, after several years of important contextual and background research into a regulatory and physical science framework for offshore wind (e.g. BOEM, 2007; Fugro Marine GeoServices, 2017), there are many GW of offshore wind that are in the design, permitting, or construction phases destined for the shelf off the Atlantic coast of the United States. This includes a stated goal of 30 GW of offshore wind power capacity by 2030 (USDOE, 2021).

To date, a major hindrance to offshore wind energy development has been the relatively high cost of site characterization, deployment, construction, and maintenance compared to onshore wind energy or conventional power production. However, rapid technology development and innovation in all aspects of the supply chain has resulted in recent claims that "*the era of subsidy-free offshore wind turbines has begun*" (Jansen et al., 2020).

A key component to de-risking offshore wind energy is the development of a geotechnical model involving the accurate and efficient geotechnical characterization of the marine soils that may host offshore wind infrastructure. Offshore wind-specific geotechnical modeling and characterization approaches are starting to reach maturity (e.g., Byrne et al., 2017; Burd et al., 2020a,b; Byrne et al., 2020). This report aims to review the current methods of offshore wind geotechnical characterization and provide context for Atlantic Canada-specific soil conditions.

1.1 Offshore Wind Turbines

Offshore wind turbines (OWTs) are constructed in a variety of sizes and are rated for different power outputs. Characterization of the wind at a proposed offshore wind project location is important in determining the correct class of OWT for that location. OWTs are classified based on design thresholds for average wind speed and turbulence: I, II and III, which correspond to average wind speeds of 10 m/s, 8.5 m/s and 7.5 m/s, respectively, and A, B and C with turbulence values of 16%, 14% and 12% (IEC, 2015). In combination, this results in 9 distinct classes of commercial OWTs. As of 2021, the most powerful commercially available

OWT has the capability of producing 10 MW. OWTs that have recently received type certification (but not yet deployed to a commercial installation) produce up to 15MW. For reference, the highest capacity production onshore turbine is 7.5 MW. The additional foundation height, rotor length, and turbine size to accommodate the requirements for larger generators also contributes to additional loads experienced by OWT foundations (Wu et al., 2019).

Commercial wind turbines of all classes incorporate a similar design in both onshore and offshore wind project developments. Wind turbines are built upon a conical steel tower that supports the power generating components of the structure. The average height of wind turbine towers, from their base to their hub, is 90 m and has been slowly increasing over that past 20 years (Wiser et. al, 2020). There is currently development and research on using towers made of reinforced concrete or wood, but at present, steel is the industry standard. Important components for the generation of power are found within the nacelle at the top of the wind turbine and include the generator, gearbox, brakes, shafts, and controller. The last component of a wind turbine is the rotor assembly which consists of the blades and the rotor. Commercial wind turbines typically have 2-3 blades with an average "swept diameter"—the diameter of the circle the rotating blades creates—of 121 meters. Since 1998, the average swept diameter of wind turbine blades in the United States has increased over 250% from 47.8m to 121 m (Wiser et. al, 2020). This growth significantly outpaces that of turbine height, indicating that larger blade diameters are being installed on relatively smaller towers.

The development of offshore wind suffers from higher ratios of investment and subsequent higher operation costs (Sánchez et al., 2019). The design, construction and installation of OWTs is generally costlier than onshore due to the inherent complexity of working in a marine environment, including factors such as corrosion, logistics, safety and weather. Furthermore, additional work must be done to connect OWTs to the power grid through underwater cables and a connection point on land.

1.2 Fixed-bottom Offshore Wind Turbine Foundations

OWT foundations must withstand high lateral loads produced by the combined force and associated moments (the measure of a force's tendency to cause a body to rotate about a specific point or axis) of the wind and waves against the length of the structure. These loads are distributed down through the foundation into the seafloor sediment, ensuring that the structure remains within its specified tolerances for movement. Decades of offshore oil and gas platform development and production provided fundamental data and theory regarding offshore foundations, the environmental applied forces, and the importance of seafloor characterization. However, the magnitude and character of forces on OWTs are different from that of oil and gas platforms (Leblanc et al., 2010). OWTs face different loads produced by the wind in comparison to oil and gas platforms as they are generally taller, are purposely located in regions with strong winds, and are designed to extract high amounts of kinetic energy from the wind for conversion into electricity. OWTs must also be designed with a specific fundamental frequency, f_0 (the first tower bending frequency) that does not lie within the same frequency of the rotational frequency of the rotor (f_{1P}), the blade passing frequency (f_{3P}), and the frequency of environmental components (winds, waves). This is done in order to prevent excitation in the structure resulting in resonance which can lead to structural failure (Kallehave et al., 2015).

Canadian waters off of the provinces of Atlantic Canada are frequently hit by large “nor’easters” (winter storms) and hurricanes (Figure 1), up to Category 4 with sustained winds of up to 251 km/hr. Offshore wind turbines and their foundations found in hurricane prone areas must be designed to deal with the increased loads produced from long lasting high winds and large waves. For turbines, a class “T” certification is granted in some jurisdictions to designate it as “typhoon proof”, generally involving shorter blade lengths and strengthened tower and foundation components. In addition, OWTs can, in extreme events, enter a “survival mode” where the rotor is locked in place and the blades are rotated to be parallel to the wind (Hartman, 2018).

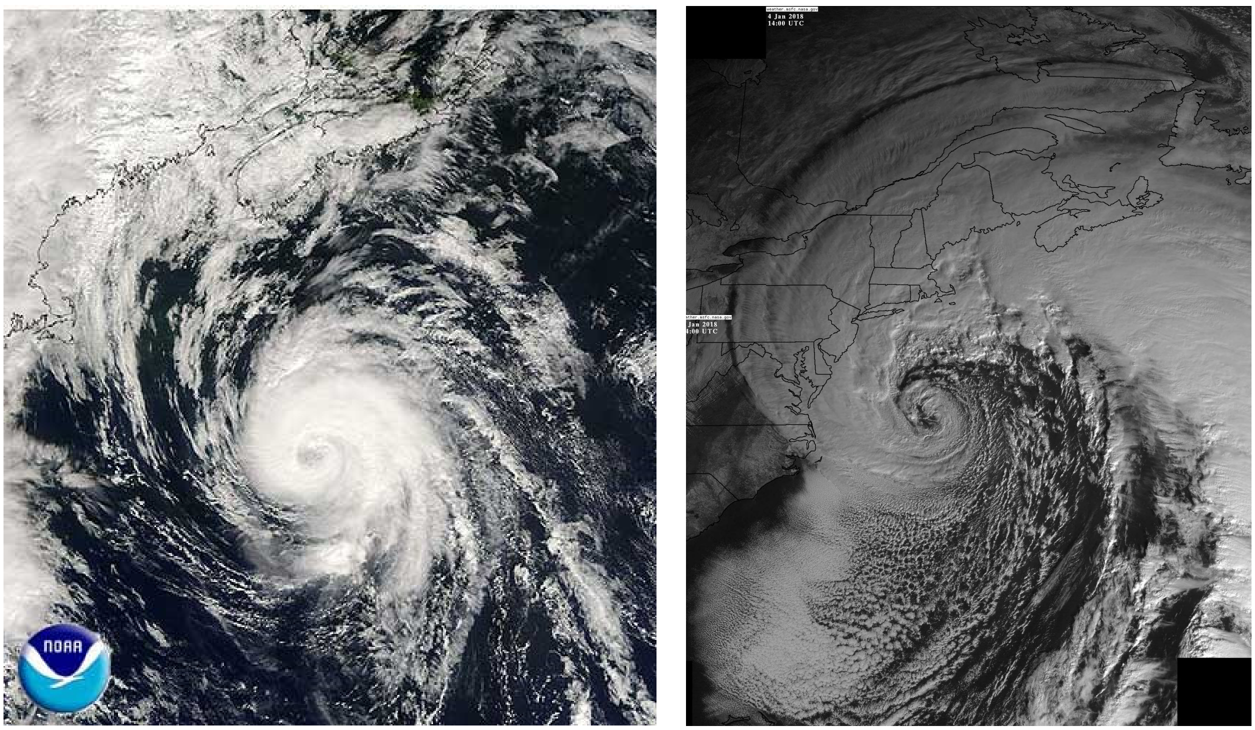

Figure 1. Left: Hurricane Juan (2003) on approach to Nova Scotia, at the time a Category 2 hurricane. Right: “Bomb” cyclone nor’easter, with hurricane force winds, in 2018. Both storms crossed Canadian waters and made landfall in Canada’s Atlantic provinces. Satellite imagery from NOAA (MODIS and GOES, respectively).

The design and installation of OWT foundations from an engineering perspective involves extensive testing and modelling to ensure that a turbine will operate safely and efficiently for its expected lifetime. Instrument deployments are typically performed during the evaluation and design phases to collect data on the meteorological and oceanographic conditions at a proposed wind project site to establish baseline conditions. Geophysical surveys are completed to obtain information on the geology of the seafloor, including bathymetry, surficial geology, and subsurface stratigraphy (BVG, 2019). Ground-truthing of the geophysical data is conducted concurrently with geotechnical surveys, using cone penetrometer testing (CPT), boreholes/coring, and grab samples of seafloor sediment. After these data are collected and interpreted, a suitable foundation type is selected (Figure 2) and the foundation parameters (e.g., depth of embedment, weight, cross-sectional area) are selected for the site conditions.

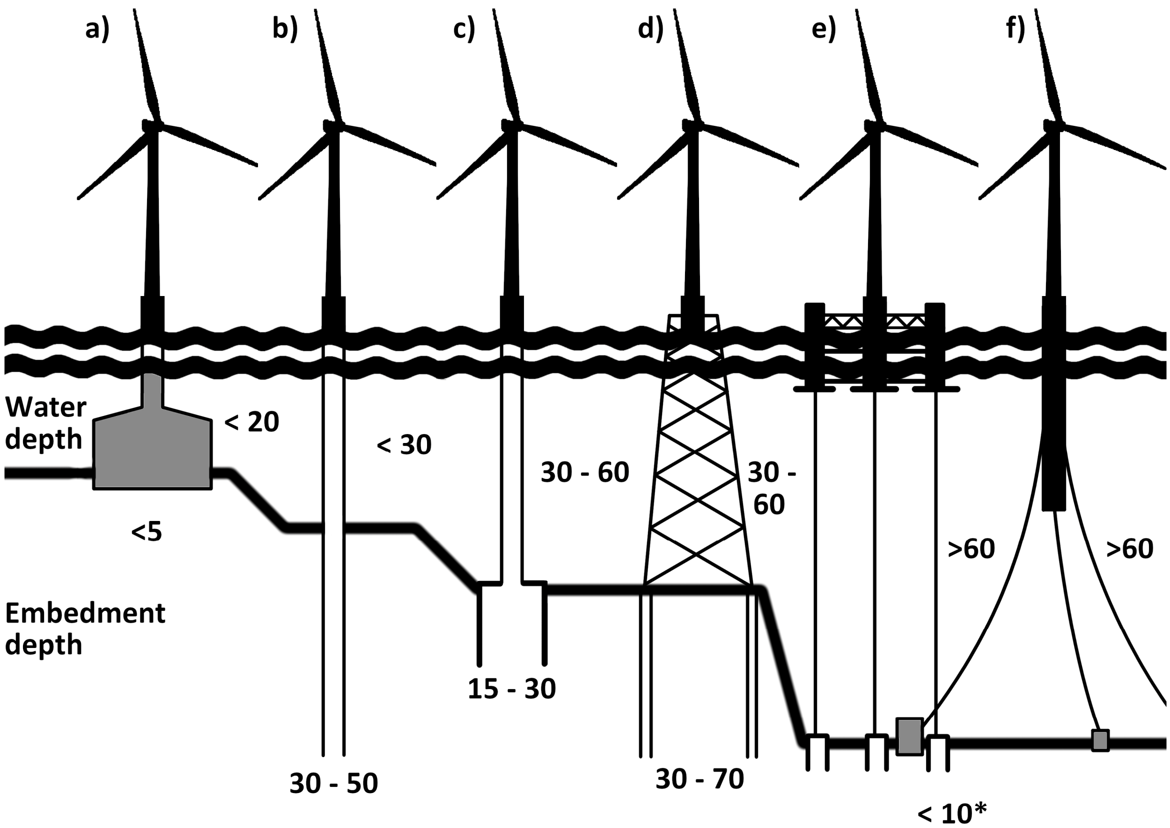

Figure 2: Offshore wind turbine foundations, with general water depths and embedment depths indicated (Eamer et al., 2021)

