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Preface

Since 2009 the GSC has undertaken a research project to study marine geohazards in the Beaufort Sea. The primary driver for the project was the oil and gas industry's interest in exploration on the outer shelf and slope, in water depths up to 1500 m. The research integrates seabed morphology, seismic stratigraphy, geology and geotechnical parameters to assess geohazards including slope instability. In light of possible hydrocarbon exploration and development, slope instability represent one of the major potential geohazards in the region so their understanding and some degree of predictability is critical. Slope stability evaluation involves delineating the linkage between failure triggering mechanisms, the sediment properties, geology and geomorology.

A comprehensive geotechnical laboratory test program was carried out on 23 Beaufort Sea slope-situated piston and gravity cores collected between 2009 and 2016. Geotechnical testing included MSCL bulk density, Atterberg limits, grain size analysis, back-pressured consolidation tests and isotropically consolidated undrained (CIU) triaxial tests. The cores were collected by the Geological Survey of Canada – Atlantic and Pacific, Woods Hole Oceanographic Institution and Scripps Institution of Oceanography.

This open file presents the geotechnical testing methodology and geotechnical results. It provides geotechnical profiles, high-resolution seismic data over most core sites and a preliminary infinite slope analysis of the Beaufort Sea surficial slope sediments

Further information on these cores is available through the Expedition Database at https://ed.gdr.nrcan.gc.ca/index_e.php

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Lloyd Kedgwin, Woods Hole Oceanographic Institution kindly supplied the HEALY 2013 core to our GSC-A laboratory. Gordon Cameron, GSC-A participated in the HEALY 2013 core collection and its geological analysis. The authors thank the officers and crew members of CCGS Amundsen, CCGS Sir Wilfrid Laurier and Korean vessel Araon who participated in the various expeditions. J. Higgins and Kate Jarrett in the GSC marine core processing laboratory and the many co-op students who assisted with core processing. We thank Justin Buis, Deanna Coombs, Elise Ouellet, and Meaghan Macquarrie for assistance in Atterberg limits, consolidation and triaxial testing in the Geomechanical Laboratory at the GSC-A. We thank Owen Brown who completed grain size analyses with assistance from co-op students. Claudia Curry performed some of the core lithological summaries.

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LIST OF SYMBOLS

Symbol	Unit	Definition
A	-	Activity (Equation 3.2)
a	-	Ground Accelaration
A_f	-	Skempton's pore pressure paramter A at failure
B	-	Skempton's pore pressure parameter B (Equation 6.4)
C_c	-	Compression index (Equation 5.3)
C_r	-	Recompression index (Equation 5.4)
C_v	-	Coefficient of consolidation (Equation 5.6)
c'	-	Effective cohesion (Equation 6.1)
e	-	Void ratio
F_h	kN	Horizontial Force (Equation 7.4)
FS	-	Factor of safety (Equation 7.1)
g	m/s^2	Gravitational Acceleration of gravity, 9.81 m/s^2
H	m	Depth below seafloor (Equation 7.1)
H_c	m	Critical height (Equation 7.2)
H_s	m	Height of solid soil particles (Equation 5.2)
ΔH	-	Change in height
k	m/s	Hydralic conductivity (Equation 5.7)
k_h	-	Horizontial seismic coefficient
k_{min}	-	Minimum horizontial seismic coefficient (Equation 7.7)
k_o	-	The coefficient of earth pressure at rest
LI	-	Liquidity index (Equation 3.3)
LL	-	Liquid limit
$MSCL$	-	Multisensor core logger
m_v	-	coefficient of volume compressibility (Equation 5.5)
NC	-	Normal consolidation
OCR	-	Overconsolidation ratio
P'_c	kPa	Maximum past effective stress
PI	-	Plasticity index (Equation 3.1)
PL	-	Plastic limit
q	kPa	Deviator stress
R	kPa	Resistance stress
S	-	Ratio of S_u/σ'_v for OCR = 1 (Equation 6.3)
S_u	kPa	Undrained shear strength
t_{90}	s	Time corresponding to 90% primary consolidation (Equation 5.6)

Symbol	Unit	Definition
W	kg	Weight
w_n	%	Natural water content
w_c	%	Water content
z	m	Depth Below Seafloor (Equation 4.2)
ε_a	-	Axial strain
ε_i	MPa	Initial Young's Modulus
β	°	Seafloor Slope Angle
β_c	°	Critical Slope Angle (Equation 7.3)
ρ_{sat}	g/cm^3	Saturated Bulk Density
ρ_{sw}	g/cm^3	Salt Water Density
σ'	kPa	Effective stress (Equation 4.1, 4.5)
σ_c	kPa	Confining stress for triaxial test
σ_3	kPa	Horizontial stress for triaxial test
σ'_c	kPa	Effective consolidation stress for triaxial test
σ_n	kPa	Total normal stress (Equation 4.1)
σ'_n	kPa	Effective normal stress
σ_v	kPa	Total vertical overburden stress (Equation 4.3)
σ'_v	kPa	Effective overburden stress (Equation 4.7)
ϕ'	(°)	Effective internal angle of friction (Equation 6.1)
γ'	kPa	Effective unit overburden stress
μ	kPa	*In-situ* pore water pressure (Equation 4.1)
μ_e	kPa	Excess pore water pressure (Equation 4.8)
μ_h	kPa	Hydrostatic pore water pressure (Equation 4.4)
τ_f	kPa	Shear stress at failure

1.0 INTRODUCTION

The Canadian Beaufort Sea study area, Figure 1.1, reaches from the Alaskan border in the west, to the outermost M'Clure Strait in the northeast. The continental shelf, here up to 100 km wide, meets a northward dipping continental slope with the shelf break at about 100 m water depth. The Beaufort Shelf, is bordered east and west by two large embayments, Amundsen Gulf and Mackenzie Trough. The Banks Island Shelf lies north of this, bordered by M'Clure Strait. All three shelf-crossing troughs were glacially excavated which drained a significant portion of the Laurentide and Innuitian Ice Sheets during the last glacial maximum and also under previous glacial phases (Batchelor et al. 2013, 2014). The study area is subdivided into four geographic regions each with a suite of cores that are summarized and then compared. The regions include the Mackenzie Trough mouth, two adjacent areas on the central Beaufort Slope, and the far eastern Beaufort Sea, offshore Banks Island. This enables a broad assessment across a range in seabed slopes, earthquake proximity, failed/unfailed slopes and sediment provenance.

The upper slope along the Beaufort Sea continental margin, has been subject to multiple and long-term gravity mass failures. Some are large, in the mega-slide realm, termed slide valley complexes comprising multiple components with different behavior and relative timing (Cameron et al, 2018 Woodworth-Lynas et al. 2016, Sainte-Ange et al, 2014). In light of possible hydrocarbon exploration and development, these represent one of the major potential geohazards in the region so their understanding and some degree of predictability is critical. Their mapping and establishing timing presents challenges, currently addressed with multiple approaches, mostly from geophysical tools. However, ground truthing and direct geotechnical measurement, assessment and intergration.is a key component. Since 2009 the GSC has undertaken a research project to study marine geohazards in the Beaufort Sea. The primary driver for the project was the oil and gas industry's interest in exploration on the outer shelf and slope, in water depths up to 1500 m. The research integrates seabed morphology, seismic stratigraphy, and geology and geotechnical parameters to assess geohazards including slope instability.

Field-based datasets primarily include sediment cores, 3.5 sub-bottom profiler, and multibeam bathymetric seabed imaging. There is evidence for large multi-component slide valley complexes, known since the initial ArcticNet and hydrocarbon industry-supported contiguous multibeam coverage on the central Beaufort Slope in 2009 (Merzouk and Levesque 2009, page 308-309). This was used to target core sites during that and subsequent cruises. The ArcticNet multibeam and sub-bottom profiler dataset is the most expansive dataset and it is supplemented with mosaics and regional tracks from two cruises by the US Coast Guard icebreaker Healy, made available online, and though collaborative research with the Korean Oceanographic Institute

A comprehensive geotechnical laboratory test program that included MSCL bulk density, Atterberg limits, grain size analysis, back-pressured consolidation tests and isotropically consolidated undrained (CIU) triaxial tests was conducted on 24 Beaufort Sea slope-situated piston and gravity cores collected between 2009 and 2016 in water depths from 70 to 1536 m. Nineteen (< 7.25 m long) were collected during CCGS Amundsen 2009 to 2016 programs, two from a CCGS Sir Wilfred Laurier 2012 cruise, one from 2013 KOPRI icebreaker Araon cruise, and 2 > 14 m JPC core during a 2013 USCGS Healy expedition. Cores targeted unfailed sediments, deeply failed scar floors, previously failed deposits and fluid or permafrost-affected glacial and post-glacial sediments.

This Open File report describes the methodology including a complete overview concepts that influence the engineering behavior of the soils (Chapters 2 to 8). A comprehensive suite of results from the geotechnical tests in the study area is presented in Chapter 9. Geotechnical profiles that include down core grainsize, bulk density, water contents, Atterberg Limits, mini vane (MV) undrained, SHANSEP derived shear strength, and stress history and factor of safety are presented for each core. Also high-resolution seismic data is presented over most core sites. A geotechnical summary is presented for each region including stress history, activity and strength parameters. Finally, a slope stability analysis and investigation of various trigger mechanisms including gravitational loading and seismic loading (earthquakes) affecting the Beaufort Sea slope (Chapter 10).

It is important to note that interpretation and significance of the geotechnical data in terms of the main study goals (failure susceptibility and predictability) necessitates each core be placed in a geologic framework, including provenance and environment of deposition, material type, diagenic or gravity failure process in a stratigraphic sense and of course chronology. In this report the basic physical properties of the cores are presented but presentation of the value-added geologic interpretation, beyond a general setting, is minimal. Such a synthesis is underway, relatively mature for some cores but entirely lacking for many.

2.0 CORE PROCESSING METHODS

2.1 Shipboard operations

All cores presented in this report were collected from the Beaufort Sea during field expeditions from 2009 to 2016. The expeditions included in this report are CCGS Amundsen Expeditions

2009804, 2010804, and 2014804, 2016805, CCGS Sir Wilfrid Laurier Expedition 2012004PGC, non-CCGS vessels Expeditions 2013004PGC and 2013HEALY. Core samples were acquired using the AGC Long Coring Facility. On-board processing of core samples entailed sectioning the cores into 1.5 m lengths, measurement of sediment strength using a handheld torvane on the section ends, and collection of constant volume samples from the section ends. Core sections were then sealed with bees wax and stored upright in refrigerated storage to await further analysis at the GSC-A laboratories.

2.2 Multi Sensor Core Logger

The initial steps in core processing at the GSC-A Core Processing Laboratory are the non-destructive measurements of whole-core X-radiography and physical properties using the Multi Sensor Core Logger (MSCL). Whole-core X-radiography enables the evaluation of core quality and the semi-quantitative assessment of sediment structure and composition. The core is brought to ambient room temperature after X-radiography is completed and it is run through the MSCL for measurement of whole core sediment physical properties at a standard 1 cm down-core resolution.

The MSCL measures compressional (P) wave velocity in a transverse direction, bulk density and magnetic susceptibility. The sensor stand includes a core detection laser, a gamma ray emitter and detector, two rolling p-wave transducers, and a magnetic susceptibility solenoid. A piece of plastic core liner filled with distilled water is run through the MSCL every four sections of core to check the precision of the MSCL system.