1.2.1 Monopile

The most common foundation type used for fixing OWTs to the seabed is the monopile (Figure 2b), with over 80% of installed turbines in Europe using monopiles at the end of 2020 (O'Sullivan et al., 2021). Monopile foundations consist of a single steel tube that is driven deep into the seafloor sediment, supporting a transition piece at the water surface to which the tower is attached. Monopiles are currently used in water depths shallower than 40 m. They are the most commonly used offshore wind foundation due to their simplicity to produce and install along with their reliability from years of use in the offshore oil and gas industry (Fugro Marine Geoservices 2017). However, monopiles are limited to certain water depths and require specific thick soil deposits. Determining the appropriate dimensions of a monopile, such as the diameter, thickness and emplacement depth is highly dependant on a variety of factors, including the size of wind turbine being installed, the depth of water, the geotechnical properties of the marine soil and the outside forces created by wind and wave action (Negro et al. 2017). For example, a 5

MW turbine being installed in a water depth of 30 m would require an expected monopile foundation with a 7 m diameter, 70 mm thickness and 28 m embedment depth (Igoe et al., 2018). This scenario is today considered a lower bound for offshore wind, where the average capacity of installed OWT in 2020 was 8.2 MW (O'Sullivan et al., 2021). In addition, monopile foundations are theoretically applicable in greater depths (e.g., >50 m), which would require larger diameter piles, deeper embedment, and greater steel thickness in order to withstand increased loads and overturning moments, or the moment of energy capable of overturning the foundation. To support the loads created at these depths, 9 m diameter piles would be required, emplaced in at least 50 m of suitable soils (Kiełkiewicz et al., 2015).

Monopile foundation designs are modified based on the loads and the subsurface conditions of the marine soils. Geotechnical and geophysical surveys are completed prior to the foundation design to determine engineering parameters that will influence how the pile is constructed. One key geological factor affecting the viability of monopile development is the requirement for thick (generally >25 m) deposits of suitable sediment, typically silts and sands. Once a monopile is driven into the soil, the tolerances for permanent displacement and deflection are very small. The accepted maximum allowable rotation of a pile is generally 0.5 degrees with a maximum deflection of 120 mm at the mudline and 20 mm at the toe of the pile (DNV, 2014). As a result of these small tolerances, monopiles need to behave as stiff structures. This differs from some of the design methods for monopiles derived from research based on piles used for the offshore oil and gas industry. Piles used in the oil and gas industry are long, slender and behave in a flexible manner (Leblanc, 2010), which differ from the larger, more rigid monopiles that host wind turbines in the offshore. As a result, these larger monopiles designed with methodologies developed for oil and gas may be outside the verifiable range of the design method (Byrne et al., 2017). Despite these unknowns, monopiles have gained recognition for being safe and reliable for offshore wind applications. Furthermore, newer modeling and design methods, for example finite element analysis, have proven to reliably estimate the pile-soil interactions of driven piles (Verlarde et al., 2016).

1.2.2 Multipiles

OWTs may also be supported by more complex structures, including lattice-like steel structures called jackets (Figure 2d) or in a tripod configuration, both of which require multiple piles (hence multipiles) embedded into the seafloor. Tripods consist of three piles that are connected to a central shaft that rises to above the water surface. Jacketed foundations consist of piles interconnected by a steel lattice structure that rises to connect to the OWT. The primary advantage of both foundations is that they offer a more rigid structure in comparison to a monopile of the same size and can be built in deeper water depths as a result (Sánchez et. al, 2019). The general principles applied to monopiles, with regards to the physical and geotechnical properties of the soils, can be applied to multipile foundations due to the similar method of emplacement. Multipile foundations are generally more expensive than monopiles as they require more material and take longer to install (Kopp, 2011). However due to the water depth versatility of jacketed foundations, they are currently the second most commonly used OWT foundation (Sánchez et. al, 2019).

1.2.3 Gravity Based Foundations

Gravity based foundations (GBF) are best suited for shallow water depths (Figure 2a). Gravity structures have a simple design, consisting of a tower structure connected to a large mass that rests on the seafloor. The mass is most commonly made of concrete and ballasted to help reduce the weight of the structure (Kopp, 2011). As a result of a large mass resting on the sea floor, specific geotechnical and geological requirements must be met in order for GBFs to be considered suitable. The marine soil must have a high bearing capacity and not underlain by weak soil layers, thus preventing unwanted settlement or displacement of the structure (Esteban et. al, 2019). Prior to the deployment of a GBF, the seabed at the site is typically prepared (leveled via removal or addition of material) to ensure that the structure remains level. More scour protection is commonly added to GBFs than other OWT foundations due to their large size and to prevent movement of the structure. Preparing the seafloor can be time consuming and expensive and adds to the cost of a project.

1.2.4 Suction Caissons

Suction caissons (Figure 2c) are attached to monopiles, jackets, or as anchors for floating OWTs and use the weight of the foundation and suction to embed into the seafloor (Kopp, 2011). The caisson, a large diameter cylinder with an open bottom, is first embedded to a shallow soil depth using gravity, followed by pumping out the remaining water in the caisson to pull the foundation into the soil via negative pressure. As with all other OWT foundations, horizontal loads and moments produced by wind and wave action are the primary concern for suction caissons. This is especially true as caissons are relatively small and light foundations yet need to resist the same forces as larger foundations. One concern with suction caissons is uplift of the caisson from the seafloor due to overturning moments (Villalobos et. al, 2004), lifting the caisson out of embedment and threatening stability. To combat uplift, greater vertical loading on the foundation was found to increase the resistance to monotonic and cyclic moments (Villalobos et. al, 2004). The use of caissons for OWT foundations have only recently been incorporated in commercial scale wind projects, but have advantages of low noise, quick installation procedures, and shorter embedment depths (Liu et. al, 2020).

1.3 Floating offshore wind

At the end of 2021, floating OWTs (e.g., Figure 2e,f) were generally considered a demonstration technology (e.g., comprising 0.2% of installed capacity in Europe, O'Sullivan et al. 2021). In addition to the complexities of mounting a large wind turbine on a floating structure, there are mooring, anchoring, array, and cabling considerations that become more complex as water depths increase. Regardless, the deep (> 60 m water depth) oceans present a vast and extractable energy potential many times greater than the projected global demand. This fact, combined with several offshore areas where the seabed conditions are not generally suitable for fixed foundations (e.g., Norway, Pacific Coast of the United States), means floating OWTs will become relevant likely within the decade. Regardless, for this report, the remaining focus will be on fixed bottom foundations. For information on floating OWT mooring and anchoring, see Musial et al. (2003).

2.0 Atlantic Canadian Inner Shelf Geology

The surficial geology of Atlantic Canada's continental shelves are primarily the product of ice advance and retreat centred on the previous glacial maximum (~ 20 000 years before present) and subsequent isostatic- and eustatic-induced relative sea level change. The thick ice sheet that advanced to the edge of the continental shelf for nearly all of the region (King and MacLean, 1976; King and Fader, 1986; Shaw et al., 2006; Dalton et al., 2020) resulted in widespread erosion, deposition of a blanket of glacially-derived till, and created much of the bedforms found on the continental shelf. At the ice margin on the Scotian Shelf, near the shelf break, large banks were formed as successive sandy, less consolidated tills were stacked with glaciomarine, glaciofluvial, and coastal sediments to form complex, deep banks of sediment (King, 2001). As ice retreated, relative sea-level changes resulted in a transgressive sequence of erosion and redistribution of sands and gravels through littoral processes (Shaw et al., 2002).

The four most common types of sediment found in the region are those typical of other previously glaciated areas and include reworked sand and gravel, postglacial mud, glaciomarine mud and glacial till. A basic summary of the geotechnical properties of these general soil types are presented in Tables 1 and 2 of Eamer et al. (2020). The general distribution of sediment thickness and type can lead to highly generalized regions where various foundation types could be employed, based on more thorough investigations of the inner shelf (e.g., Eamer et al., 2020; 2021). These regions are shown in Figure 3. The only likely location for widespread deployment of monopiles is the outer banks Sable Bank (region 12), due to thick packages of unconsolidated sands. Locations for GBF are concentrated in the shallow shelf areas of the Gulf of St. Lawrence (regions 2-5), where thin Quaternary sedimentation and shallow water depths are more conducive to that foundation type. Caissons may be suitable in the low relief, thick muds of Baie des Chaleurs (region 1), and there are a number of locations (region 22, but also smaller scale areas elsewhere) where multipile foundations (jackets) may be suitable.

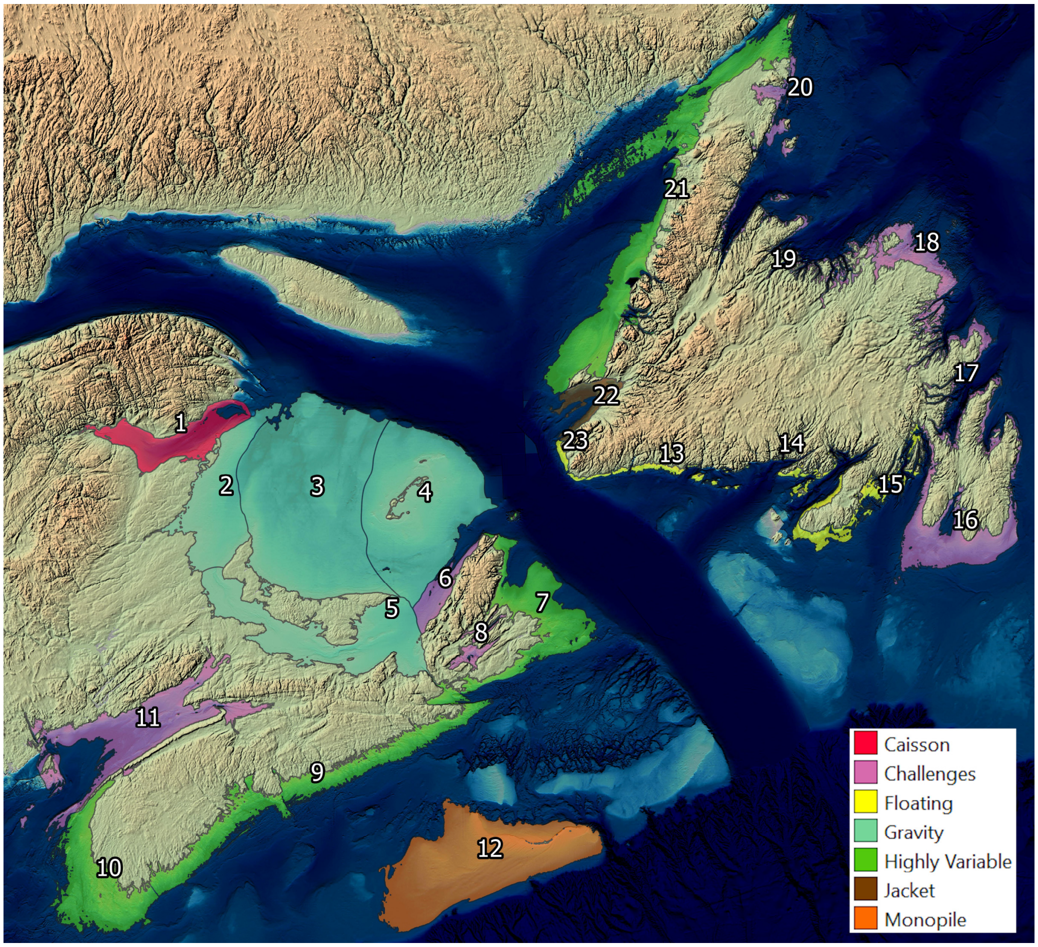

Figure 3. General regions of foundation types, based on analysis from Eamer et al., (2020). Regions where floating OWTs are indicated are based solely on a narrow inner shelf unable to support fixed foundations. "Challenges" regions are where currents, seabed rugosity, or sea ice/icebergs pose a challenge to any seabed infrastructure. Highly variable regions are difficult to ascribe to any one class. For region numbers, refer to Eamer et al., (2020).

3.0 Geotechnical parameters relevant for offshore wind energy

Geotechnical parameters are used to characterize soils according to their probable engineering behavior. Density, water content, and void ratio are elementary soil properties which characterize the state of the soils in the field. Grain size analysis and Atterberg limit tests are used to classify a soil and attribute to soils a label that represents their engineering properties – including permeability, compressibility and strength. Consolidation testing is used to measure the compressibility of soils in response to the application of loads. In the same test, it is feasible to derive maximum past stress (i.e., past load or overburden experienced by the sample) and it is possible to measure its hydraulic conductivity (permeability) relative to void ratio. Shear strength includes the soil properties (friction ratio and cohesion) that characterize the soil's ability to withstand lateral (shear) loads and is used in stability analysis of slopes. The elastic properties of the soil, including the soil's bulk and shear moduli, are used in the analysis of the soil's response to dynamic loads as experienced from the wind turbines.

Geotechnical characterization of the seafloor and subsurface sediment is required to determine the site suitability for infrastructure installation, from OWT foundation to cable landfall, and is subsequently used in design, development and deployment of the structures. The foundation characteristics are different for each piece of infrastructure installed, as well. As an example, an underwater cable may be more dependent on the upper soil mobility because the cable needs to be protected from lateral displacement and free spans that may result in breakage. In comparison, OWT foundations will be more reliant on the shear strength and bearing capacity (compressibility and friction angle) of the soil the foundation is to be emplaced in. Currently in Canada, there are no standardized or legislated testing regimes for a geotechnical investigation for an offshore wind turbine foundation; however, there are regulations on offshore oil and gas installations (SOR/96-118) and a standards document has been published for offshore renewables (Hill et al., 2015). In other jurisdictions, a variety of guidance and best practice documents exist, outlining recommended testing regimes for a geotechnical investigation for specific OWT foundations. In the following sections, common geotechnicial parameters used for characterizing offshore sediments for OWT foundation emplacement are outlined.

3.1 Grain Size Analysis

Grain size analysis includes the quantitative determination of the distribution of particle sizes in soils (ASTM D422). These distributions are often displayed on absolute or cumulative curves and bar graphs (e.g., Figure 4). Grain size analysis often requires different procedures. Sieve analysis is generally used for gravel and sand particles (> 75 μm). A sedimentation procedure or laser granulometer is used for silt and clay size particles (< 75μm). Sands and gravels can be classified into three main types: uniform, well graded and poorly graded using the uniformity coefficient (C_u) and the coefficient of curvature (C_c). C_u and C_c are calculated using

$$C_u = \frac{D_{60}}{D_{10}} \tag{1}$$

$$C_c = \frac{(D_{30})^2}{(D_{10})(D_{60})} \tag{2}$$

where D_{10}, D_{30}, D_{60} are the finest grain diameter (mm) of the coarsest 10 %, 30 %, and 60 % of grains, respectively. The distribution of the sands and gravels and the C_u and C_c are used as part of the USGS classification of soils (ASTM D2487).

The distribution of sediment particle size for a given soil or stratigraphic layer is most often determined from samples obtained from discrete sampling of the seabed or from subsamples obtained from sediment cores. Variations in grain size can be used to identify stratigraphic boundaries, for which geotechnical parameters can be broadly applied. This is relevant to offshore wind development as underlying stratigraphic heterogeneity can affect the viability of a given foundation type. An example of this relevant to the Nova Scotia inner shelf is (typically buried) fine grained organic terrestrial sediments, drowned after relative sea-level rise during the Late Pleistocene and Holocene, are highly compressible due to the organic component and could pose a risk to foundation stability (Eamer et al., 2020). In addition (although rarely identified through laboratory analyses but rather through geophysical methods), large clasts such as cobbles and boulders at depth may cause refusal for driven pile foundations. These sediments are common in glacial till and occur in glaciomarine sediments as dropstones. Another key consideration is susceptibility to scour, the process where currents erode sediment around and under the foundation. The relationship between grain size and scour susceptibility is non-linear, in that cohesive sediments (sediments with > 50% silt and clay by weight, particles that adhere to each other) erode differently than non-cohesive (granular) sediments (Whitehouse et al. 2011; Harris and Whitehouse 2017). The latter, however, can generally be described as less erodible as grain size increases (Mirtskhoulava 1991). Often, scour mitigation for offshore wind foundations is composed of large boulders, exemplifying this relationship. The former (cohesive sediments) can often be less erodible, but the relationship depends on many additional variables beyond grain size (Harris and Whitehouse, 2017).

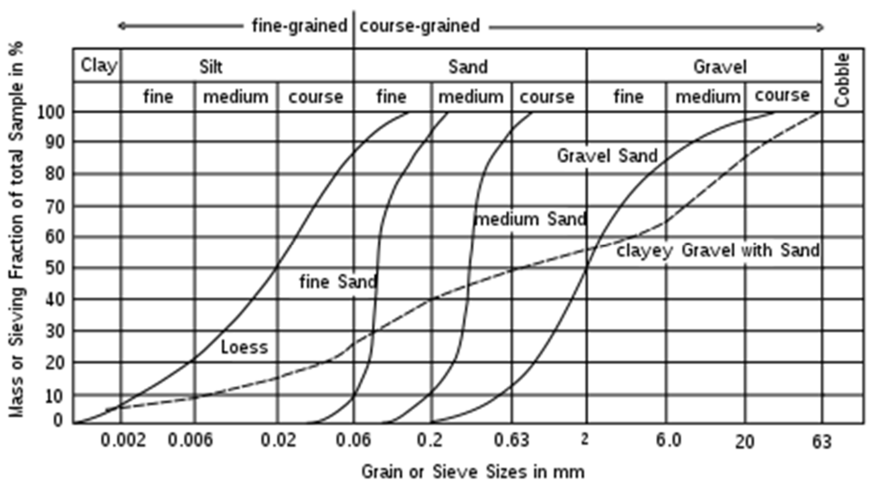

Figure 4. Example cumulative grain size distribution curves for various soil types.

The foundation types used in offshore wind projects each behave differently in different soil conditions. Monopiles and driven piles are typically installed in both sandy and clayey soil, and can be installed in certain semi-consolidated sedimentary rock types (CFMS, 2019). GBFs require a marine soil with a high bearing capacity that is capable of supporting the large vertical loads. As a result, dense sands and coarser sediments, as well as bedrock, are considered the most viable seabed type for GBFs (Alonso, 2013; Esteban et. al, 2019). Lastly, suction caissons are best deployed in both sands and clays (Iskander, et. al, 2011). The two sediment types are capable of resisting the pullout of the caisson and supporting the lateral and vertical loads of a wind turbine (Iskander et. al, 2011). Overall, a grain size distribution provides important index parameters for engineering classification of marine soils and enables initial assumptions about other geotechnical parameters based on the makeup of the soil.

3.2 Density

Soil density can be qualified in different ways, each having a different application. The bulk density (ρ_b) is the ratio of the total mass (M_t) of the soil to the total volume (V_t) of the soil. The dry density (ρ_d) is the ratio of the mass of the dry soil (M_d) to the total volume (V_t) of the soil. The particle density (ρ_s) if the ratio of the mass of the solid soil particles to the volume of the soil particles (V_s). The bulk, dry and particle densities are calculated using:

$$\rho_{b,d,s} = \frac{M_{t,d,s}}{V_{t,t,s}} \tag{3}$$

A number of methods exist for measuring bulk and dry density. Typically, direct measurements of the mass and volume of a sample and water displacement are used, the latter deriving volume from the amount of water displaced when a sample is submerged. At GSCA laboratories, bulk density is measured using the Gamma Ray Attenuation method and discrete constant volume samples (Section 5.2.3). Dry density is determined from the constant volume samples. Particle density is determined using the gas pycnometer method (ASTM 5550).

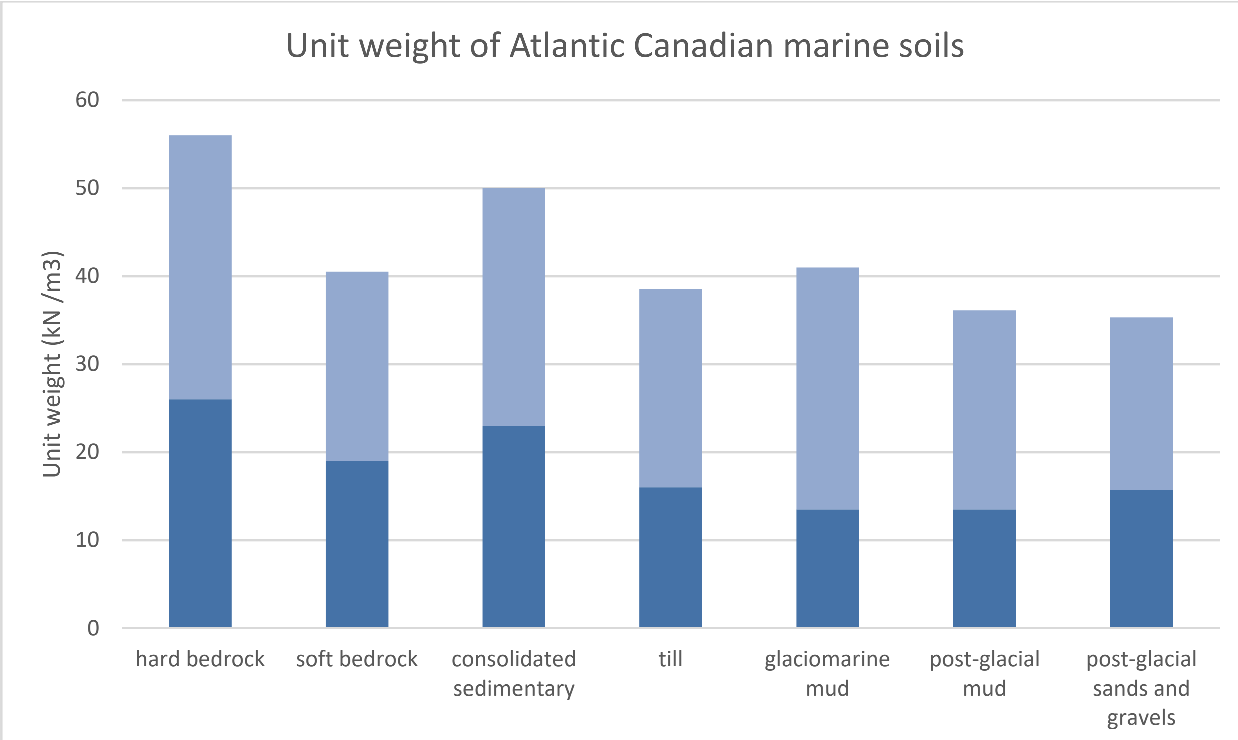

Figure 5. Unit weight (force per unit volume) values for various seabed materials found in Atlantic Canada. Upper and lower limits shown by the upper limits of light and dark blue, respectively. Data from Eamer et al. (2020, Table 2).