2.2.1 Compressional Wave Velocity (PWL)

The P-wave logger system consists of two spring loaded compressional wave transducers (PWT) and two laser distance transducers attached to the PWT mountings. The PWTs are pushed against either side of the core as it moves between the transducers. A short 500 kHz compressional wave pulse is produced at the transmitting transducer at a repetition rate of 1 kHz. This wave pulse travels through the core and is detected by the receiving transducer and the time of flight of the wave pulse is measured. The two laser distance transducers measure the displacement of the active faces of the PWT transducers. The diameter of the sediment core is calculated by subtracting the liner thickness from the measured distance between the distance transducers. This calculation assumes that the core liner is filled with sediment. The P-wave travel time delay caused by the core liner and the electronics of the system is calculated using a distilled water standard of known diameter and temperature. The measured sediment P-wave travel time is corrected for the P-wave travel time delay. The sediment P-wave velocity is calculated as the sediment diameter/corrected P-wave travel time.

2.2.2 Gamma Ray Attenuation (GRA)

The GRA unit measures the bulk density of the sediment. It comprises a 10 millicurie ^{137}Cesium capsule housed in a 150 mm ø primary lead shield with 2.5 and 5 mm collimators and a sodium iodide scintillation detector housed in a 100 mm ø collimated lead shielding. The source and detector are mounted on opposite sides of the core as it moves through the central unit assembly. A narrow (pencil size) beam of gamma rays with energies principally at 0.662 MeV is emitted from the ^{137}Cesium source and passes through the diameter of the sediment core. At these energy levels Compton scattering is the primary mechanism for the attenuation of the gamma rays in most sedimentary material. The incident photons are scattered by collision with electrons encountered in the core and there is a partial energy loss. The attenuated gamma-ray beam is measured by the scintillation detector. The Compton scattering of the photons is directly related to the number of electrons in the path of the gamma ray beam. A two-phase model representing the mineral and interstitial water of fully saturated marine sediment is assumed for the MSCL GRA calibration. Aluminum is assumed to have an attenuation coefficient similar to common minerals found in marine sediments and represents the mineral phase. Distilled water represents the interstitial water phase. A calibration standard consisting of different thicknesses of aluminum and distilled water is used to calibrate the GRA. The measure of density of the sediments assumes that the marine sediment is fully saturated and completely fills the core liner. The diameter of the sediment is determined using the measured displacement between the laser distance transducers and the thickness of the liner. Sediment density is calculated using the calibration coefficients and the measured diameter of the sediment.

2.2.3 Magnetic Susceptibility (MS)

A Bartington loop sensor (MS2B) measures the magnetic susceptibility of the sediment. It is mounted to minimize the effects of magnetic or metallic components of the MSCL system. An oscillator circuit in the sensor loop produces a low intensity non-saturating, alternating magnetic field. Changes in the oscillator frequency caused by material that has a magnetic susceptibility is measured and converted into magnetic susceptibility values. Air measurements taken at the beginning and end of each section are used to correct the measurements for equipment drift during each section run.

2.3 Geotechnical Whole Round Sampling

The objective of the whole round sampling was to obtain undisturbed samples to conduct consolidation, triaxial and Atterberg limit tests. The whole round samples were selected to be representative of the different lithologies of the surficial Beaufort Sea marine sediments. The quality of the whole round samples were evaluated using the X-radiography images. The X-rays highlighted areas containing materials unsuitable for testing such as larger drop stones and coring disturbance.

2.4 Split Core

The plastic core liner is then split longitudinally by pulling a piece of fine wire through the sediment along the cuts in the plastic core liner. The two core halves are designated archive and working and are temporarily covered with plastic wrap. Each half is labelled with an up arrow, cruise number, sample number and section information.

Meter tape is placed along the length of the split core section to indicate down-core depth. The archive half is photographed, measured for colour reflectance, and described visually. The working half is immediately measured for physical properties, before the core dries, at intervals of 10 cm for velocity and shear strength and 50 cm for bulk density and water content.

Additional samples are taken depending on the specific core-site objectives. The core halves are re-covered with plastic wrap, sealed in labeled plastic core sleeving, placed in labelled plastic D-tubes and stored at 4°C in the GSC-A Marine Geoscience Collection Facility.

2.4.1 Core Photography

The archive half of the core is photographed using a Nikon D300 12.3 megapixel digital camera. Overlapping digital photographs are taken at two scales. The first is a close up image covering a 30 cm interval with a 5 cm overlap, and the second is a long shot image covering a 90 cm interval with a 25 cm overlap. The images are saved in raw, tiff and jpg formats.

2.4.2 Sample Description

Written laboratory descriptions for the sediment cores include: 1) condition of sample (e.g. cracks, disturbance, oxidation), 2) consistency of sample (e.g. soft, hard, firm) 3) reaction to 10% hydrochloric acid which indicates the presence of calcium carbonate, 4) colour, based on Munsell soil colour charts and 5) visual core description consisting of colour, texture, grain size, stratification, depositional contacts, bedforms, sedimentary and post-sedimentary structures, presence of organic material, shells, bioturbation and any other visible feature. Lithologic summaries were created on the basis of sedimentary facies developed from visual sediment colour, spectrophotometry data, sediment structures, sediment texture and shear strength.

2.4.3 Laboratory Mini Vane Shear Strength Measurements

Laboratory miniature vane shear strength measurements following (ASTM D4648/D4648M) are made using a motorized shear apparatus. A four bladed vane is inserted to at constant depths and rotated 90°/min until sediment failure. The difference in rotational strain between the top and bottom of a linear spring is measured and the torque required to shear the cylindrical surface around the vane is calculated. Peak and remoulded shear strength values are calculated according to ASTM D4648/D4648M. Peak shear strength measurements are taken at standard 10 cm intervals. Two to three measurements of remoulded shear strength are taken per 1.5 m core section.

2.4.4 Constant volume sampling

Constant volume samples are taken using a stainless steel cylinder of known volume. The cylinder is gently introduced into the sediment at a constant rate. The cylinder is then carefully removed from the core and trimmed using a wire saw. The sediment is extruded from the cylinder, weighed, dried at 105°C for 24 hours and weighed again. Bulk density, water content and void ratio values are calculated according to (ASTM D2216).

2.5 Grain Size

Grain size tests were completed on Atterberg limit and consolidation trimmings. Grain size analysis were completed on samples using the Beckman Coulter Laser LS-230 laser diffraction analyzer at the GSC-A sedimentology laboratory. The Beckman Coulter Laser LS-230 has a grain size analysis range of 0.4 to 2000 μm. Analysis of the >63 μm fraction were manually wet sieved and merged with the <63 μm fraction. The coulter laser method used material from liquid limit test residues and additional subsamples taken from the cores.

3.0 ATTERBERG LIMITS

3.1 Introduction

Atterberg limits are index property tests applied to fine grained sediments to define the degree of plasticity. A soils degree of plasticity depends on the ratio of silt to clay sized particles within the soil matrix and the mineralogical composition of the colloid particles. Standard testing procedures developed by Atterberg (1911) and Casagrande (1932b) experimentally derive plastic (PL) and liquid limits (LL) at which a soil changes phase between the solid and liquid states (ASTM D4318). Although Atterberg limits provide no information on the grain size distribution or the type of minerals present, they nevertheless offer quantitative information on the engineering behavior of sediments and a means to classify the soil.

Under a similar principle as water changes phase from a solid, to a liquid, to a gas with increasing temperature, a fine grained soil will change phase from a solid, to a plastic, to a liquid with increasing water content. The PL defines the water content at which a fine grained sediment, in the remolded state, changes between the solid to plastic condition. Likewise the LL defines the water content at which a fine grained sediment, in the remolded state, changes between the plastic to liquid condition. The plasticity index (PI) defines the range of water content in which behaves in the plastic condition, defined by the difference between the LL and PL values,

$$PI = LL - PL \qquad \text{Equation 3.1}$$

A sediment classifies as fine grained if the majority has a particle size of less than 420 μm (#40-mesh sieve). Silt and clay particles are fine grained. Clay particles contribute effectively to a soils plasticity. Silts particles are similar to sands, being spherical in shape, between 2 and 60 μm, and controlled through mass-derived forces. Clay particles are colloidal, flat plated structures of 2 μm or less and are controlled through electrostatic forces on the surface of the particle. The mineralogical composition of colloidal particles dictates how strongly the particle will bond. Expansive clays, like montmorillonite, can absorb large quantities of water while remaining in the plastic state.

It is widely recognized that the higher the plasticity index the more pronounced the colloidal properties of the clay. A direct relationship is shared between the clay fraction and the *PI*, with *PI* increasing with colloid particles. This relationship is the activity (*A*) and expressed by the ratio of *PI* to the clay fraction (Skempton, 1953),

$$A = \frac{PI}{Clay\ Fraction} \qquad \text{Equation 3.2}$$

Where sediments with *A* < 0.75 are inactive, sediments between 0.75 < *A* < 1.25 are of normal activity, and sediments with *A* > 1.25 are active where the colloidal properties of clay are more pronounced (Skempton, 1953). Active clays are prone to large water volume changes causing swelling that can lead to landslides. Whereas inactive clays, or non-swelling passive clays, which tend to stabilize soils (Hansen et al, 2012).

The plasticity chart classifies soil as predominantly silt or clay, with the added denotation of fat or lean, and inorganic or organic. The terms fat (i.e. high *LL*) and lean (i.e. low *LL*) describe a soils ability to attract and retain water. Inorganic sediments form from rocks weathering overtime into their basic mineralogy. Due to the highly complex nature of organic chemistry, organic sediments are not well understood by the soil engineer. There is evidence to suggest that organic sediments behave as inorganic particles. This is observed with diatoms that exist in both the size and shape of silt and clay particles.

It is important to note Atterberg limits are empirically derived after the sediment has been reworked into a remolded state. Fine grained sediments with an *in-situ* water content equal to the *LL* have little strength when remolded, but may have considerable *in-situ* strength. Therefore fine grained sediment several meters below the seafloor with water contents above their *LL* do not exist in the form of a fluid mud; however, it would reduce to such a state in the remolded condition (Skempton, 1970).

3.2 Liquid Limit Test

The *LL* defines the water content at which a fine grained sediment changes from a plastic to a liquid state. The *LL* is experimentally derived by placing a remolded soil into a Casagrande's cup (Figure 3.1) and separating the soil into two halves with a standard cutting tool. A lever is turned at a consistent and rapid rate to raise and drop the Casagrande's cup until the gap closes by ½ inch. A water content sample is then taken at the point of closure. The liquid limit test is repeated a minimum of three times, adding or removing water as necessary to achieve blow counts from 15 to 40. The blow count plotted against the water content on a semi-logarithmic graph yields a linear relationship (Figure 3.2). A line of best fit is drawn and the water content at 25 blow counts represents the *LL*.

3.3 Plastic Limit Test

The *PL* is determined through a standard procedure to define the boundary between the plastic and solid state. A soil is remolded and then rolled into a clay thread until the thread begins to crumble at a diameter of ⅛ inch (Figure 3.3). The water content of the broken soil is then determined. This is repeated for a second piece of soil with the plastic limit reported as the average of the 2 tests.

3.4 Soil Classification

The *LL* and *PL* are used to classify fine grained cohesive soils (Casagrande, 1948) following ASTM D2487. Particle size tests provide quantitative data on the range of sizes of particles and the amount of clay present, but say nothing about the type of clay (Head, 1992). Fine grained soils with greater than 50% passing the #200 sieve (75 μm) are classified using a plasticity chart plotting *LL* versus *PI* (Figure 3.4). The plasticity chart classifies fine grained sediments through the naming convention displayed on the chart. The A-line separates silts "M" from clays "C". The chart used in this report follows British Standard practice (Head, 1992). The British Standard plasticity chart is divided into 5 zones:

(1) clays of low plasticity (CL) with less than 35 *LL*.
(2) clays of medium plasticity (CI) with *LL* from 35 to 50.
(3) clays of high plasticity (CH) with *LL* from 50 to 70.
(4) clays of very high plasticity (CV) with *LL* from 70 to 90.
(5) clays of very extreme plasticity (CE) with *LL* exceeding 90.