The bulk, dry and particle densities of marine soils are key properties for understanding the composition, deposition, and deformation history of marine soils. The density of a given marine soil type can vary greatly, even within regions (Figure 5), with implications for the emplacement and long-term stability of marine infrastructure. Soil density is an important characteristic for all offshore infrastructure. Soils with high densities are correlated with having higher internal angles of friction (Section 3.4) and corresponding unit weights, which results in an increased bearing capacity (Koekkoek, 2015), important for GBFs. The suitability of sandy marine soils is dependent on bulk density, whereas the suitability of fine-grained soil (clays and silts) is more a function of shear strength and stress state (Russel, 2020). For piled foundations and caissons, the density of soil affects both the performance and ease of installation. Piles driven into dense sands, while requiring more installation time and force, have been shown to perform better in comparison to those driven into loose sands. Dense sands experience less deflection at the mudline in comparison to loose sands. Also when the same monotonic load is applied, loose sands show signs of plastic deformation resulting in greater likelihood of failure due to rotation (Black et. al, 2016). Furthermore, when a pile undergoes cyclic loading, common for OWT foundations, the dense sand is less likely to redistribute as a result of the movement. Suction caissons are affected by density in a similar manner to driven piles. Dense marine soils can pose a greater issue for the installment of caissons as they rely on their weight and suction to penetrate the soil instead of being driven. Sands of low to medium density are susceptible to liquefaction under cyclic loading and exhibit a very low shear strength, increasing the likelihood of failure (Liu, 2020). Dense sands are generally more resistant to scour, as well, primarily due to grain friction and reduced porosity.

3.3 Water Content & Atterberg Limits

The water content of a soil is one of the most important properties in determining the engineering behaviour of a fine grained soil. The water content of soils is the ratio of the mass of water (M_w), to the mass of solids (M_s). Since the pore fluid is saline in marine soils, a correction is required to account for the dissolved salts. The water content (*W*) for marine soils is calculated using

$$W = \frac{M_w}{M_s - r(M_t)} \tag{4}$$

where *W* is expressed as a percentage, *r* is the fluid salinity, assumed to be 0.0035, and M_t is the total mass of the sample. Water content is easily measured (ASTMD2216) by calculating the weight of water removed from the soil by oven drying.

The consistency or state of a fine-grained soil is altered by changes in the water content which in turn affects its engineering properties. For each soil there is a range of water contents where a fine-grained soil can exist in a solid, semisolid, plastic or liquid state. The water content at the boundaries between theses states define important limits of engineering behaviour and are called Atterberg limits. The Atterberg limit test (ASTM 4318) is a standardized procedure to estimate the plasticity of the soil by measuring its plastic and liquid limits. The liquid limit (*LL*) is the water content at which the soil transitions from a plastic to liquid state. The plastic limit (*PL*) of a soil is the water content at the transition between the plastic and semi-solid state. The water content range between the *PL* and *LL* is known as the plasticity index (PI) calculated as

$$PI = LL - PL \tag{5}$$

and is a measure of the plasticity of a fine grained soil.

Atterberg limits are used in the USGS engineering classification (ASTM D2487) of fine grained soils that allows for the evaluation of its engineering behaviour (Figure 6). Inorganic soils of high plasticity generally have higher cohesion, lower permeability and higher compressibility than low plasticity soils. Organic soils, in particular organic clays—designated as OH—have very high compressibility and low shear strength that reduces stability and hence are not desirable for engineering purposes. Seed et al. (2003) uses the water content of the sediment (Wc), and plasticity chart to identify sediments that are potentially susceptible to cyclic liquefaction (Zone A), sediments requiring further testing to evaluate liquefiable potential (Zone B) and sediments not susceptible to liquefaction (not in Zones A or B). Atterberg limit tests (ASTM D4318)) are typically done in a geotechnical lab using marine sediment samples collected from cores.

The state of a fine grained soil cannot be defined solely by its water content. Two different soils with identical water content may have quite different characteristics. It is preferable to “normalize” the water content of a soil by relating it to its liquid and plastic limits using the liquidity index (*LI*), calculated by:

$$LI = \frac{(w - PL)}{PI} \tag{6}$$

where LI = liquidity index, w = natural water content of the soil, PL = plastic limit, and LL = liquid limit. The compressibility of the sediment decreases and strength increases and as the LI decreases from > 1 (liquid state) to < 0 (semisolid state). LI also approximates a soil's stress history (See section 3.5). A soil is considered to be normally consolidated if LI =1, underconsolidated if $LI>1$ or overconsolidated if $LI <1$).

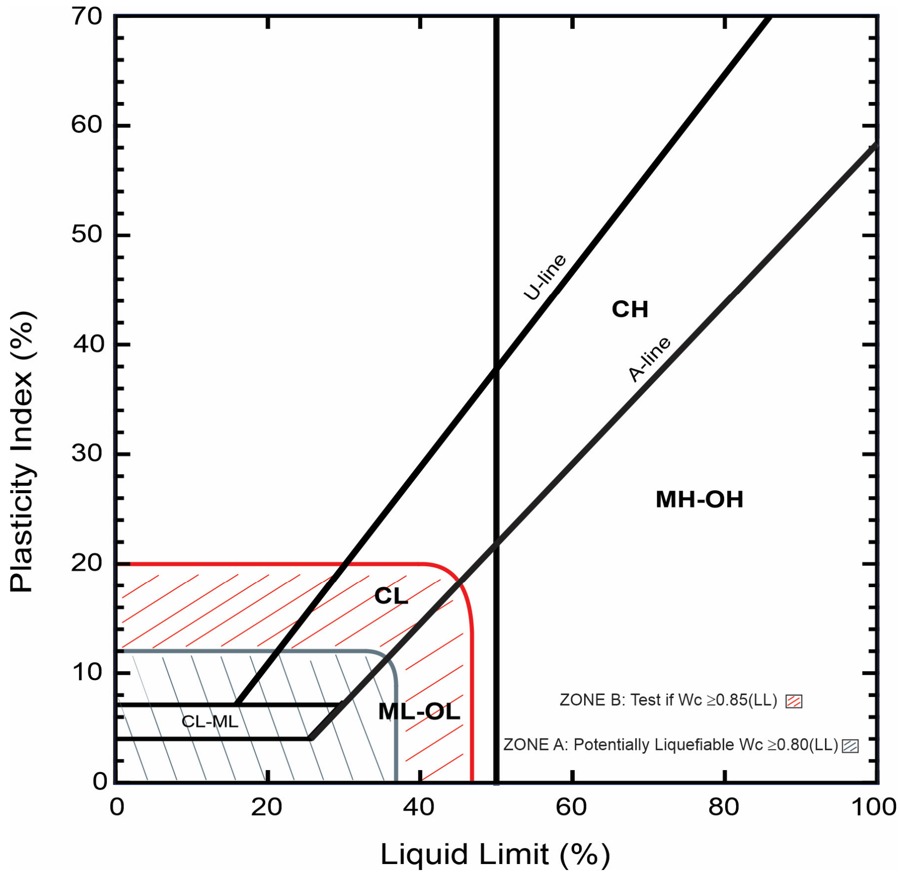

Figure 6. Plasticity chart, adapted from ASDM D2487. The A-line (PI = 0.73(LL-20) separates claylike (C) sediments from silty (M) sediments and also organic rich sediments (O) (clay or silt) from inorganic sediments. The U-line (PI = 0.9(LL-8) indicates the upper plasticity index and liquid limit values for all sediments. The chart is further subdivided on the basis of low (L) and high (H) liquid limits.

Understanding the *in-situ* state of marine soils is important when planning and designing aquatic structures such as foundations. In nature, undisturbed soils with water contents greater than the liquid limit (i.e. LI >1) do not exist in a liquid state and may have a high shear strength. However if failure occurs the remolded soils may have low residual strength and even behave as liquids in extreme cases with the potential for significant seabed instability. Generally, the higher the water content is above the liquid limit, the larger the potential for deformation. The water content and

Atterberg limits can be used to evaluate the behaviour of the soil under cyclic loads and are particularly relevant in areas that are seismically active. Low plasticity, cohesionless fine soils with water content close to or above the LL are susceptible to liquefaction under cyclic loading. During this process the soils experiences a loss of strength and stiffness and increase in pore pressure. When an offshore wind foundation is installed, its design is dependant on the soil behaving in a specific way in response to the lateral and vertical loads applied. The liquid limit of marine soil is also correlated with the compressibility (Section 3.5) of clayey marine soils, particularly important for GBFs (Fugro Marine GeoServices, 2017).

3.4 Shear Strength

The shear strength of a soil is the maximum resistance a soil displays against failure and is directly controlled by its effective stress. Several sources contribute to a soil's shear strength, but the dominant factor is gravitational compaction and the resultant reduction of pore space under the weight of overburden sediments. Other sources of strength include bioturbation, desiccation, particle bonding (effective cohesion) and ultimately lithification (diagenesis). The most common failure criteria applied to soil is the Mohr-Coulomb failure criteria, defined as:

$$\tau_f = c' + \sigma_n' \, tan\phi' \tag{7}$$

where τ_f is the drained shear stress at failure, σ_n' is the effective normal stress, c' is effective cohesion and ϕ' is effective internal angle of friction. Note that the effective normal stress, σ_n' is defined as

$$\sigma_n' = \sigma_n - \mu \tag{8}$$

where σ_n is the total normal stress and μ is the pore pressure of the pore fluid. Equation 7 is used to estimate τ_f for effective stress or long term stability analysis when μ is only hydrostatic and $\sigma_n' = \sigma_n$.

The relationship between the shear stress (τ_f) and the effective normal stress (σ'_n) at failure define the Mohr-Coulomb strength parameters, c' and ϕ' (equation 7) and the Mohr-Coulomb failure envelope defines the stresses at which failure occurs. At stress conditions below the failure envelope, the soils are stable. Factors that affect Mohr-Coulomb strength parameters include grain size distribution, angularity of the particles, relative density and stress history. Generally, well sorted sands and gravels have higher friction angles than clays or poorly sorted soils. The stress history for sands have little effect on the strength of sand. However the strength of fine grained soils is dependant on the soil's stress history. The same soil with the same present σ_n' could have different strength parameters due to their stress history. Generally, over-consolidated clays have higher c' and ϕ' values.

Typically, engineering loads are applied much faster than water can dissipate from the pores in fine grained cohesive soils. Loading thus results in excess pore pressure which affects effective stress of the soil (see equation 8). In this condition, equation 7 is rewritten in terms of total stresses (undrained conditions), as

$$S_u = c_t + \sigma_n \tan\phi_t \tag{9}$$

where S_u is the undrained shear strength, c_t is the total cohesion and ϕ_t is the total friction angle. If a soil is saturated and undrained, then $\phi_t = 0$ because newly applied loads are carried entirely by the pore pressure and $S_u = c_t$.

The most common laboratory method to determine Mohr Coulomb strength parameters are triaxial tests, which can be conducted using a variety of conditions including consolidated/unconsolidated or drained/undrained. Triaxial test are performed on a cylindrical specimen of soil that is placed in the triaxial cell with an axial load applied in compression or extension to the top of the specimen. Triaxial tests allow for different consolidation and drainage conditions in order to best represent the *in-situ* nature and subsequent response of the soil. The three main types of triaxial tests are unconsolidated undrained, consolidated undrained, and consolidated drained. The direct shear test is used to determine the shear strength of soils on a predetermined failure surface. The soil sample is confined in a metal box consisting of 2 halves and subjected to a normal load (σ_n). The shear stress required to slide one half of the box relative to the other half in a horizontal direction is measured.

A common laboratory method for the determination of undrained shear strength (S_u) of fine grained soils is the laboratory miniature vane shear test (ASTM D4648). Mini-vane shear measurements provide an indication of the undrained shear strength in saturated fine grained soils. Vane shear tests are quick tests and require minimal equipment and as such are often done on a sampling vessel as a rapid initial measurement of undrained shear strength on core samples. *In-situ* methods also exist for determining the undrained shear strength of a marine soil, most commonly cone penetrometer testing (CPT), which can be performed downwards from the seabed or at intervals down a borehole. Seabed CPT testing can penetrate multiple meters of soil and provide a continuous undrained shear strength profile of the soil (ASTM D5778).

The shear strength of marine soils is a key parameter for development of any offshore wind project and is essential in determination of the viability of an OWT design. Shear strength governs how a soil will react when a load is applied and at what point the load is too great and failure is likely to occur. Both the undrained and drained shear strengths of a soil are relevant for OWT foundations, as during installation or after emplacement, forces applied to the soil can cause an undrained marine soil to become partially drained or drained. Drained and partially drained conditions can also result from the dynamic load exerted on OWT foundations (Oh et. al, 2018).

For GBFs, shear strength is used for stability analysis in marine environments (Gaard, 1982). A stability analysis for a GBF must consider the effect of short term (undrained) and long term (drained) pore pressure changes on shear strength parameters caused by the installation of the GBF. (DNV, 2018). The initial installation process results in an increase in pore pressure which dissipates over time. Therefore the initial analysis uses undrained shear strength while the long term condition uses drained shear strength based on effective stress. The dynamic analysis of GBF can use a continuum approach for homogenous soils and finite elements for non-homogenous soils (DNV, 2018). For suction caissons, pullout forces and the associated withdrawal of the caisson from the seafloor is a significant design consideration. The pullout capacity of caissons is dependant on the shear strength of the marine soil and the frictional side shear capacity between the caisson and soil interface (Iskander et. al, 2011). In piled foundations, shear strength is a required geotechnical parameter for soil pile interaction calculations and modeling to ensure that the OWT remains within the tolerances for displacement and deflection (Leblanc et al. 2010; Fugro Marine GeoServices, 2017). Shear strength is used to define plastic deformation limits, deformation that can occur due to dynamic loading on piles which can cause degradation in the shear properties of the surrounding soil (Oh et. al, 2018). The friction angle is typically used in soil response equations such as p-y curves. P-y curves describe the relationship between the soil resistance p as a result of the lateral pile displacement y at any depth along the pile below the mudline (Leblanc et al., 2010), but have proven inadequate for offshore wind infrastructure due to load profiles, foundation dimensions, dynamic site-scale characterization, and specificity (Byrne et al., 2017).

Given the wide range of shear strengths found in Atlantic Canada's offshore, even within common geological units (Figure 7), initial site characterization for this parameter is key to foundation selection and siting.

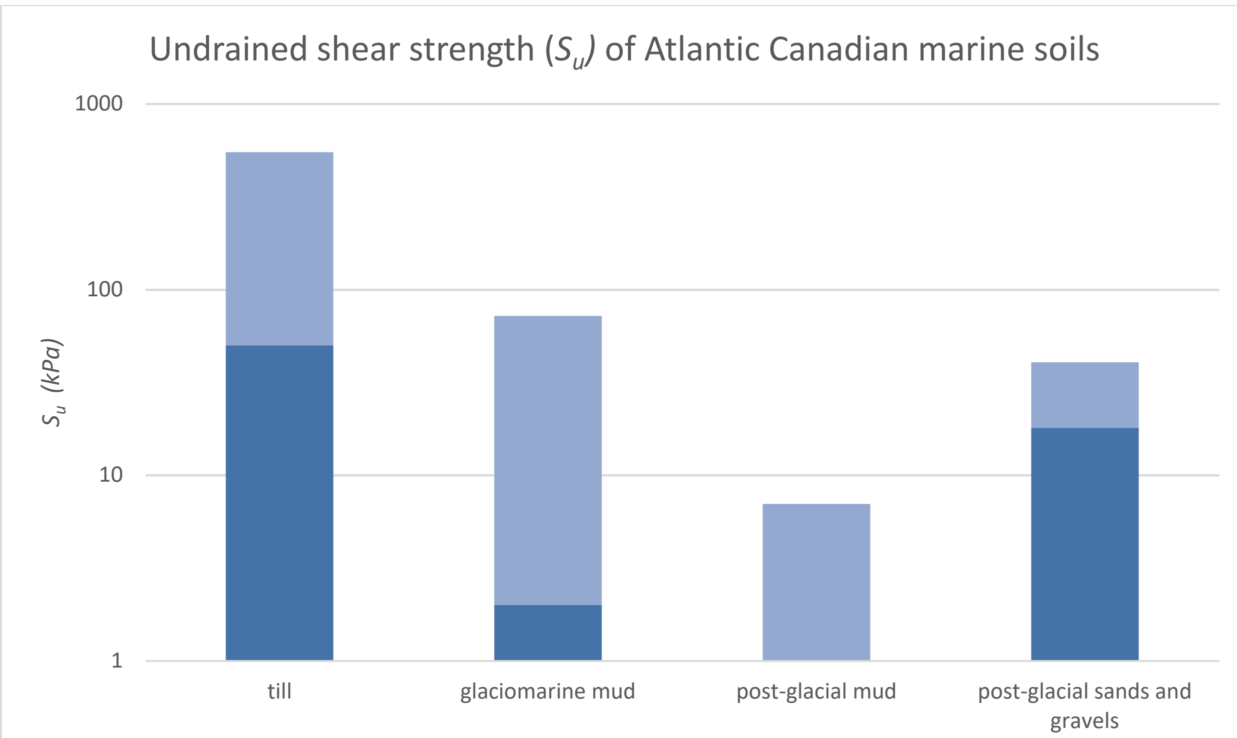

Figure 7. Undrained shear strength for various seabed materials found in Atlantic Canada. Upper and lower limits shown by the upper limits of light and dark blue, respectively. Note the logarithmic scale. Data from Eamer et al. 2020, Table 2.

3.5 Consolidation Parameters

The consolidation of a soil occurs under an applied load which results in the expulsion of water from the soil and a corresponding decrease in volume. The compressibility and rate of consolidation are two of the most important soil characters used in foundation design and depend on the soil's composition, grain size distribution, permeability and stress state. Consolidation characteristic are described using a variety of parameters including compression index (C_c), coefficient of volume compressibility (m_v), coefficient of consolidation (C_v), and overconsolidation ratio (OCR). These parameters govern how the soil will react under applied loads (Fugro Marine GeoServices, 2017).

The relationship between effective stress and void ratio is used to obtain the stress state of the soil (OCR), compression index (C_c) and recompression index (C_r) of the soil. The OCR is the ratio between the current stress applied on a soil and the maximum stress it has experienced. A normally consolidated soil is one currently under its highest geologic stress. An over-consolidated soil is one that experienced a higher geologic stress prior to present day. Conversely, a soil can be considered underconsolidated if a load is applied and excess pore water pressure has not yet dissipated, such as after foundation emplacement, the installation of scour protection, or in an area of rapid sediment deposition (although these effects are temporary). The compression index is used to estimate the degree of settlement expected under a given load. The relationship between soil deformation and time is used to calculate coefficient of consolidation (C_v) and estimates the rate of pore pressure dissipation under applied load. The laboratory test used to measure the one dimensional compressibility of a soil is a consolidation test (ASTM D2435M).

With regards to OWT foundations, compressibility and rate of consolidation is most important for the construction of GBFs, as they exert the greatest vertical load on the underlying soil. Geologic changes resulting in variable C_c values can cause differential settlement which can cause distortions within a structure. Over-consolidated soils are preferred for OWTs as they behave in a more elastic manner and result in less post-installation settling (Le et. al, 2014). For pile and caisson foundations, highly consolidated soils may result in increased skin friction during installation. This increase in friction can slow down the installation of the pile/caisson significantly. Coefficients found from consolidation testing can help estimate this effect. (Fugro Marine GeoServices, 2017).

3.6 Permeability

The coefficient of permeability (k), also known as the hydraulic conductivity, represents the rate of flow of water through a soil in standardised conditions. Permeability is highly dependant on the soil's grain size distribution, and grain packing arrangement, which is in part a function of grain shapes and style of deposition as well as stress history. Generally, coarser soils, such as sands and gravels, have high permeability as water can easily flow through the large voids between the particles. Conversely, fine grained soils, such as clays and silts, generally have low permeability. Similarly, soils that are poorly sorted tend to have lower permeability due to fine particles filling voids between larger particles. Dense soils tend to have lower permeability because grains are packed more tightly together, which reduces pore space and pore interconnectivity.

The most common tests to measure permeability for coarse and fine grained soils are the constant and falling head tests, respectively. Permeability can also be derived using the rate of deformation during consolidation tests. The permeability of marine soils is highly relevant for

GBFs and suction caisson foundations, especially in clays and silts where excess pore water pressure can develop when loads are applied. Excess pore water pressure in fine grained soils will dissipate slower than in coarse grained soils and will cause long term settlement patterns that may be spatially and temporally irregular. For GBFs, the foundation exerts a large vertical load on the seafloor sediment and it is important to evaluate the sediment short and long term response to this laod. For suction caissons, permeability is critical in determination of the suction pressure needed to install the caisson (Fugro Marine GeoServices, 2017). If too much suction pressure is applied, channels may form in the soil that surrounds the caisson which disrupt the viability of installation. Shear strength is also dependent on permeability through pore water pressure. Triaxial tests can be used to determine the effect decreased permeability and subsequent increases in pore water pressure. has on a soil's shear strength, and thus stability.

3.7 Shear Modulus

The shear modulus is a property used to characterize the dynamic stiffness of a soil and is the ratio of the shear stress to shear strain:

$$G = \frac{\tau}{\gamma} \tag{10}$$

where τ is the shear stress and γ is the shear strain.

At low shear strains within the elastic range the shear modulus measurements are linear and the shear modulus is considered to be an elastic parameter and termed G_{max}. Under large shear strain ranges, where plastic deformation occurs, shear modulus measurements show a large decrease with increasing strain and is termed *G*. The stress and strain levels therefore must specify the type of measurement involved. Soils hosting OWT foundations can experience small elastic strains G_{max} and larger strains in the regimes outside the elastic range, *G*. G_{max} and G can be related using

$$\frac{G}{G_{max}} = \frac{1}{1 + {}^{\gamma}/_{\gamma_a}} \tag{11}$$

where γ = the shear strain and γ_a = the reference strain, a constant dependent on the *PI* (equation 5) of the soil.

The G_{max} shear modulus of marine soils is generally evaluated through the measurement of shear wave velocity using seismic CPT tests (ASTM 7400) in the field and bender elements during laboratory triaxial tests. G_{max} is calculated using

$$G_{max} = \rho_b V_s^2 \tag{12}$$

where ρ_b is bulk density and V_s is the shear velocity.

Simple shear and hollow cylinder tests are also used to measure shear modulus. Shear velocity and thus G_{max} is a function of void ratio and is therefore dependant on confining pressure (i.e. depth below seafloor), composition and stress history. A relationship between G_{max} void ratio and stress history is presented in Oh et al. (2018) as

$$G_{max} = 625 \frac{\mathrm{OCR}^k}{0.3 + 0.7e^2} \tag{13}$$

where OCR = the overconsolidation ratio, *e* is the void ratio and *k* is a constant between 0 and 0.5 and is dependent on the *PI* (equation 6),

A profile of G_{max} (Oh et al., 2018) can be estimated as a function of depth using

$$G_{max}(z) = Az^{\alpha} \tag{14}$$

where *A* is a derived factor for cyclic or dynamic loading and is dependant on OCR, *e*, and the *PI* and α generally ranges from 0.4 – 0.6. An example of a modulus profile as a function of depth on Georges bank, just south of Atlantic Canada's shelf, calculated with equation 14 as $G_{max} = 88.44z^{0.3322}$ and compared to measured values for G is presented in Oh et al. (2018, figure 8c).