3.5 Liquidity Index

The natural water content (w_n) of fine grained sediments can be compared to their Atterberg limits by using a ratio defined as the liquidity index (*LI*),

$$LI = \frac{w_n - PL}{PI} \qquad \text{Equation 3.3}$$

The *LI* approximates whether a soil is normally consolidated ($LI=1$), underconsolidated ($LI>1$) or overconsolidated ($LI<1$). The *LI* values generally decrease with depth below seafloor for normally consolidated soils, and therefore *LI* can be used to evaluate trends in water content with depth. As an example, consider two samples at different core depths but with the same *in-situ* water content. Because the water contents are equal, it may be inferred that the deeper sample has not experienced additional compaction due to the greater overburden. If the *LI* however for the deeper sample is less than the shallow sample this would imply normal consolidation for both samples.

Seed et al., 2003 suggests that lean silts and clays with high *in-situ* water contents relative to their *LL* may be susceptible to liquefaction under a cyclic load. The Atterberg limits have been used by Seed et al, 2003 to identify potentially liquefiable fine grained sediments under cyclic loading (Figure 3.5). It is recommended that soils that plot within the zones undergo a testing program to evaluate further their potential for liquefaction.

4.0 SOIL STRESSES

4.1 Introduction

Sediments are a multi-phase system of solid soil particles and void space. The cross-sectional area of a soil subject to an applied load is shown in Figure 4.1. Under a microscope it is made visible that the sediment is composed of discrete, individual particles as well as void space; hence, the soil particles are relatively free to move with respect to one another.

Soil grains transmit forces from one adjacent particle to the next. At each point of contact there is an equal and opposite normal (compressive) force and tangential (shear) force (Figure 4.1). As the load increases, the void space decreases and the contact area between particles increase. This results in greater frictional resistance between soil grains. The inter-granular frictional resistance is termed the effective stress (σ'), which is the support system of soils.

The space between the particles is the void space, which contain variable amounts of water or air. Marine sediments are considered to be fully saturated, meaning the void space is entirely filled with water. Water within the void space has a profound effect on the effective stress of sediments. Water pressure above the hydrostatic condition will reduce the frictional resistance between soil particles, thus reducing the effective stress (σ') of soils.

4.2 Total Stress

The total normal stress (σ_n) is simply the vertical pressure exerted by the weight of the materials above the point considered (Skempton, 1970). A saturated soil in equilibrium will support the total stress partly by the soil grains at their points of contact (σ') and partly by the water pressure (μ, Figure 4.2) and is represented by,

$$\sigma_n = \sigma' + \mu \qquad \text{Equation 4.1}$$

The total stress is calculated using the saturated bulk density (ρ_{sat}) of the soil and the thickness of the soil layer. The total vertical stress σ_v is the sum of the saturated bulk densities of the material at depth (z) multiplied by the gravitational constant g,

$$\sigma_v = \int_0^z \rho_{sat} g z \qquad \text{Equation 4.2}$$

The σ_v, expressed as kN/m^2 or kPa, is calculated using,

$$\sigma_v = \rho_{sat} z * 9.81 \qquad \text{Equation 4.3}$$

4.3 Hydrostatic Pore Water Pressure

Water in a state of equilibrium is in the hydrostatic condition, meaning there is no pressure differential, or fluid flow. Hydrostatic pore water pressure increases proportionally with depth and is calculated using,

$$\mu_h = \mathrm{gz}\rho_{sw} \qquad \text{Equation 4.4}$$

where ρ_{sw} is the density of salt water. Under hydrostatic conditions, water does not impart a stress to the soil grains. This is observed in porous sediments on the seafloor that retain their open structure even at extreme water depths. Intuitively speaking, a change in the water depth would scarcely cause any change in the compaction of clay underlying the seabed (Skempton, 1970).

4.4 Effective Stress

Effective stress is one of the most important concepts in soil mechanics and is the difference between the total stress and pore water pressure. Mechanical loading by deposition of soils reduces the void spaces and increases the grain-to-grain contact and frictional resistance between soil particles. As the void space reduces, there is a corresponding increase in strength as the soil particles rearrange into a new shape. The study of soil mechanics is exclusively dealt with in terms of effective stress. The effective stress is defined as,

$$\sigma' = \sigma_n - \mu \qquad \text{Equation 4.5}$$

The effective overburden stress, σ'_v is calculated at a depth of z by,

$$\sigma'_v = \int_0^z \left(\rho_{sat} - \rho_{sw}\right) gz \qquad \text{Equation 4.6}$$

The σ'_v effective stress in kN/m^2 or kPa is calculated using,

$$\sigma'_v = \left(\rho_{sat} - \rho_{sw}\right) z * 9.81 \qquad \text{Equation 4.7}$$

4.5 Excess Pore Water Pressure

When a marine sediment is in a state of equilibrium, the water in the void spaces will be at hydrostatic conditions and the weight of the overburden is entirely supported by grain-to-grain contact. Excess pore water pressures (μ_e) generate when the *in-situ* pressures (μ) rise above hydrostatic conditions (μ_h) and is defined as,

$$\mu_e = \mu - \mu_h \qquad \text{Equation 4.8}$$

Under excess conditions, water applies a pressure in all directions and reduces the frictional resistance between particles. In the special case where μ_e equals the total stress (σ), the effective stress (σ') reduces to zero resulting in liquefiable conditions. This most often occurs in sands or silty sands where cyclic loading can cause a significant generation in μ_e and removes the strength entirely. Excess pore water pressures generate when there is an imbalance to the system. These conditions may exist under rapid loading (i.e. delta fronts, landslides or earthquake induced cyclic loading).

5.0 CONSOLIDATION

5.1 Introduction

Soils undergo consolidation under a compressive load, which increases the pore pressure above hydrostatic and induces fluid flow through the sediment, resulting in a reduction in volume. As water is squeezed from the voids, pore pressure dissipates and the applied load is transferred to the soil grains. This continues until the pore water pressure returns to hydrostatic conditions.

The rate at which the excess water pressure dissipates is dependent on the hydraulic conductivity and thickness of the soil layer. Coarse-grained soils like sands and gravels have a high hydraulic conductivity, which means excess pore water pressures usually do not develop. Fine grained soils like silts and clays have a low hydraulic conductivity and may require considerable time for pore pressure dissipation.

5.2 Stress History

Soils are said to carry a memory because the arrangement and orientation of the grains remembers its maximum past effective stress, known as the pre-consolidation pressure (P'_c). A comparison of the P'_c to the present day effective overburden (σ'_v) is termed the *OCR* ratio and used to determine the soils stress history. A soil is normally consolidated if the P'_c is equal to the *in-situ* effective overburden (OCR=1), over consolidated if the *in-situ* effective overburden pressure is less than the P'_c ($OCR>1$), and under consolidated if the *in-situ* effective overburden stress is greater than P'_c (OCR<1).

Knowing the soils geologic history enables a better understanding of the stress history. Consider a recently deposited soil on the sea floor, illustrated by point *(a)* in Figure 5.1. The soil exhibits a high void ratio and low effective stress. As deposition continues and the sediment is buried beneath the sea floor, gravitational compaction increases the effective overburden stress and decreases the void ratio, represented by point *(a)* through *(c)*. A reduction in effective overburden stress due to erosion allows the grains to expand elastically, represented by point *(c)* to *(d)*. The soils at point *(b)* and *(d)* are under the same present day effective overburden stress and in all respects identical except for their stress history (Skempton, 1970). Therefore, two similar soils at different locations with the same *in-situ* effective overburden stress could have a dramatically different stress history, resulting in one soil being normally consolidated *(b)* and the other being over consolidation *(d)*. Possible mechanisms for over consolidation include erosion, wave loading, ice loading and cementation (Poulos, 1988).

Marine sediments in the upper 2 m generally have an *OCR* value greater than 1, even though the sediments geological history suggests it was normally consolidation. This is termed apparent over consolidation and is throughout the world's oceans. Possible causes of apparent over consolidation include bioturbation, creep or particle bonding (Gourvenec and White, 2010). This phenomenon creates difficulty in distinguishing between true over consolidation and apparent over consolidation.

Under consolidation indicates the sediments have not fully consolidated under the *in-situ* stresses and suggests the presence of excess pore water pressures. Possible causes of under consolidation include rapid sedimentation or the presence of gas in marine sediments. In either case, the presence of excess pore pressures reduce the frictional resistance between soil particles resulting in effective stress that is less than the total stress.

5.3 Consolidation Testing

5.3.1 Introduction

Consolidation tests reproduce gravitational compaction in a controlled environment to simulate a soils response to vertical loading. A consolidation plot is developed from a series of increasing incremental loads applied to a soil sample. At each load the soil is exposed to an increase in total normal stress (σ_n), which results in sample deformation and corresponding decrease in volume as measured by void ratio. The process is repeated at different vertical stresses until a consolidation plot is defined (Figure 5.2).

The initial portion of the consolidation curve is the reloading of the sample. Little void ratio change occurs over these pressure intervals, since the sample has previously experienced this stress *in-situ.* Beyond the P'_c marks the beginning of the virgin compression line and effective stresses not previously experienced by the soil sample. The slope of the virgin compression line (C_c) represents the reduction in void ratio as a function of effective stress. C_c is a measure of the compressibility of the sediment and defines the void ratio-depth function of the sediment. Each point on the virgin compression line experiences both plastic and elastic deformation. Plastic deformation is an unrecoverable strain caused by the soil skeleton taking on a new shape to support the load. The unloading portion of the curve represents the elastic rebound of the material. The slope of this portion of the curve defines the recompression index (C_r), which characterizes the sediments elastic response to stress relief.

The maximum curvature of the transition between reloading and the application of new loads approximates the past maximum stress experienced by the soil (P'_c). P'_c can be approximated using several procedures (Grozic et al., 2003). The procedure developed by Casagrande (1936b) is the most widely used. Comparing the P'_c to the present day effective overburden stress, expressed as the over consolidation ratio determines the degree of consolidation and defines whether the soil is normally consolidated ($OCR=1$), over consolidated ($OCR>1$) or under consolidated ($OCR<1$). The consolidation tests followed the general test procedures outlined in ASTM D2435 (test method B).

5.3.2 Consolidation Equipment

The GSC-A geomechanical laboratory has two GDS back pressured consolidation testing systems. The constant rate of strain (CRS) system (Figure 5.3) consists of a CRS consolidation cell for 6.35 cm diameter samples, a 50 kN load frame, one GDS 1 MPa standard pressure/volume controller, 25 mm linear displacement transducer, 2 MPa pore pressure transducer, 2 kN submersible load cell, 16 bit DAQ system, computer and GDSLab software. The Rowe cell system (Figure 5.4) consists of two GDS 2 MPa standard pressure/volume controller, 25 mm linear displacement transducer, 2 MPa pore pressure transducer, a linear displacement transducer, 16 bit DAQ system, computer and GDS software. The GDSLab software can perform incremental loading and the constant rate of strain (CRS) consolidation tests.

5.3.3 Sample Preparation

The samples for the consolidation tests were taken from whole round samples. The CRS cell sample preparation includes extrusion of sediment into a consolidation ring with a sharp cutting edge and an ID of 6.35 cm. The samples were trimmed in the consolidation ring with a wire saw to a height of 2.2 cm. The sample is then placed on the base of the CRS consolidation cell (Figure 5.5). The CRS cell top is secured and filled with de-aired water.

The Rowe cell sample preparation includes pushing a cutting shoe with a sharp cutting edge into the whole round sample which extrudes the sediment into a consolidation ring with an ID of 6.3 cm. The samples were trimmed in the consolidation ring with a wire saw to a height of 2.58 cm. The consolidation ring was then placed on the Rowe cell base (Figure 5.6) and the cell top was placed over the consolidation ring. Initial measurements of dimension, weight and water contents were taken after the samples were trimmed in the consolidation rings.