The shear modulus is intrinsically related to shear strength, and the degradation of a soil's shear modulus is governed primarily by shear strain (Oh et al., 2018), as such it can be estimated by the amount of movement in the foundation. Dynamic loads from wind, wave or earthquakes can cause degradation to a soil's shear modulus and a loss of stiffness, thus itis a relevant factor to consider (Oh et al., 2018).

4.0 Geotechnical profiles

The integration of the geotechnical data with associated geological interpretations allows for a better understanding of the geotechnical data and is critical in developing an accurate geotechnical characterization of the sediments. The geotechnical profile is commonly used to correlate the geology with the geotechnical properties. Figure 8 is an example of a geotechnical profile drawn from an Arctic Canadian piston core illustrating the variations in the lithological and geotechnical data with depth below seabed. The geotechnical data presented includes grain size, bulk density, natural water content, Atterberg limits water contacts, liquidity index, undrained miniature laboratory shear strength and the stress state of the sediments.

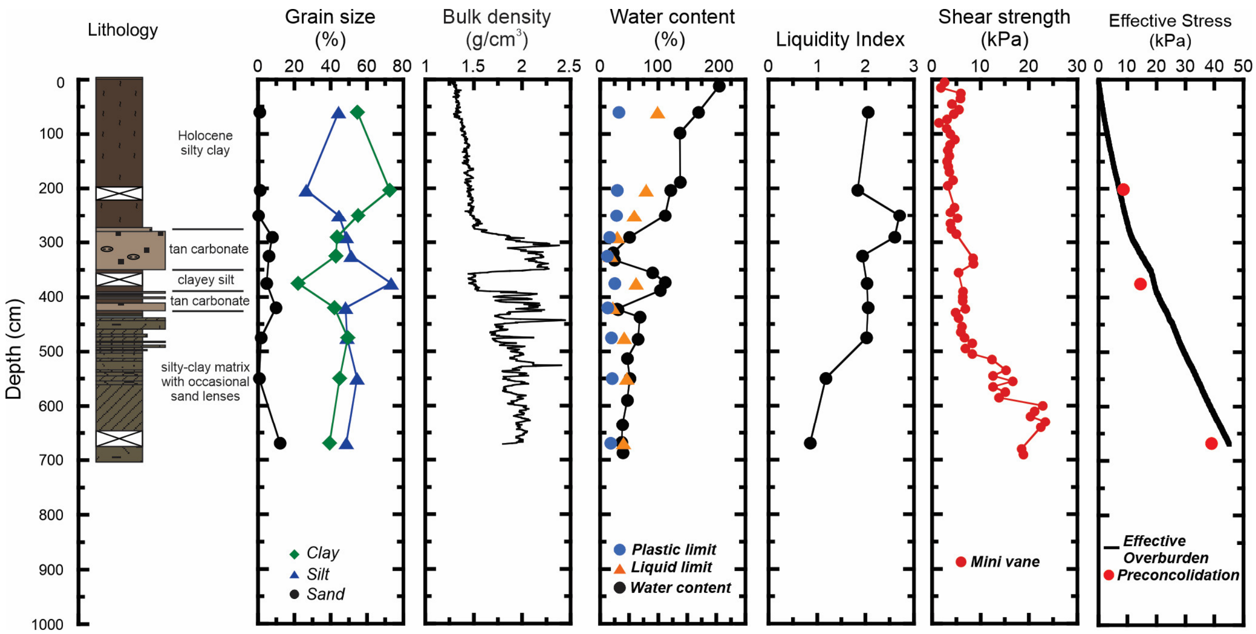

Figure 8. Geotechnical profile from an Arctic Canadian piston core illustrating the link between changes in lithology (geology) and geotechnical properties.

5.0 Relevant capacity of the Geological Survey of Canada – Atlantic

5.1 Existing geotechnical data on Canada's Atlantic shelf

Decades of marine seabed sampling conducted by GSC-A and collaborators have been compiled in the Expedition Database, available online (https://ed.gdr.nrcan.gc.ca/index_e.php). This database can be queried and contains grain size data and geotechnical data, where available. A compilation of legacy geotechnical data compared with interpreted geology is underway, which should lead to a further refinement of the expected ranges of parameters for each geologic unit.

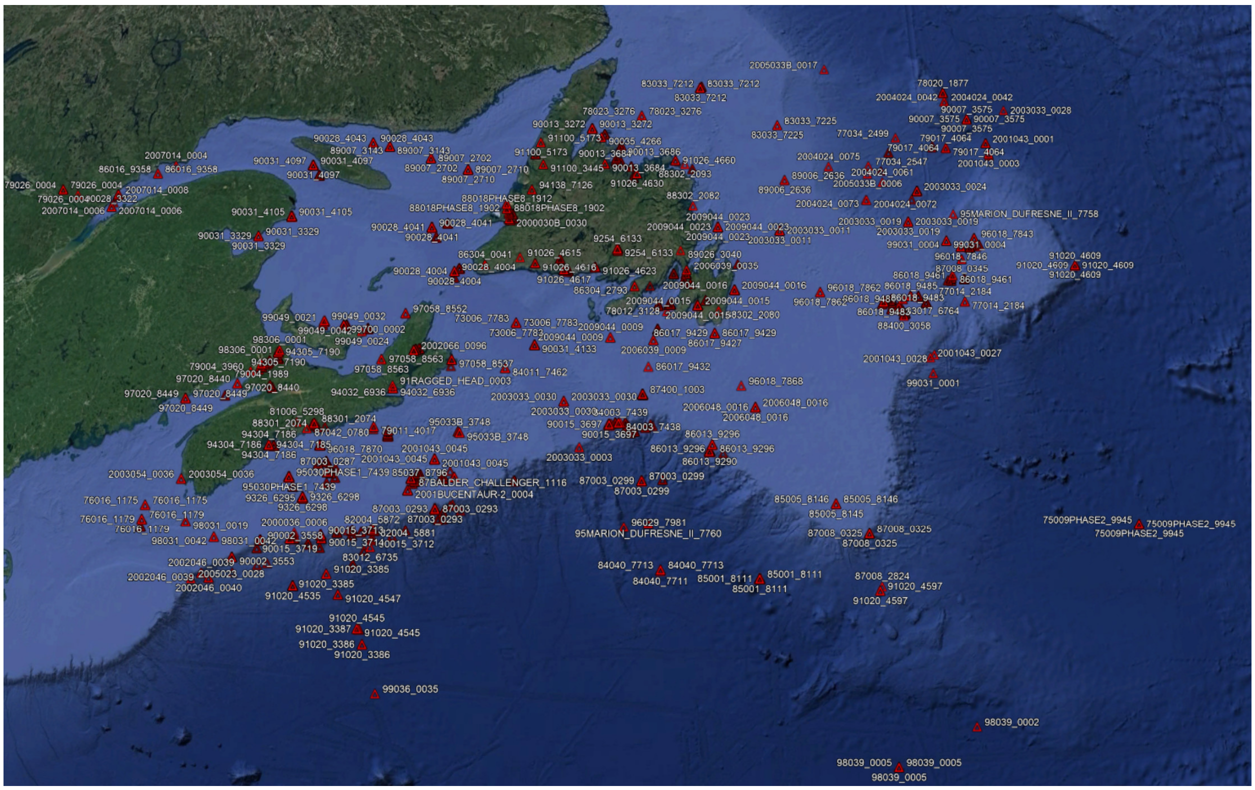

Figure 9. Grain size data in the Expedition Database for the Atlantic Shelf. Imagery - Google.

Generally, the bulk of data relevant to foundation emplacement is grain size data, covering most of the Atlantic shelf (Figure 9). Exceptions include:

- Sable Island Bank, offshore Nova Scotia, where an extensive borehole program was undertaken leading to data found in King (2001) and Eamer et al. (2020; 2021)
- Canso Strait, between mainland Nova Scotia and Cape Breton, where a broad geotechnical study was undertaken (Brown and Rashid 1975).
- The Grand Banks and Halibut Channel, offshore Newfoundland, where borehole data is published in Moran et al. (1990), Mosher and Sonnichsen (1992), and Miller (1996).

5.2 Capabilities of GSCA laboratories

5.2.1 Core Processing Lab

The Geological Survey of Canada-Atlantic (GSC-A) has developed high-resolution methods for physical property analysis of piston and gravity cores. Core processing is performed systematically, following a series of procedures that ensure all appropriate measurements are made as optimally as possible.

A non-destructive multi-sensor core logging system (Figure 10) manufactured by GEOTEK measures bulk density, magnetic susceptibility, electrical resistivity and acoustic compressional wave velocity on split core sections. Data can be measured at a resolution of 1 cm.

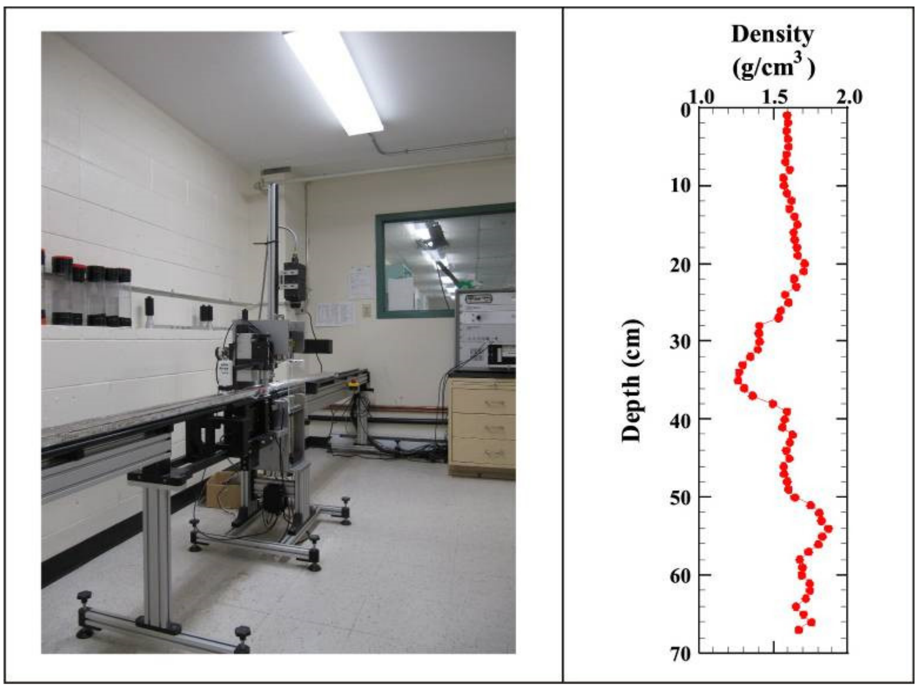

Figure 10. Multi-Sensor Core Logger and density profile. Photo by K. MacKillop, NRCan photo 2021-704.

Two automated miniature laboratory vanes (Figure 11) are used to measure undrained shear strength. The core processing time is reduced when making 2 undrained shear strength measurements simultaneously.

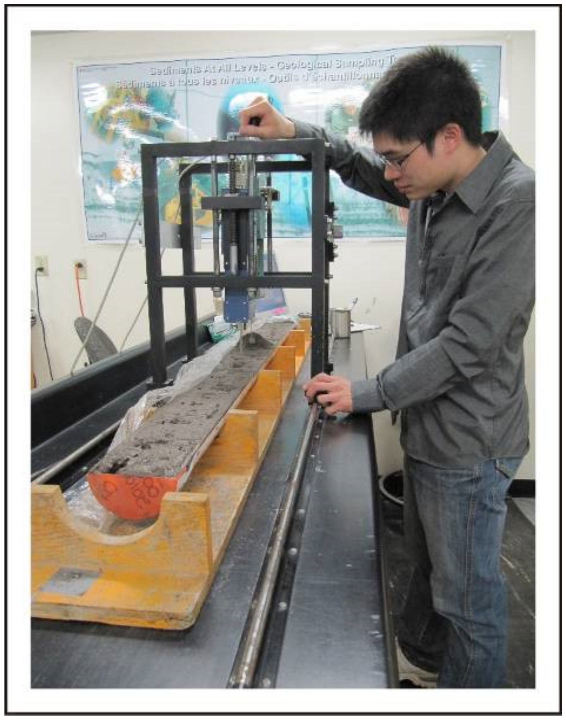

Figure 11. Laboratory miniature shear vane system. Photo by K. MacKillop, NRCan photo 2021-705.

5.2.2 Marine Sediment Lab

The standard focus of this laboratory is to provide a range of high resolution grain size analyses using a suite of size spectra measuring technologies. In addition to performing grain size analyses, this lab plays a vital role in support services such as:

- calibration of sensors deployed in the marine environments to monitor/model sediment mobility
- light/heavy mineral separation of >63 <180 micron particles utilizing Sodium Poly Tungstate solutions (the methodology involves centrifuging for separation, liquid nitrogen for isolation and vacuum filtering for collection)
- preparation of sediments for foraminifera identification

The grain size lab is also utilized by local and international graduate students from a variety of marine disciplines who are trained and supervised by the lab manager to operate grain size instruments and develop methodologies for collecting data. This directly benefits the science programs at the GSCA, BIO and the marine research community as a whole.

Instrumentation at the sediment lab includes:

Beckman Coulter LS13320 Laser Diffraction Analyser w/ Autosampler

- rapid down core summaries of split-core (technology in use since 2002, 780 nm wavelength)

- Ideal also for treated and freeze dried mud fractions
- Dedicated computer using Beckman Coulter LS230 Software

Horiba Retsch Camsizer Dynamic Imaging Particle Size Analyser
- grain size analysis (range 30 to 30,000 microns) but primarily for sand and gravel
- Dedicated computer
- Capable of providing shape analysis information – (shape analysis to be developed)

Manual Sieves
- Wet washing of mud fractions from >53 or 63 um fractions and analysis of the gravel component; done for all particles > 1 mm at ¼ PHI intervals
- Any fractional sieving required for various types of sample preparation (from % gravel, sand & mud to forams, heavy mineral, petrographic size ranges)

Sorval EGFX Centrifuge (w/ 4 * 750 ml swing bucket rotor for use with or without inserts)
- used to prepare <53 um samples for SediGraph; <63 um samples for freeze drying; heavy mineral and foram separation (purchased, 2011)

IEC GP6 Refrigerated Centrifuge (w/ 4 * 750 ml swing enclosed bucket rotor for use with or without inserts)
- used to prepare <53 um samples for SediGraph; <63um samples for freeze drying; heavy mineral and foram separation

Labconco Feezone 6 cart-mounted mobile freeze dry system

Femto's Particle Sizing Software Sizing System (PSS) Windows 10 Compatible Version 5.6
- hardware and software tool used for collecting, editing, processing, and presenting data related to grain size distributions of gravel, sand, silt mixtures (hardware delivered with the system consists of the settling columns).
- system enables the processing and merging of grain size spectrums analyzed by laboratory sieves; SediGraph; settling tubes; Camsizer and laser diffraction. Data is imported from external sources into PSS, processed at 10th phi intervals and merged at 5th phi intervals. % sortable silt and mean sortable silt size is also reported.

5.2.3 Geomechanical Lab

The GSC-A geomechanical laboratory has 2 fully automated stress-path triaxial systems manufactured by GDS Instruments (Figure 12). The system is based on the classic Bishop and Wesley-type stress path triaxial cell, and the GDS pressure/volume controller. The system can run such advanced tests such as stress paths, slow cyclic loading and K_0, all under computer control.

Figure 12. The GDS stress-path triaxial system. Photo by K. MacKillop, NRCan photo 2021-526.

An electromechanical dynamic triaxial loading system (ELDYN) was manufactured by GDS and allows for cyclic loading of triaxial samples under strain or load control while monitoring pore pressure (Figure 13). The ELDYN is based on an axially-stiff load frame with a beam mounted electro-mechanical actuator. The maximum axial load capability is 10 kN at 5 Hz. As well as dynamic triaxial tests, the ELDYN system can be utilized to carry out traditional triaxial tests such as UU, CU and CD as well as more advanced tests such as stress paths, K0, resilient modulus and creep tests. Due to the high precision electro-mechanical actuator the ELDYN system supersedes most systems using pneumatic actuators in terms of costs and overall useable performance.

Figure 13. The GDS electromechanical dynamic triaxial loading system. Photo by K. MacKillop, NRCan photo 2021-706.

There are two bender elements (Figure 14) sets manufactured by GDS that can be used in the stress-path and ELDYN triaxial systems. The bender elements enable measurement of the maximum shear modulus (G_{max}) of a soil at small strains in a triaxial cell. The bender element system is made up of bender element inserts, adapted top cap and pedestal and an external USB control box. The bender element software allows for a source signal type of sine wave, square wave and user defined wave shape. The shear velocity *vs* confining pressure is routinely measured during triaxial testing.

The GSC-A geomechanical laboratory is also equipped with 2 fully instrumented, back-pressure constant-rate-of-strain (CRS) consolidation testing systems manufactured by GDS. One system uses the Rowe type consolidation cell and the GDS pressure/volume controllers (Figure 15). The second system uses GDS CRS cell and 50 kN load frame (Figure 16). With these systems samples can be back-pressure saturated prior to consolidation. Step loading, CRS and hydraulic gradient controlled tests can be performed at loads of up to 2000 kPa

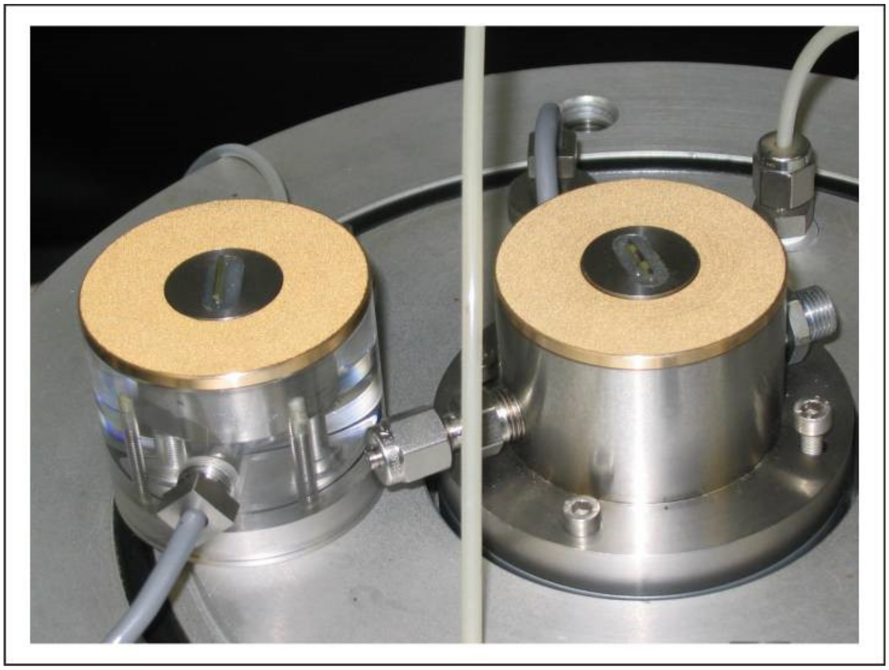

Figure 14. Piezoelectric bender element inserts in the top cap and base pedestal. Photo by K. MacKillop, NRCan photo 2021-707.

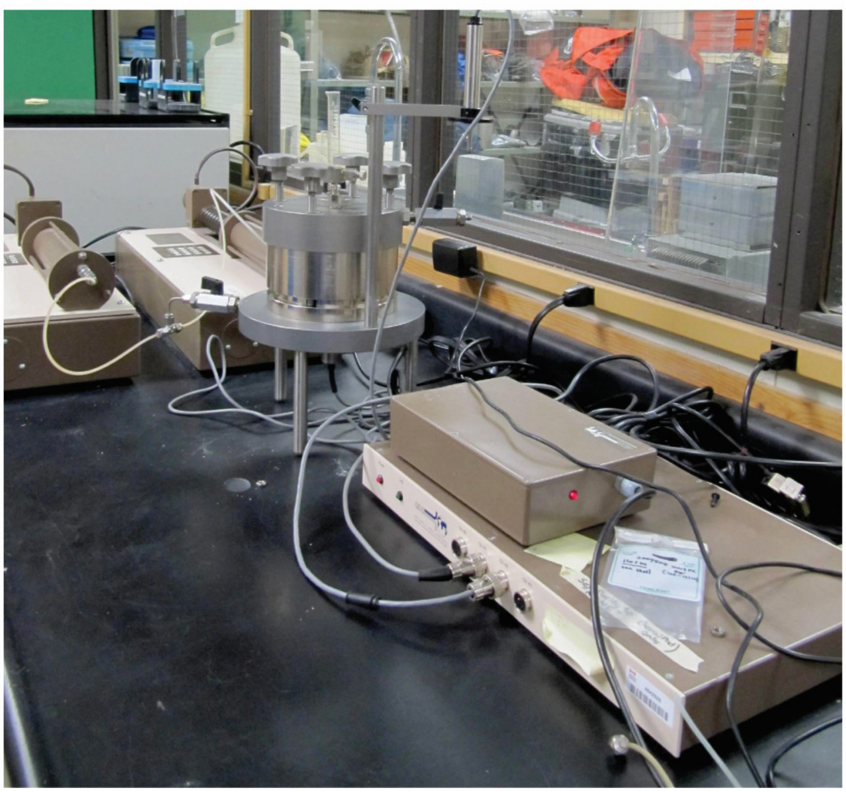

Figure 15. Rowe cell consolidation testing system. Photo by K. MacKillop, NRCan photo 2021-708.

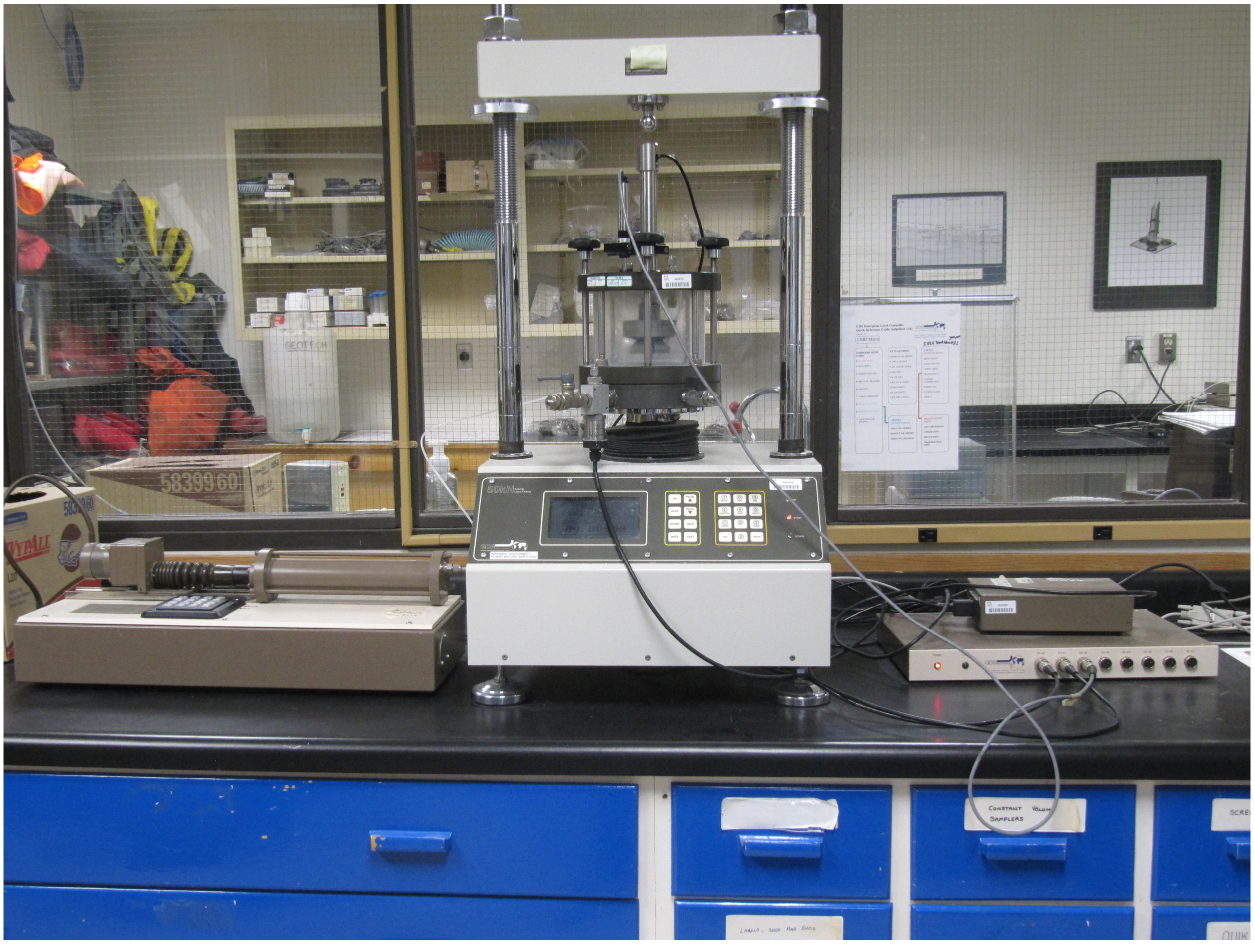

Figure 16. CRS cell consolidation system. Photo by K. MacKillop, NRCan photo 2021-520.