5.3.4 Sample Saturation

The removal of the sediment from the marine environment permits the formation of gas bubbles. The minute gas bubbles may be entrapped in the pore fluid and have considerable effect on the consolidation and permeability of the sediments. The gas bubbles in partially saturated sediment are highly compressible compared with the relatively incompressible pore water and sediment. The gas bubbles also impede the flow of water through the pores, reducing the effective permeability of the sediment.

The saturation stage is designed to ensure all voids within the test sample are filled with de-aired water. The sample saturation involves increasing the axial stress and the pore fluid pressure. The axial stress is set to 2 kPa higher than the backpressure. The duration of the saturation stage ranged from 3 to 6 hours. The samples were then allowed to adjust to the back-pressure for a minimum of 12 hours.

5.3.5 Loading/Unloading

Vertical loads were applied until the virgin compression line (*VCL*) was established which enabled the determination of C_c. Load increments increased by 50% of the previous load. The vertical load is initially carried by the pore water resulting in an increase in pore pressure (Figure 5.7). The dissipation of excess pore pressure to back pressure and corresponding volume change marks the end of primary consolidation. Most volume change occurs in this stage. The time required for excess pore pressures to dissipate depends on the hydraulic conductivity, compressibility, and the length of the drainage path. Load increments were applied at time intervals which ensured the completion of primary consolidation.

Once the *VCL* is defined, the samples are unloaded by applying increments of decreasing load. Load increments decreased by 100% of the previous load. The reduction of the vertical load resulted in an initial decrease in pore pressure as the soil skeleton expands. The pore pressure will return to the backpressure as water returns to the sample (Figure 5.8). Once the pore pressure returns to the backpressure the next unload increment is applied.

The effective axial stress (σ') for the CRS cell was calculated as the difference between the pore pressure (μ) and total axial stress (σ). The axial stress for the Rowe cell was calculated using,

$$\sigma_v' = (\sigma_{up} * 0.996) - \mu_b \qquad \text{Equation 5.1}$$

where σ_{up} is the load clamber pressure, μ_b is the back pressure and 0.996 is the area correlation for the drainage port on the spindle that is in contact with the sample. The pore pressure in the drainage port remains constant throughout the test and does not apply vertical load on the sample.

5.3.6 Data analysis

The sample volume change is determined using the change in sample height since the cross-sectional area of the sample is constant throughout the test. The volume is represented as a void ratio and is calculated at the end of each load increment using,

$$e_{end} = e_i - \frac{\Delta H}{H_s} \qquad \text{Equation 5.2}$$

where e_{end} is the void ratio at the end of the load stage, e_i is the void ratio at the start of the load stage and ΔH is the change in height during the load stage and H_s is the height of the solids.

Soil parameters derived from the consolidation plot include the compression index (C_c) and the recompression index *(C_r).* C_c is calculated from the slope of the *VCL* and defines the compressibility of the soil,

$$C_c = \frac{\Delta e}{\log(P_1'/P_2')} \qquad \text{Equation 5.3}$$

C_r is the slope of a straight-line approximation of the unloading portion of the consolidation curve and calculated by,

$$C_r = \frac{\Delta e}{\log(P_2'/P_1')} \qquad \text{Equation 5.4}$$

The compressibility can also be described as the compressibility per unit thickness of the soil. This is known as the coefficient of volume compressibility (m_v) and defined by,

$$m_v = \frac{1}{1+e_1}\left(-\frac{\Delta e}{\Delta p}\right) \qquad \text{Equation 5.5}$$

where e_1 is the void ratio at the start of the stress increment, Δp is the change in effective axial stress and Δe are the change in void ratio for the stress increment.

The rate at which the sample compresses during each load increment is termed the coefficient of consolidation (C_v) and is dependent on the hydraulic conductivity of the soil. The C_v values were calculated using Taylor's square root of time method (Holtz and Kovacs, 1981). The time versus height compression curves are used to calculate C_v using,

$$C_v = \frac{0.848H^2}{t_{90}} \qquad \text{Equation 5.6}$$

where H is the length of the drainage path and t_{90} is the time corresponding to 90% of primary consolidation.

The hydraulic conductivity (k) is calculated using,

$$k = C_v\, m_v \rho_{sw} g \qquad \text{Equation 5.7}$$

6.0 SHEAR STRENGTH PROPERTIES

6.1 Introduction

The shear strength of a soil the maximum resistance a soil displays against failure and directly controlled by its effective stress. Several sources contribute to a soils shear strength, but gravitational compaction and the accompanying squeezing out of pore water under the weight of overburden sediments contributes, by far, the most. Other sources of strength include bioturbation, desiccation, and particle bonding. The most common failure criteria applied to soil is the Mohr-Coulomb failure criteria defined as,

$$\tau_f = c' + (\sigma_n - \mu)\ tan\phi' \qquad \text{Equation 6.1}$$

where τ_f is the shear stress at failure, σ_n is the total normal stress, μ is the pore pressure, c' is effective cohesion and ϕ' is effective internal angle of friction.

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' (Figure 6.1). The Mohr-Coulomb failure envelope defines the stresses at which failure occurs. At stress conditions below the failure envelope, the soils are stable.

The cohesion marks the bonding strength of fine grained soils. Cohesion typically increases with clay content, but effectively depends on the type of clay minerals and their colloidal activity. While cohesion is negligible in most soils, there are soils where it provides a considerable source of strength, particularly soils containing the clay mineral montmorillonite (Lambe and Whitman, 1969). The internal angle of friction (ϕ') is the frictional resistance of soils grains and depends on their shape and size. Coarse-grained soils like sands have a higher friction angle than clays.

The shear strength of sediments at different depths can be compared by normalizing the shear strength with respect to the effective overburden (S_u/σ'_v). The stress history influences the soils S_u/σ'_v ratios. Over consolidated soils possess a greater strength than what is suggested by the *in-situ* overburden and results in a relatively high S_u/σ'_v. Normally consolidated soils generally have S_u/σ'_v values in the range of 0.2-0.5, while over consolidated clays have a S_u/σ'_v greater than 0.5 (Skempton, 1970).

6.2 Strength Profiles

6.2.1 Introduction

A method to characterize the undrained shear strength of soils was presented by Ladd and Foot in 1974. This is known as the SHANSEP (Stress History And Normalized Soil Engineering Properties) method and is the most widely used. The SHANSEP method was used to create continuous strength profiles for normally consolidated sediment (*OCR=1*). Essentially the profile's shear strength is the expected shear strength increase with depth for a soil in a normal stress state.

6.2.2 SHANSEP Method

The SHANSEP method based on the observation that the shear strength (S_u) of most cohesive soils increase with depth below the seafloor and can be normalized with respect to the present-day effective overburden (S_u/σ'_v). A comprehensive analysis of the sediments undrained shear strength using the SHANSEP method includes the determination of S_u/σ'_v for several OCR values (Figure 6.2) and is presented as,

$$\frac{S_u}{\sigma'_v} = S\,(OCR)^m \qquad \text{Equation 6.2}$$

where S_u is the undrained shear strength, S is the S_u/σ'_v ratio for normally consolidated soils and m is a soil constant. For simplicity, S will be used to represent the S_u/σ'_v ratio for normally consolidated soils for the remainder of the report. A continuous strength profile for normally consolidated sediments is plotted using,

$$S_u = S \times \sigma'_v \qquad \text{Equation 6.3}$$

The stress history of the sediments can be estimated by comparing the shear strengths calculated using Equation 6.3 with the measured laboratory MV data (Figure 6.3). If the measured MV shear strength is greater than the calculated S_u, the sediments are considered over consolidated. If the MV shear strength data is less than the calculated S_u, the sediments are considered under consolidated.

Assuming that shear strength is a function of gravitational compaction, a linear correlation can be made between the shear strength and depth below the seafloor. The S ratio does not account for other sources of strength, therefore S_u/σ'_v provides only an approximation of predicted shear strength and must be used with caution.

6.3 Triaxial Testing

6.3.1 Introduction

The triaxial test is one of the most versatile and widely performed geotechnical laboratory tests, allowing the shear strength and stiffness of soil to be determined for use in geotechnical design. Advantages over simpler tests include the ability to control specimen drainage and take measurements of pore fluid pressures. Primary parameters obtained from the test include the Mohr Coulomb failure criteria, ϕ' and c'.

The triaxial test typically involves placing a cylindrical soil sample sealed in a rubber membrane, into a cell that can be pressurised. The specimen is saturated, consolidated, and sheared, allowing the soil response to be observed under conditions that may approximate those *in-situ*. The confining pressures of consolidation strengthen the soil sample by creating a pressure differential between the pore water in the void spaces and the fluid cell pressure which surrounds the sample. This pressure differential creates an increase in effective stress as the sample consolidates. A general set-up of the triaxial sample is shown in Figure 6.4.

There are three primary types of triaxial tests conducted in the laboratory including unconsolidated undrained (UU), consolidated undrained (CU) and consolidated drained (CD). The CU test is the most common triaxial test and was used for this study following ASTM D4767. In this test method, the soil sample is fully consolidated under a stress applicable to field conditions (i.e. effective overburden stress). The sample is then subjected to a compressive axial stress without allowing further consolidation to take place.

The stresses applied to a soil sample in a triaxial compression test are displayed in Figure 6.5. The confining stress, σ_c is applied by pressurising the cell fluid surrounding the sample during the consolidation stage. The difference between the σ_c and the pore pressure, μ if the effective stress, σ'_c and represents the consolidation stress. The deviator stress, q is generated by applying an axial stress greater than the confining pressure (σ_c) during the shear stage and is equal to σ_1 - σ_3.

6.3.2 Triaxial Test Equipment

The triaxial system used is a GDS computer controlled hydraulic triaxial testing system consisting of a 50 mm Bishop & Wesley stress path triaxial cell, 3 GDS 2 MPa pressure/volume controllers, a 5 kN submersible load cell, pore pressure transducer, linear displacement transducer, a DAQ system, a computer and GDS software (Figure 6.6). The GDS pressure/volume controllers and the 3 transducers control and read the cell's back pressure, pore pressure, back volume, axial displacement, axial load and effective stress.

6.3.3 Triaxial Test Procedure

Introduction

The CU triaxial test as described by (ASTM D4767) consists of four main stages: 1) sample and system preparation, 2) sample saturation, 3) consolidation and 4) compressive shear. Generally, three specimens are tested at different effective consolidation stresses to define a strength envelope. The lack of available sediment for this study necessitated conducting multi-stage triaxial tests. This type of test is practical when economizing on soil samples or when limited sample is available. Multi-stage triaxial tests are performed in three consolidation and shear sequences. In each sequence, the sample is consolidated to an increasing consolidation stress and then sheared. The first 2 shear stages are stopped when the deviator stress levels off or at approximately 4% strain. The third shear stage continued until 15-18 % total strain. The stresses at failure define the failure envelope and maximum shear strength.

Specimen Preparation

A thin-walled sampling tube (14 cm, 50 cm ID) with a sharp cutting edge was pushed into the whole round core sample and extruded with sediment from the core liner. The samples were trimmed in the sampling tube with a wire saw to a height of approximately 12.0 cm and initial measurements of dimensions and weights for water content were made. The samples were extruded from the sampling tube into a rubber membrane fitted in a split form, which was attached to the base pedestal of the triaxial cell Figure 6.7. Following placement of the sample, the triaxial cell and other system components are assembled. During this stage the cell is filled with de-aired water, pressure/volume controllers connected, and transducer readings set as required.

Saturation

The saturation stage is designed to ensure all voids within the test sample are filled with de-aired water. The sample saturation involves increasing the axial stress and the pore fluid pressure. The axial stress is set to 2 kPa higher than the backpressure. The duration of the saturation stage ranged from 3 to 6 hours. The samples were then allowed to adjust to the back-pressure for a minimum of 12 hours.