6.0 Conclusions

Offshore wind is a renewable source of energy that offers higher average hub-height wind speeds and provides the necessary room to construct large wind projects when compared with onshore wind resources. Generally, the *direct* cost of offshore wind as an energy source is higher than onshore, however prices have been steadily decreasing as OWT design, construction and deployment have become more efficient, and this doesn't account for socioeconomic costs (e.g., land use). Thorough and standardized geotechnical characterization of offshore seabed sediments at the design stage offers one method for reducing costs associated with offshore wind energy development. OWT foundations must be both able to support the lateral loads produced from wind, wave and tidal action and vertical loads from the weight of the structure. These loads are transferred down through the foundation into the marine soil, which ultimately supports the entire structure. These loads are different than those experienced by offshore oil and gas infrastructure, upon which many of the geotechnical models are based. Despite these differences, it is important that marine soils are capable of supporting these unique loads without failure. Important geotechnical properties, such as grain size distributions, density and Atterberg limits provide insight into the history of the soil and the behaviour of soil when loads are applied. Undrained shear strength, effective internal friction angle, compressibility and permeability are critical in the design and modelling of all OWT foundation types. Recommendations for future work in evaluating offshore wind development in Atlantic Canada would include using all available geotechnical data to characterize the engineering properties and foundation conditions of the offshore geology.

Summary Tables

Table 1: Geotechnical properties relevant for OWT foundations.

Geotechnical Property	Symbol	Test	Standard	Importance	Notes
Grain Size Distribution	n/a	Sieve grain size analysis (Lab)	ASTM D422-63	• Used to classify the marine soil at a site of interest. • Also identifies degree of sorting of the soil.	• Used for particles larger than 0.075 mm (no. 200 sieve). • For marine soils, sample must not be dried as it may alter the structure of particles.
		Hydrometer Grain Size analysis (Lab)	ASTM D422-63	" "	• Used for particles smaller than 0.075 mm (no. 200 sieve).
		Particle Size Analyzer (Lab)		" "	• Used for soils containing sands and smaller particles. • Outputs grain distribution chart from sample.
Dry Density	p_d	Drive Cylinder Method (Lab)	ASTM D7263 - 21	• High density soils are beneficial for all foundation times. • Increased ability to support lateral and vertical loads. • Less deformation in dense soils.	• Sample must be undisturbed. • The volume of the sample must be unchanged.
Bulk Density	p	Water Displacement (Lab)	ASTM D7263 - 21	" "	• Sample must be undisturbed. • Sample is coated in paraffin wax and submerged in water. • The density determined based on the displacement of the water.
		Linear Measurement method (Lab)	ASTM D7263 - 21	" "	• Weight sample of a known volume. • Sample must be undisturbed.
Water Content	w	Oven Drying Method (Lab)	ASTM D2216	• Used for Atterberg limits which describe the behaviour of the soil.	• Sample weighed before and after drying in the oven. • Correction for salt content must be taken in account for marine soils.
Liquid Limit	w_L	Atterberg Limit Tests (Lab)	ASTM D4318	• Water content at which the soil transitions between liquid and plastic state • Soil behaving in a liquid manner is likely to cause	• Water content at the transition between liquid and plastic states in soil • Describes the physical behaviour of soil

				failure for OWT foundations	
Plastic Limit	w_P	Atterberg Limit Tests (Lab)	ASTM D4318	• Water content at which the soil transitions between plastic and semi solid state. • Need to understand soil behaviour at these critical water content.	• Water Content at the transition between the plastic and semi solid state of soil.

Table 2: Geotechnical properties required for selected OWT foundations.

Foundation Type	Geotech Property	Symbol	Test	Standard	Importance	Notes
Piles	Undrained Shear Strength	s_u	Unconsolidated Undrained Triaxial (UU) (Lab)	ASTM D2850	• Shear strength is required to determine soil-pile interactions. • Used for calculations and modeling.	• The test is performed in a saturated environment. • Provides undrained soil strength properties and stress-strain.
			Consolidated Undrained Triaxial (CU) (Lab)	ASTM D4767-11	“ ”	• Test is performed on a saturated, consolidated, cohesive soils. • Provides undrained soil strength properties and stress-strain.
			Vane Shear Test (Field)	ASTM D2573	“ ”	• Used on cohesive fine grained soils. • Torque required to rotate a four bladed vane in the soil is measured and converted to a shear strength.
			Cone Penetrometer (CPT) (Field)	ASTM D5778	“ ”	• Measure’s resistance of soil to the constant penetration of cone penetrometer. • Provides a continuous undrained shear strength profile of soil.
			Dilatometer Test (Field)	ASTM D6635	“ ”	• Blade with expanding membrane and pressures needed to move the soil are recorded. • Undrained shear stress is measured in fine grained soils
	Friction Angle	φ	Consolidated Undrained	ASTM D4767-11	• Friction angle is the failure envelope of a soil.	• The test is performed in a saturated environment.

			Triaxial (CU) (Lab)		• Used in pile modelling and calculations	
			Consolidated Drained Triaxial (CD) (Lab)	ASTM D7181-20	“ ”	• The test is performed in a unsaturated environment.
			Cone Penetrometer (CPT) (Field)	ASTM D5778	“ ”	• The test is performed in a saturated environment. • Correlations are made and have been published between cone tip resistance and the peak friction angle of the soil.
	Shear Modulus	G	Resonant Column (Lab)	ASTM D4015-15e1	• Shear modulus is ratio of shear strain to shear stress. • Describes a soils response to shear stress.	• Used for determining soil structure reactions and the seismic response of soils.
			Bender Elements (Lab)	ASTM D8295-19	“ ”	• Shear wave velocity is measured over a known distance to find shear modulus. • Test can be performed in Triaxial tests, direct shear tests and oedometer tests to find G.
			Seismic Cone (Field)	ASTM D7400	“ ”	• Uses seismic wave velocities to determine important geotechnical elastic properties of soil. • Can be used to determine shear modulus, bulk modulus and young’s modulus.
	Compress-ibility	n/a	Consolidation test (Lab)	ASTM D2435M-11	• Pore water pressures can build during installation and increase installation time for piles. • Coefficients from consolidation tests estimate this pore water behaviour	• Vertical loads are gradually applied to a soil sampled and the response is measured. • Provides compressive information on the soil and information of past stresses.
Gravity Based Foundations	Undrained Shear Strength	s_u	Unconsolidated Undrained Triaxial (UU) (Lab)	ASTM D2850	• Shear strength of soil is required to complete a stability analysis for GBF.	• The test is performed in a saturated environment. • Provides undrained soil strength properties and stress-strain.

					• Stability analysis ensures that unwanted or uneven settlement does not occur.	
			Consolidated Undrained Triaxial (CU) (Lab)	ASTM D4767-11	" "	• Test is performed on a saturated, consolidated, cohesive soils. • Provides undrained soil strength properties and stress-strain.
			Vane Shear Test (Field)	ASTM D2573	" "	• Used on cohesive fine grained soils. • Torque required to rotate a four bladed vane in the soil is measured and converted to a shear strength.
			Cone Penetrometer (CPT) (Field)	ASTM D5778	" "	• Measure's resistance of soil to the constant penetration of cone penetrometer. • Provides a continuous undrained shear strength profile of soil.
			Dilatometer Test (Field)	ASTM D6635	" "	• Blade with expanding membrane and pressures needed to move the soil are recorded. • Undrained shear stress is measured in fine grained soils
	Compress-ibility	n/a	Consolidation test (Lab)	ASTM D2435M-11	• Compressibility is the most important to GBF's as they apply the largest vertical loads. • Any movement or settlement below a GBF is detrimental	• Vertical loads are gradually applied to a soil sampled and the response is measured. • Provides compressive information on the soil and information of past stresses
	Permeability	k	Constant Head Test (Lab)	ASTM D2434 - 19	• When the large load of GBF is applied to the seafloor, it can cause movement of water in the soil. • Porewater pressures can build up, which can result in long term settlement. • Important to understand the permeability behaviour of soil as a result.	• Measured under a constant stress state. • The test is run until a steady state is reached.

Suction Caisson	Undrained Shear Strength	s_u	Unconsolidated Undrained Triaxial test (UU) (Lab)	ASTM D2850	• Pull out capacity of caissons is dependant on the shear strength of the surrounding soil. • Also used for design and modeling of caissons.	• The test is performed in a saturated environment. • Provides undrained soil strength properties and stress-strain.
			Consolidated Undrained Triaxial Test (CU) (Lab)	ASTM D4767-11	" "	• Test is performed on a saturated, consolidated, cohesive soils. • Provides undrained soil strength properties and stress-strain.
			Vane Shear Test (Field)	ASTM D2573	" "	• Used on cohesive fine grained soils. • Torque required to rotate a four bladed vane in the soil is measured and converted to a shear strength.
			Cone Penetrometer (CPT) (Field)	ASTM D5778	" "	• Measure's resistance of soil to the constant penetration of cone penetrometer. • Provides a continuous undrained shear strength profile of soil.
			Dilatometer Test (Field)	ASTM D6635	" "	• Blade with expanding membrane and pressures needed to move the soil are recorded. • Undrained shear stress is measured in fine grained soils
	Friction Angle	Φ	Consolidated Undrained Triaxial (CU) (Lab)	ASTM D4767-11	• Describes the failure criteria for soil in and around suction caisson. • Required to determine how soil will respond when loads are applied	• The test is performed in a saturated environment.
			Consolidated Drained Triaxial (CD) (Lab)	ASTM D7181-20	" "	• The test is performed in an unsaturated environment.
			Cone Penetrometer (CPT) (Field)	ASTM D5778	" "	• The test is performed in a saturated environment.

						• Correlations are made and have been published between cone tip resistance and the peak friction angle of the soil.
	Compress-ibility	n/a	Consolidation test (Lab)	ASTM D2435M-11	• Helps to understand Field (in-situ) nature of the soil. • Increased porewater pressure can increase friction and slow installation	• Vertical loads are gradually applied to a soil sampled and the response is measured. • Provides compressive information on the soil and information of past stresses
	Permeability	k	Constant Head Test (Lab)	ASTM D2434 - 19	• Required to determine the pressure needed to seal the caisson.	• Measured under a constant stress state. • The test is run until a steady state is reached.

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