The degree of specimen saturation is determined by conducting a *B*-check to determine Skempton's *B*-value (Skempton, 1954) which is defined as,

$$B = \frac{\Delta\mu}{\Delta\sigma_3} \qquad \text{Equation 6.4}$$

The sample is considered to be 100% saturated at a *B* value of > 0.95. The back pressure required to obtain 100% saturation ranged from 200 kPa to 300 kPa.

Consolidation

The consolidation stage is used to bring the sample to the effective stress required for shearing by increasing the confining stress (σ_c) while maintaining a constant pore pressure. The increase in cell pressure results in an initial increase and then dissipation of the sample's pore pressure (Figure 6.8). The consolidation process continues until at least 95% of the excess pore pressure generated has dissipated.

The first consolidation stress is set to the *in-situ* effective overburden stress calculated from MSCL bulk density data. After the sample has consolidated and sheared, a second confining pressure is applied. The second confining pressure is at a minimum of 2.5 times greater than the effective overburden stress to remove the effect of stress history (Ladd and Foot, 1974). The 3rd confining pressure is set at twice the 2nd confining pressure. The typical 3 confining pressures chosen are illustrated in Figure 6.9. Confining pressures 2 and 3 become the new P'_c for the sample and the $OCR=1$.

Shearing

The samples were sheared by applying an axial stress to the test sample at a constant rate of strain through upward compression by movement of the base pedestal. The sample response during the shear stage was monitored by plotting the deviator stress (q) against axial strain ($\varepsilon_a = \Delta L/L$). The shear stage was continued until a specified failure criterion was reached. The first 2 shear stages of the multistage tests were stopped when the deviator stage leveled off or at a maximum 4% axial strain. The third shear stage was stopped at total axial strains ranging between 15 to 18% (Figure 6.10).

6.3.4 Data Analysis

A triaxial test determines the Mohr circle failure envelope to define the shear strength characteristics, cohesion (c') and the internal angle of friction (ϕ'). The strength properties used for Mohr-Coulomb failure theory are determined by defining three Mohr's circles at failure. The Mohr-Coulomb failure envelope of a normally consolidated soil, should plot as a straight line, indicating that both cohesion and the internal angle of friction are properties of the soil that remain constant despite the confining pressure.

The A_f value (Skempton, 1954) relates the change in pore pressure to the change in deviator stress and is defined as,

$$A_f = \frac{\Delta\mu}{\Delta\sigma_1} \qquad \text{Equation 6.5}$$

A_f values vary depending on soil stress state (Figure 6.11, Bishop and Henkel, 1962). For normally consolidated, saturated soils the change in pore pressure should be approximately equal to the load applied, reflected by an A_f value approximately equal to 1. Heavily overconsolidated soils, generally dilate when sheared and generates a negative change in pore water pressure resulting in a negative A_f value.

The normalized shear strength ratio, S_u/σ'_v in the normally consolidated stress (S) range was estimated using the average of maximum shear strength and confining pressure for the 2nd and 3rd confining pressures.

7.0 BENDER ELEMENTS

The bender element system uses a pair of bender elements, a source and a receiver, to send shear (S) waves or compressional (P) waves through a soil sample in the triaxial cell. The bender elements are made from piezoelectric ceramic bimorphs that protrude a small distance into the soil sample. An excitation voltage is used to produce a displacement in the source transducer resulting in a wave being propagated through the sample. This wave generates a displacement in the receiver which induces a voltage that can be read. The S wave source comprises two piezoelectric strips polarized in the same direction. When an excitation voltage is applied one strip extends and the other strip contracts causing the strips to *bend*. The P wave source comprises two piezoelectric strips polarized in opposite directions. Both strips extend when an excitation voltage is applied.

A typical schematic detailing a pair of vertical bender elements set in a triaxial specimen is shown in Figure 7.1. The difference between the two body wave types (often known as

‘Primary’, ‘Pressure’ or ‘Compression’, and ‘Secondary’ or ‘Shear’ respectively) is best described by the direction of soil particle motion with respect to the direction of wave propagation. P-waves are longitudinal, meaning the soil particles move in the same direction as that of the wave propagation, while S-waves are transverse, meaning the particles move in a direction perpendicular to that of the propagation (Figure 7.2).

The P-wave propagation is controlled by the bulk (K) and shear (G) moduli of the soil. P-waves are transmitted through water meaning sediment saturation may have a significant effect on the P-wave. The S-wave is controlled by the shear modulus (G) of the soil. S-waves are generally unaffected by the degree of saturation as water has a negligible shear modulus.

There is a marked difference between the P and S –wave test specifications used for the GDS bender element system. The P-wave results are obtained using a square wave source signal with a 4 ms period. The received signal is however of generally poor quality. In contrast the S-wave uses a sinusoidal wave as the source signal. The period can be varied between 0.1 and 0.9 ms. The specifications for the transmit signal is dependent on the quality of the receive signal. This is determined by the degree of noise in the received signal and the similarity between transmit and receive signals.

Practically speaking, the bender element test is used to obtain values of V_p and V_s at various confining pressures. This is done by recording the time, t, taken for the generated wave to travel from the transmitter element to the receiver element, then dividing the distance between the elements by this travel time. It should be noted that determining the travel time is not necessarily straightforward as it is often unclear at which exact time the propagated wave has arrived at the receiver element. This is especially true for the P wave tests due to the quality of the data. Although the S-wave data is of good quality there is still debate as to best for the method used in determining the travel time. GDS recommends that the method used for determining the S-wave arrival time should be selected by the user, based on recommendations in the geotechnical literature (Yamashita et al., 2009 and Camacho-Tauta et al., 2012).

An example of the S-wave data from triaxial sample 20130290048PC_483-494cm are illustrated in Figure 7.3 and Figure 7.4. The transmit and receive elements are in-phase therefore the first peak of the transmit and receive signals are positive (Figure 7.3). These data are of high quality with little noise and a good match between the send and receive signals. The strength of the receive signal's first peak is weak but readily distinguishable. In order to enhance the quality of the first return a series of tests are run using different transmit frequencies. The method used to determine the travel time for this study was the peak to peak method. The S-wave results were considered to be more consistent than using the first return method. The Bender Element system enables easy measurement of the maximum shear modulus *(G_{max})* of a soil at small strains in a triaxial cell.

GDS bender elements were used to measure shear wave and compressional wave velocities during the CIU test. The elements are made from piezoelectric ceramic bimorphs. Two sheets are bonded together with a metal shim in between. An excitation voltage is used to produce a displacement in the source transducer, resulting in a wave being sent through the sample. The system comprises 2 bender elements inserted into the top cap and base pedestal, external control box with a high speed 16 bit data acquisition and control card and GDS software. The software allows for stacking of data and user control of source signals.

8.0 SLOPE STABILITY

8.1 Introduction

Submarine slides in the Beaufort Sea represent a significant potential geohazards for offshore infrastructure and coastal communities. Submarine slides can be triggered by human activities, or external processes including seismic loading (earthquakes), over steepening and excess pore water pressures (Dimakis et al, 2000). Other than anthropogenic activity causing submarine slides, earthquakes are probably the most common trigger of offshore slope instability. Hance (2003) reported that among the 534 events in his database, over 40% of the slope failures were attributed to earthquake and faulting mechanisms (Figure 8.1). Assessing slope instability requires information on the frequency of failure, trigger mechanism, soil conditions and the morphology of the area.

8.2 Slope Stability Analysis

The limit equilibrium methods are most commonly used to assess the slope stability in a marine environment. The limit equilibrium analysis evaluates a well-defined body on a slope as if it is about to fail and determines the shear stress induced by varying trigger mechanisms. The shear stresses are then compared to the soils shear strength to determine the *FS* (equation 8.1) with the slope considered to be unstable if the FS is equal to or less than 1.

$$FS = \frac{Sediment\ Strength\ (Available\ Shear\ Strength)}{Trigger\ Mechanisms\ Stress\ (Driving\ Force)} \qquad \text{Equation 8.1}$$

The infinite slope method was used for the slope stability analysis. It assumes the failure surface is parallel to the slope, the slope is planar and of infinite length. The failure length of the slope is significantly greater than the failure thickness. The triggering mechanisms analysed were gravitational loading and earthquake loading.

8.2.1 Gravitational Loading

Gravity forces are a mechanism for general downslope movement causing slopes to fail or sediments to consolidate under their own weight. An analysis can be conducted for undrained (short term) or drained (long term) conditions. A *FS* can be calculated for ***undrained conditions or total stress analysis (TSA)*** if the slope angle, bulk density and the undrained shear strength of the sediment are known. Figure 8.2 illustrates the static condition under which the slice of soil is loaded. The gravitational force, *T* parallel to the slope is equal to the effective weight of the soil and the sediments shear strength. *R* is the undrained shear strength (S_u) times the length (l) along the base of the slice. From consideration of the equilibrium of the slope at the point of failure, the forces parallel to the slope are equal therefore,

$$S_u l = W' sin\beta \qquad \text{Equation 8.2}$$

where W' is the weight of the slice with width b and height h having a unit dimension into the page and equals,

$$W' = \gamma' bh \qquad \text{Equation 8.3}$$

where γ' is the submerged or effective unit weight of the soil and equates,

$$\gamma' = \gamma_{soil} - \gamma_{sea\ water} \qquad \text{Equation 8.4}$$

Since $l = b/cos\beta$, Equation 8.2 can be rewritten as,

$$S_u = \frac{\gamma' bhsin\beta cos\beta}{b} \qquad \text{Equation 8.5}$$

Using the double trigonometry identify sin 2β = 2 sinβcosβ Equation 8.5 is reduced to,

$$2S_u = \gamma' hsin2\beta \qquad \text{Equation 8.6}$$

The *FS* (Equation 8.1) is then calculated using Morgenstern's (1967) basic infinite slope analysis equation,

$$FS = \frac{2}{sin2\beta} \frac{S_u}{\gamma' h} \qquad \text{Equation 8.7}$$

The critical height (H_c) where *FS =1* can be obtained using,

$$H_c = \frac{2S_u}{\gamma' \sin 2\beta} \qquad \text{Equation 8.8}$$

The critical slope angle (β_c) where *FS =1* can is determined by,

$$\beta_c = \left[\frac{1}{2}\left\{sin^{-1}\left(\frac{2S_u}{\gamma' h}\right)\right\}\right] \qquad \text{Equation 8.9}$$

A FS for *__drained or effective stress conditions (ESA)__* can be calculated if the slope angle, bulk density and the Mohr Coulomb effective stress parameters c', ϕ' (see equation 6.1) are known. Considering the horizontal and vertical equilibrium of the slice the slope angle at failure can be shown to be:

$$tan\,\beta = tan\,\phi' + \frac{c'}{\gamma' \mathrm{h}} sec^2\beta \qquad \text{Equation 8.10}$$

For normally consolidated soils where c' is approximately zero, the slope angle at failure is approximately equal to ϕ' and the FS is

$$FS = tan\phi'/tan\beta \qquad \text{Equation 8.11}$$

A FS for *partially drained conditions* is calculated in a similar manner to the drained analysis. The difference is the presence of excess pore pressure (u_e) resulting in the reduction of the sediment's shear strength due to a decrease of vertical effective stress. Conditions which can result in the presence of excess pore pressure include presence of gas, disassociation of gas hydrates, or high sedimentation rates that would prevent complete dissipation of excess pore pressure (Poulos, 1988).

The effect of excess pore pressure on slope stability can be evaluated with Mohr-Coulomb strength parameters to calculate the FS using

$$FS = \frac{c' + \sigma \, cos^2 \, \beta tan\phi' - \mu_e tan\phi'}{\sigma' sin\beta cos\beta} \qquad \text{Equation 8.12}$$

where c' is the cohesion factor, σ is the total stress (kPa), β is the slope angle (°), ϕ' is the effective friction angle (°), σ is total stress (kPa), σ' is the effective stress (kPa), and μ_e is the excess pore pressure (kPa). Setting the FS = 1 allows the minimum excess pore pressure necessary to trigger a slope failure to be estimated using:

$$\mu_e = \frac{1}{tan\phi'}(c' + \sigma cos^2\beta tan\phi' - \sigma' sin\beta cos\beta) \qquad \text{Equation 8.13}$$

8.2.2 Earthquake Seismic Loading

A slope can become unstable due to seismicity effects induced by an earthquake. The seismic motions released by an earthquake have the ability to induce large destabilizing inertial forces. These forces act in the horizontal and vertical directions; however, the vertical forces have little impact on *FS* since vertical acceleration is generally less than horizontal acceleration. The stability of submarine slides triggered by earthquakes can be evaluated by using the Morgenstern (1967) pseudo-static approach (Figure 8.3) with an earthquake-induced horizontal acceleration (*k*) introduced into the static *FS* equation. Morgenstern's pseudo-static approach neglects vertical acceleration (Morgenstern 1967) and is presented as:

$$FS = \frac{S_u}{\gamma' h} * \frac{1}{\frac{1}{2} sin2\beta + kcos^2\beta\left(\frac{\gamma}{\gamma'}\right)} \qquad \text{Equation 8.14}$$

where S_u is undrained shear strength (kPa), γ is the unit weight, γ' is the submerged unit weight, h is the height and β is the slope angle.

Setting the *FS* = 1 allows the minimum peak horizontal ground acceleration coefficient (k_{min}) necessary to trigger a slope failure to be estimated using:

$$k_{min} = \frac{S_u}{\gamma' hcos^2\beta}\frac{\gamma'}{\gamma} - \frac{\gamma'}{\gamma} tan\beta \qquad \text{Equation 8.15}$$

The peak horizontal acceleration coefficient can be related to magnitude and distance from epicenter using attenuation relationships. In other words, given the study area is at X distance from a seismic activity zone, it would require a Y magnitude earthquake to produce a Z peak ground acceleration which would deem the slope unstable (Abrahamson and Shedlock, 1997).

There is an insufficient amount of ground-motion recordings to use a direct empirical estimation of the seismic motions expected from an earthquake capable of triggering a submarine slide (Boore et al., 1997). Hence, ground-motion prediction equations (GMPE), or attenuation relationships, can be used to determine expected ground motions from earthquakes. GMPEs are relationships developed from empirical and model data.

In Canada, four different GMPEs can be used depending on the variable physical properties of the crust in eastern and western Canada, and the different nature of the earthquake sources in southwestern Canada (Adams and Halchuk, 2004). Canada can be separated into three regional seismicity zones: East, West, and Stable (Figure 8.4)

The earthquake sources in southwest Canada are of different nature than those in eastern Canada. Hence, western Canada requires the use of strong ground motion relations, or GMPEs. Beaufort Sea is located within the western Canada seismicity zone. For shallow source zones, Boore et al. 1997 GMPEs can be used to relate distance and magnitude to peak ground acceleration (Adams and Halchuk, 2003).

Once the minimum peak horizontal ground acceleration is calculated, Boore et al 1997 GMPEs estimate distance and magnitude of an earthquake capable of triggering a submarine slide based on the critical horizontal acceleration (k_{min}). Boore et al.'s attenuation relationships are:

$$\ln(k_{min}) = b_1 + b_2(M-6) + b_3(M-6)^2 + b_5 ln\, r + b_V \ln\left(\frac{V_s}{V_A}\right) \quad \text{Equation 8.16}$$

where,

$$r = \sqrt{r_{jb}^2 + h^2} \qquad \text{Equation 8.17}$$

and,

$$b_1 = \begin{cases} b_{1SS} \\ b_{1RS} \\ b_{1ALL} \end{cases} \qquad \text{Equation 8.18}$$

where M is moment magnitude, r_{jb} is the distance from the study site to a seismic activity zone (km), and V_s is the averaged shear-wave velocity (m/s) of the upper 30 m. V_s can be determined by using either NBCC 2005 values based on seismic site class, or by extrapolating from the best fit of total effective overburden pressure calculated using MSCL bulk density data vs depth, and bender elements shear-wave analysis during triaxial tests. It should be noted that the most appropriate seismic site class to use for the Beaufort Sea is Site Class E ($V_s < 180$ m/s). The other coefficients to be used for minimum peak horizontal ground acceleration have been determined by Boore et al, 1997 and their values are listed in the table below.

Coefficient	**Value**
b_{1SS}, for strike-slip earthquakes	-0.313
b_{1RV}, for reverse-slip earthquakes	-0.117
b_{1ALL}, for earthquakes with no specified mechanism	-0.242
b_2	0.527
b_3	0
b_5	-0.778
b_V	-0.371
V_A, in m/s	1396
h	5.57

9.0 GEOTECHNICAL RESULTS OF INDIVIDUAL CORES

9.1 Introduction

The Canadian Beaufort Sea study area has been divided into 4 regions: Region 1 – Western; Region 2 – Central; Region 3 – Eastern; and, Region 4 – Banks Island (Figure 9.1). Each region contains submarine failures. The region selection is rather arbritray and partly reflects the geographic focus of the hydrocarbon industry and the quality and quantity of data collected. The Western Region, is controlled by Chevron and has a distinct cahnge in morpholy. The Central region (Ajurak;Imperial Oil Resources Ventures Limited) is marked by large failure near the shelf break while the Eastern Region (Pokak; British Petroleum Canada) has a generally smooth morphology and is considered to be reference seabed. The Banks area, a far field "control" , is both more distant from the earthquake cluster and supplied sediment from the Amundsen Gulf and M'Clure Strait glaciers.

Geotechnical testing included MSCL bulk density, Atterberg limits, grain size analysis, back-pressured consolidation tests and isotropically consolidated undrained (CIU) triaxial tests. A summary of the type and number of tests completed for each core are presented in Table 9.1.

MSCL bulk density measurements taken at 1 cm intervals are used to develop a continuous bulk density profile. Natural *in-situ* water contents *(*w_n*) w*ere measured at 75 cm intervals. Atterberg limit samples are representative of the different lithologies and correspond to natural water content measurements. Stress history, compressibility and hydraulic conductivity were determined from consolidation tests.

MV measurements were taken at 10 cm intervals and remoulded shear strengths at 75 cm intervals. The Mohr-Coulomb failure parameters, and *S* ratios were determined from CIU multi-stage triaxial tests. Continuous undrained strength profiles for normal consolidation were complied using the SHANSEP method. Comparisons of the MV data and the strength profiles estimates the stress history.

Slope instability was determined using the infinite slope method. The minimum *FS*, critical height, critical slope angle and minimum horizontal acceleration coefficient were calculated for the length of each core. ***It should be noted that the FS is high in the upper 2 to 3 m due to the high Su/σ'_v values. Therefore sites with limiited core recovery will present as very stable.***

The geotechnical data were compiled into geotechnical profiles where the various soils properties are presented as a function of depth. The development of the geotechnical profile results from many stages of development including natural consolidation due to gravitional compaction and depostional or eroisonal events. The profiles include down core plots of lithology, grain size, density, natural water content, plastic and liquid limits, undrained shear strengths, effective overburden stress and P'_c values. The geotechnical results for each region are presented in the following sections.

9.2 Region 1 – Western

Region 1 is located in western Beaufort Sea, Northern Mackenzie Trough to outer slope (Figure 9.2). ArcticNet and Araon-collected multibeam bathymetry depict shelf-break canyons and two large N-S oriented exhumed ridges (note that colour schemes between the two data sets do not blend). Despite the obvious mass failure elements, the sub-bottom profiler and core data demonstrate a post-glacial stratified blanket uniformly and conformably covering both the plains and the ridges. The 3.5 kHz sub-bottom profiles illustrating the acoustic stratigraphy and position (if possible) for the cores in region 1 are presented in Figure 9.3.

9.2.1 Piston Core 20148040029

Introduction

Piston core 20148040029 was collected in the channel between the two ridges at a water depth of 1500 m and a slope angle of 3.4°, determined from multi-beam data (Figure 9.2). The core recovered 375 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and position for core 20148040029 is presented in Figure 9.3b.

Atterberg Limits

Atterberg limit tests were completed on 5 samples (Figure 9.4). The samples were of intermediate to very high plasticity. The liquid limits range from 46.6 to 85.5% and the plastic limits range from 29.3 to 43.2%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 330-332.5 cm (Figure 9.5) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.39 and recompression index, C_r, of 0.077. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 3.72E-08 cm/sec. The consolidation test characterizes the sediment as slightly over consolidated with respect to the effective overburden with OCR value of 1.70.

Triaxial Test Results

A CIU triaxial test was conducted on 1 sample at a core depth of 318-330 cm (Figure 9.6). The Mohr-Coulomb failure envelope is defined by effective friction angle (ϕ') of 23.3° and effective cohesion value (c') of 0.0 kPa. The pore pressure coefficient at failure (A_f) is 0.54. The S ratio is 0.28. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.6948 m/s kPa^{-1} and 44.63 m/s (Figure 9.7).

Slope Stability

The *FS* was calculated at slope angles of 1°, 3.4°, 5°, 10°, 15°, and 20° (Figure 9.8). The slope is stable under static conditions with a minimum *FS* of 4.5 at a core depth of 300 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 27 m, 16.2°, and 0.08, respectively.

Geotechnical Profile

The sediment consists of silty clay to clayey silt. Gravitational compaction is the dominant feature in this core with gradual increase in density interrupted by variable density from 30 to 150 cm corresponding with interbedded layers with varying amounts of silt (Figure 9.9).

The grain size fractions are variable with < 5% sand, 40-50% silt, and 50% clay in the upper 1 m. Sand content increases to < 15% below 1 m, silt content decreases to < 40%, and clay content remains uniform ranging from 50 to 55%. Water contents are variable with depth following a general decreasing trend with the exception of an abrupt decrease to 60% and increase between 50 and 150 cm. The liquidity indices decrease from 2 to 1.

A consolidation test at 330 cm suggests the sediments are over consolidated with an OCR value of 1.7. MV measurements are uniform and show a slight decrease with depth and lie above S values. A CIU triaxial test at 318 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS of 4.5 indicates the core is stable under gravitational loading.

9.2.2 Jumbo Piston Cores 2013HEALY0001 & 0003

Introduction

Piston core 2013HEALY0001 was collected from the Mackenzie Trough in the Chevron region at a water depth of 685 m and a slope angle of 0.26°, determined from multi-beam data (Figure 9.2). The core recovered 1412.5 cm of sediment. Piston core 2013HEALY0003 was taken as a duplicate to piston core 2013HEALY0001, and was extruded at a water depth of 665 m. The duplicate core recovered 1643.5 cm. All geotechnical data, with the exception of grain size, use sediment collected in piston core 2013HEALY0001. The 3.5 kHz sub-bottom profile illustrating the equivalent acoustic stratigraphy core Healy 20130001 is presented in Figure 9.3a.

Atterberg Limits

Atterberg limit tests were completed on 9 samples (Figure 9.10). The samples range from intermediate to very high plasticity. The liquid limits range from 44.4 to 78.7% and the plastic limits range from 25.2 to 36.2%. There is a general decrease in plasticity with core depth. Two samples at depths of 1112-1122 cm and 1340-1350 cm plot in Zone B liquefaction potential identified by Seed et al., 2003.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 1340-1342.5 cm (Figure 9.11) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.53 and recompression index, C_r, of 0.068. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 7.89E-08 cm/sec. The consolidation test characterizes the sediment as under consolidated with respect to the effective overburden with OCR value of 0.54.

Triaxial Test Results

A CIU triaxial test was conducted on 1 sample at a core depth of 1344-1356 cm (Figure 9.12). The Mohr-Coulomb failure envelope is defined by effective friction angle (ϕ') of 26.73° and effective cohesion value (c') of 0.0 kPa. The pore pressure coefficient at failure (A_f) is 0.73. The S ratio is 0.28. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.4976 m/s kPa^{-1} and 100.21 m/s (Figure 9.13).

Slope Stability

The *FS* was calculated at slope angles of 0.3°, 1°, 5°, 10°, 15°, and 20° (Figure 9.14). The slope is stable under static conditions with a minimum *FS* of 20.8 at a core depth of 1250 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 220 m, 5.4°, and 0.04, respectively.

Geotechnical Profile

The sediment consists of clayey silt with minimal sand near the base of the core. Gravitational compaction is the dominant feature in this core, with gradual increase in density and decrease in water content with depth. (Figure 9.15)

The grain size fractions are uniform with 30 to 40% clay, 60 to 70% silt, and <10% sand. Water contents decrease abruptly from 125 to 80% from 0 to 2 m, then decreases gradually from 2 m to the base of the core. The liquidity indices are uniform with depth, ranging from 1 to 1.5, suggesting possible under consolidation.

A consolidation test at 1340 cm suggests the sediments are under consolidated with an OCR value of 0.54. MV measurements are uniform and increase with depth, matching S values from 0 to 7.5 m and fall below S values at depths greater than 7.5 m. There is apparent coring disturbance near the base of the core. A CIU triaxial test at 1344 cm consolidated to in-situ effective overburden pressure correlate with the upper limit of the S value range. The minimum FS > 20 indicates the core is stable under gravitational loading.

9.2.3 Gravity Core 2013004PGC0066

Introduction

Gravity core 2013004PGC0066 was collected by GSC-P in the channel between the two ridges, in close proximity to 20148040029pc, at a water depth of 1536 m and a slope angle of 0.7° determined from multi-beam data (Figure 9.2),. The core recovered 387 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 3 samples (Figure 9.16). The samples were of high to very high plasticity. The liquid limits range from 61.9 to 87.6% and the plastic limits range from 33.3 to 57.9%.

Consolidation Test Results

Consolidation tests were conducted on 3 samples at core depths of 116-118.5 cm, 188-190.5 cm and 362-364.5 cm (Figure 9.17a and 9.17b) to assess the compressibility (C_c), hydraulic conductivities (k) and stress histories (OCR). The compressibility are moderate to high with compression indices, C_c, of 0.85, 0.89 and 0.53, and recompression indices, C_r, of 0.089, 0.129 and 0.107. The hydraulic conductivities, k, at the void ratio equivalent to the P'_c are 2.65E-07 cm/sec, 1.16E-07 cm/sec and 1.92E-07 cm/sec. The consolidation tests characterize the sediment as over to normally consolidated with respect to the effective overburden with OCR values of 2.14, 1.85 and 1.05.

Triaxial Test Results

CIU triaxial tests were conducted on 3 samples at core depths of 100-108 cm, 191-203 cm and 366-374 cm (Figure 9.18a, 9.18b and 9.18c). The Mohr-Coulomb failure envelopes are defined by effective friction angles (ϕ') of 25.3°, 20.7° and 21.5° and effective cohesion values (c') of 0.0, 0.6 and 0.0 kPa. The pore pressure coefficients at failure (A_f) are 0.78, 0.34 and 0.72. The S ratios are 0.27, 0.22 and 0.20. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.8627 m/s kPa^{-1} and 38.27 m/s for a core depth of 191.0 -203.0 cm (Figure 9.19).

Slope Stability

The *FS* was calculated at slope angles of 0.7°, 1°, 5°, 10°, 15°, and 20° (Figure 9.20). The slope is stable under static conditions with a minimum *FS* > 30 at a core depth of 235 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 55 m, 30.3°, and 0.15, respectively.

Geotechnical Profile

The sediments consists of silty clay to clayey silt. Gravitational compaction is the dominant feature in this core with gradual increase in density and decrease in water content with depth. (Figure 9.21)

There was only one subsample for grain size analysis. The grain size contents indicates the sediment consists of 40% silt, 60% clay, with little sand at 362 cm. Water contents decrease linearly from 110 to 65% in the upper 1.5 m, then is uniform to the base of the core, except at 1.75 m where water content increases to 100%. The liquidity indices increase with depth from 1 to 1.25.

Three consolidation tests at 116, 188, and 362 cm suggests the sediments range from apparently over consolidated in the upper 2 m with OCR values of 2.14 and 1.85, then become normally consolidated at 3.6 m with an OCR value of 1.05. MV measurements increase linearly with depth and lie above S values. CIU triaxial tests at 100, 191, and 366 cm consolidated to in-situ effective overburden pressure correlate with MV measurements. The minimum FS > 30 suggests the core is stable under gravitational loading.

9.2.4 Summary

The sediments in Region 1 range from intermediate to very high plasticity and plot along the A-line (Figure 9.22). Gravitational compaction is the dominant feature in all cores within Region 1 characterized by a linear increase in density with depth with the exception of piston core 20148040029 where there is variable density corresponding to interbedded silt and clay rich layers. Water contents show general decrease with depth with the exception highlighted above. Table 9.2 summarizes all Atterberg test results for Region 1.

The colloidal activity ranges from inactive to normal for cores in Region 1. Clay fractions are uniform but distinct for each core. Plasticity indices are variable ranging from 10 to 40% (Figure 9.23).

A total of 5 consolidation tests were performed on samples within Region 1 and indicate the sediments are apparently over consolidated in the upper 2 m with OCR values decreasing from 2.1 to 1.7, then normally consolidated at 3.6 m with an OCR value of 1.13, then under consolidated at 13.4 m with OCR < 1 (Figure 9.24). Table 9.3 summarizes consolidation test results for Region 1.

A total of 5 CIU triaxial tests were performed on samples within Region 1 and indicate S values range from 0.20 to 0.28. Table 9.4 summarizes the CIU triaxial test results of samples from Region 1.

A comparison of MV data to effective overburden pressure (Figure 9.25) shows good correlation between the cores within Region 1. MV data indicates high strength sediments in the upper 20 kPa of effective overburden pressure, with the exception of two normal strength measurements in

2013HEALY0001pc and 1 normal strength measurement in 20148040029pc. Below 20 kPa, sediment gradually become weaker with increasing effective overburden pressure and are of low strength below 50 kPa. Sediments of low strength are possibly under consolidated and have the potential for excess pore pressure.

FS calculated at various slope angles for cores within Region 1 indicate that sediments at all core sites are presently stable under gravitational loading (Figure 9.26). Shallower sediments (upper 4 m) require a slope angle greater than 12.8° to become potentially unstable. Deeper sediments require a slope angle greater than 5° to become potentially unstable. Table 9.5 summarizes the stability analysis under gravitational loading for all core sites within Region 1.

9.3 Region 2 – Central

Region 2 is located on the western Kugmallit Fan (Figure 9.27) and includes a large slide complex termed Ikit Slide Valley Complex (Cameron and King, 2018). The Ikit Slide is about 24 km wide, along the shelf break, extending beyond 55 km (limit of multibeam coverage) downslope with an area of over 2000 km2 involving an estimated 45-50 cubic kilometres of failed sediment. The eastern most area consists of unfailed sediments. Geotechnical work was conducted on cores collected from the Amundsen and the CCGS Laurier between 2009 and 2016 (Figure 9.27). The 3.5 kHz sub-bottom profiles illustrating the acoustic stratigraphy and position (if possible) for the cores in region 1 are presented in Figure 9.28.

9.3.1 Piston Core 20148040012

Introduction

Piston core 20148040012 was collected from failed sediment in the Ajurak Block at a water depth of 601 m and a slope angle of 2.1°, determined from multi-beam data (Figure 9.27). The core recovered 485.5 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and position for core 20148040012 is presented in Figure 9.28a.

Atterberg Limits

Atterberg limit tests were completed on 5 samples (Figure 9.29). The samples range from high to very high plasticity. The liquid limits range from 65.7 to 79.1% and the plastic limits range from 32.2 to 36.4%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 425-427.5 cm (Figure 9.30) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.42 and recompression index, C_r, of 0.146. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 4.43E-08 cm/sec. The consolidation test characterizes the sediment as under consolidated with respect to the effective overburden with OCR value of 0.71.

Triaxial Test Results

A CIU triaxial test was conducted on 1 sample at a core depth of 432-444 cm (Figure 9.31). The Mohr-Coulomb failure envelope are defined by effective friction angle (ϕ') of 20.21° and effective cohesion value (c') of 3.2 kPa. The pore pressure coefficient at failure (A_f) is 0.51. The

S ratio is 0.27. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.6513 m/s kPa^{-1} and 58.73 m/s (Figure 9.32).

Slope Stability

The *FS* was calculated at slope angles of 1°, 2.1°, 5°, 10°, 15°, and 20° (Figure 9.33). The slope is stable under static conditions with a minimum *FS* of 10.8 at a core depth of 340 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 38 m, 26.7°, and 0.14, respectively.

Geotechnical Profile

The sediment consists of silty clay with minimal sand. Gravitational compaction is the dominant feature in this core, with increase in density and decrease in water content with depth. (Figure 9.34).

The grain size fractions are uniform throughout the core with 50 to 60% clay, 40 to 50% silt, and <10% sand. Water contents decrease abruptly from 125 to 75% in the upper 1.5 m, then decreases gradually from 1.5 m to the base of the core. The liquidity indices are uniform with depth, ranging from 1 to 2. A consolidation test at 425 cm suggests the sediments are under consolidated with an OCR value of 0.71. MV measurements increase uniformly with depth; however, the measurements lie above S values. The gap between MV measurements and S values suggests possible coring disturbance. A CIU triaxial test at 432 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS < 20 indicates the core is stable under gravitational loading.

9.3.2 Piston Core 20148040016

Introduction

Piston core 20148040016 was collected from failed sediment in the deep canyon in the Ajurak Block at a water depth of 1247 m and a slope angle of 0.9° (Figure 9.27), determined from multi-beam data. The core recovered 338 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and position for core 20148040016 is presented in Figure 9.28b.

Atterberg Limits

Atterberg limit tests were completed on 8 samples (Figure 9.35). The samples range from high to extreme plasticity. The liquid limits range from 54.3 to 93.4% and the plastic limits range from 24.9 to 36.6%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 201-203.5 cm (Figure 9.36) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.44 and recompression index, C_r, of 0.101. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 3.06E-08 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value of 1.79.

Triaxial Test Results

A CIU triaxial test was conducted on 1 sample at a core depth of 208-220 cm (Figure 9.37). The Mohr-Coulomb failure envelope are defined by effective friction angle (ϕ') of 15.9° and effective cohesion value (c') of 3.9 kPa. The pore pressure coefficient at failure (A_f) is 0.57. The S ratio is

0.22. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.7375 m/s kPa^{-1} and 55.96 m/s (Figure 9.38).

Slope Stability

The *FS* was calculated at slope angles of 0.9°, 1°, 5°, 10°, 15°, and 20° (Figure 9.39). The slope is stable under static conditions with a minimum *FS* of 25.2 at a core depth of 280 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 52 m, 28.1°, and 0.18, respectively.

Geotechnical Profile

The sediment consists of silty clay to clayey silt. Gravitational compaction is the dominant feature in this core, with a general increasing trend in density and decrease in water content with depth. (Figure 9.40).

The grain size fractions are predominantly uniform with < 5% sand, 40 to 50% silt, and 50 to 60% clay, except between 1.75 to 2.75 m where silt and clay contents match closely with 45 to 55%. Water contents decrease abruptly from 125 to 50% in the upper 2 m, then varies between 40 and 50% to the base of the core. The liquidity indices decrease linearly with depth from 1.5 to 0.5.

A consolidation test at 201 cm suggests the sediments are over consolidated with an OCR value of 1.79. MV measurements increase uniformly with depth in the upper 2m, then becomes highly variable ranging from 5 kPa to a peak shear strength of 37 kPa. This peak MV measurement seems to correlate with the high clay and low silt fractions below 2 m. An erosion calculation was performed using peak MV measurements and estimates the thickness of sediment removed to be between 15 (S = 0.2) and 23.7 m (S = 0.3) of sediment. A CIU triaxial test at 208 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS < 30 indicates the core is stable under gravitational loading.

9.3.3 Piston Core 20148040019

Introduction

Piston core 20148040019 was collected from unfailed sediment from above the shallowest failure scarp in the Ajurak Block at a water depth of 133 m and a slope angle of 2.7° (Figure 9.27), determined from multi-beam data. The core recovered 296 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and for core 20148040019 is presented in Figure 9.28c.

Atterberg Limits

Atterberg limit tests were completed on 3 samples (Figure 9.41). The samples were of high plasticity. The liquid limits range from 58.7 to 59.0% and the plastic limits range from 31.2 to 32.9%.

Consolidation Test Results

A consolidation test was conducted on 2 samples at a core depths of 92-94.5 cm and 200-202.5 cm (Figure 9.42) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression indices, C_c, of 0.59 and 0.58 and recompression indices, C_r, of 0.090 and 0.089. The hydraulic conductivities, k, at the void ratio equivalent to the P'_c is 1.16E-07 cm/sec and 1.42E-07 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with *OCR* values of 5.34 and 2.72.

Triaxial Test Results

A CIU triaxial test was conducted on 1 sample at a core depth of 210-222 cm (Figure 9.43). The Mohr-Coulomb failure envelope are defined by effective friction angle (ϕ') of 24.6° and effective cohesion value (c') of 0.0 kPa. The pore pressure coefficient at failure (A_f) is 0.73. The S ratio is 0.26. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.5489 m/s kPa^{-1} and 51.79 m/s (Figure 9.44).

Slope Stability

The *FS* was calculated at slope angles of 1°, 2.7°, 5°, 10°, 15°, and 20° (Figure 9.45). The slope is stable under static conditions with a minimum *FS* of 9.6 at a core depth of 235 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 33 m, 32.8°, and 0.17, respectively.

Geotechnical Profile

The sediment consists of silty clay. Gravitational compaction is the dominant feature in this core, with a uniform density profile and linear decrease in water content with depth. The liquidity indices decrease linearly with depth from 1.5 to 1. (Figure 9.46)

Two consolidations tests at 92 and 200 cm suggests the sediments are over consolidated with OCR values of 5.34 and 2.72, respectively. This may be indicative of apparent over consolidation which exists in the upper 2 m. MV measurements increase uniformly with depth and lie above S values. A CIU triaxial test at 210 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS < 10 indicates the core is stable under gravitational loading.

9.3.4 Piston Core 20148040024

Introduction

Piston core 20148040024 was collected from the ridge between the canyons in the Ajurak Block at a water depth of 1065 m and a slope angle of 1.2° (Figure 9.27), determined from multi-beam data. The core recovered 334.5 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and position for core 20148040024 is presented in Figure 9.28d.

Atterberg Limits

Atterberg limit tests were completed on 5 samples (Figure 9.47). The samples were of high to very high plasticity. The liquid limits range from 59.0 to 93.6% and the plastic limits range from 31.1 to 41.5%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 267-269.5 cm (Figure 9.48) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.44 and recompression index, C_r, of 0.087. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 1.04E-07 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value of 2.26.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 277-289 cm (Figure 9.49). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 19.1° and effective cohesion value (c') of 4.1 kPa. The pore pressure coefficient at failure (A_f) is 0.53. The S ratio is 0.26. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.6786 m/s kPa^{-1} and 60.92 m/s (Figure 9.50).

Slope Stability

The *FS* was calculated at slope angles of 1°, 1.2°, 5°, 10°, 15°, and 20° (Figure 9.51). The slope is stable under static conditions with a minimum *FS* of 36.5 at a core depth of 240 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 63 m, >45°, and 0.30, respectively.

Geotechnical Profile

The sediment consists of silt and clay. Gravitational compaction is the dominant feature in this core with increase in density and decrease in water content with depth. (Figure 9.52)

The grain size fractions are uniform in with 50 to 60% clay, 40 to 50% silt, and < 5% sand. Water contents decrease abruptly from 150 to 70% in the upper 1.5 m, then decreases linearly from 1.5 m to the base of the core. The liquidity indices decrease linearly with depth, ranging from 2 to 0.5.

A consolidation test at 267 cm suggests the sediments are over consolidated with an OCR value of 2.26. MV measurements increase uniformly with depth and lie above S values. A CIU triaxial test at 277 cm consolidated to in-situ effective overburden pressure lies between MV measurements and S values. The minimum FS > 30 suggests the core is stable under gravitational loading.

9.3.5 Piston Core 20098040013

Introduction

Piston core 20098040013 was collected from the shelf in Ajurak Block at a water depth of 69 m and a slope angle of 0.2° (Figure 9.27), determined from multi-beam data. The core recovered 307 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 4 samples (Figure 9.53). The samples were of intermediate to very high plasticity. The liquid limits range from 38.9 to 87.0% and the plastic limits range from 21.4 to 35.0%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 127-129.5 cm (Figure 9.54) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is high with compression index, C_c, of 0.73 and recompression index, C_r, of 0.074. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 1.59E-07 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value of 2.81.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 108-120 cm (Figure 9.55). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 18.8° and effective cohesion value (c') of 2.0 kPa. The pore pressure coefficient at failure (A_f) is 0.52. The S ratio is 0.23. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 1.7847 m/s kPa^{-1} and 76.41 m/s (Figure 9.56).

Slope Stability

The *FS* was calculated at slope angles of 0.2°, 1°, 5°, 10°, 15°, and 20° (Figure 9.57). The slope is stable under static conditions with a minimum *FS* of 149.4 at a core depth of 288 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 520 m, >45°, and 0.27, respectively.

Geotechnical Profile

The sediment consists of silty clay to clayey silt. Gravitational compaction is the dominant feature in this core with increase in density and decrease in water content with depth. (Figure 9.58)

The grain size fractions are variable ranging from 4 to 10% sand, 40 to 50% silt, and 40 to 55% clay. Water contents decrease linearly from 100 to < 50%. The liquidity indices are uniform with depth, ranging from 0.9 to 1.2.

A consolidation test at 127 cm suggests the sediments are apparently over consolidated with an OCR value of 2.81. MV measurements increase uniformly with depth and lie above S values. A

CIU triaxial test at 108 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS > 30 suggests the core is stable under gravitational loading.

9.3.6 Piston Core 20098040019

Introduction

Piston core 20098040019 was collected in failed material downslope and in proximity to the headwall of the failure in Ajurak Block at a water depth of 193 m and a slope angle of 3.3° (Figure 9.27), determined from multi-beam data. The core recovered 415 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 4 samples (Figure 9.59). The samples were of high plasticity. The liquid limits range from 60.9 to 66.5% and the plastic limits range from 25.6 to 32.4%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 227-229.5 cm (Figure 9.60) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate to high with compression index, C_c, of 0.60 and recompression index, C_r, of 0.093. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 6.41E-07 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value of 2.01.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 212-224 cm (Figure 9.61). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 26.9° and effective cohesion value (c') of 1.3 kPa. The pore pressure coefficient at failure (A_f) is 0.46. The S ratio is 0.29. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.7622 m/s kPa^{-1} and 52.07 m/s (Figure 9.62).

Slope Stability

The *FS* was calculated at slope angles of 1°, 3.3°, 5°, 10°, 15°, and 20° (Figure 9.63). The slope is stable under static conditions with a minimum *FS* of 7.2 at a core depth of 370 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 28 m, 27.3°, and 0.13, respectively.

Geotechnical Profile

The sediment consists of silty clay. Gravitational compaction is the dominant feature in this core with increase in density and decrease in water content, except between 20 to 220 cm where higher density offsets from the linear increase. Water contents decrease from 80 to 50%, then remains consistent for the remainder of the core. (Figure 9.64)

The grain size fractions are uniform with depth with < 10% sand, 40 to 50% silt, and 45 to 55% clay. Water contents are uniform with depth and range from 60 to 80%. The liquidity indices are uniform with depth ranging from 1 to 1.2.

A consolidation test at 227 cm suggests the sediments are over consolidated with an OCR value of 2.01. MV measurements increase uniformly with depth from 0 to 2.2 m, then decrease linearly to the base of the core. MV measurements lie above S values. A CIU triaxial test at 212 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS of 7.2 suggests the core is stable under gravitational loading.

9.3.7 Piston Core 20098040026

Introduction

Piston core 20098040026 was collected from failure in Ajurak Block at a water depth of 469 m and a slope angle of 2.5° (Figure 9.27), determined from multi-beam data. The core recovered 590 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 7 samples (Figure 9.65). The samples were of high to very high plasticity. The liquid limits range from 62.4 to 84.7% and the plastic limits range from 29.7 to 36.5%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 252-254.5 cm (Figure 9.66) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate to high with compression index, C_c, of 0.59 and recompression index, C_r, of 0.036. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 1.46E-04 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value of 2.25.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 232-244 cm (Figure 9.67). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 23.6° and effective cohesion value (c') of 0.2 kPa. The pore pressure coefficient at failure (A_f) is 0.55. The S ratio is 0.29. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.8048 m/s kPa^{-1} and 47.46 m/s (Figure 9.68).

Slope Stability

The *FS* was calculated at slope angles of 1°, 2.5°, 5°, 10°, 15°, and 20° (Figure 9.69). The slope is stable under static conditions with a minimum *FS* of 8.6 at a core depth of 400 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 23 m, 24.0°, and 0.13, respectively.

Geotechnical Profile

The sediments consist of clay and silt. Gravitational compaction is the dominant feature in this core with uniform density and decrease in water content with depth. (Figure 9.70)

The grain size fractions are uniform with < 6% sand, 40 to 50% silt, and 45 to 60% clay. Water contents decrease abruptly from 125 to 70% in the upper 1.5 m, then decreases linearly from 1.5 m to the base of the core. The liquidity indices decrease linearly with depth, ranging from 2 to 0.8.

A consolidation test at 252 cm suggests the sediments are over consolidated with an OCR value of 2.25. MV measurements increase uniformly with depth and lie above S values. MV measurements become more variable below 220 cm. A CIU triaxial test at 232 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS of 8.6 suggests the core is stable under gravitational loading.

9.3.8 Piston Core 20098040036

Introduction

Piston core 20098040036 was collected from unfailed material in Ajurak Block at a water depth of 444 m and a slope angle of 2.5° (Figure 9.27), determined from multi-beam data. The core recovered 721 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 8 samples (Figure 9.71). The samples were of high to very high plasticity. The liquid limits range from 67.4 to 82.3% and the plastic limits range from 21.9 to 35.3%.

Consolidation Test Results

Consolidation tests were conducted on 2 samples at a core depths of 230-232.5 cm and 701-703.5 cm (Figure 9.72) to assess the compressibility (C_c), hydraulic conductivities (k) and stress histories (OCR). The compressibility are moderate to high with compression indices, C_c, of 0.77 and 0.67, and recompression indices, C_r, of 0.054 and 0.085. The hydraulic conductivities, k, at the void ratio equivalent to the P'_c are 4.65E-07 cm/sec and 1.25E-07 cm/sec. The consolidation tests characterize the sediment as over consolidated with respect to the effective overburden with OCR values of 1.21 and 1.29.

Triaxial Test Results

Isotropic CIU triaxial tests were conducted on 2 samples at a core depths of 241-253 cm and 688-700 cm (Figure 9.73). The Mohr-Coulomb failure envelopes are defined by effective friction angles (ϕ') of 20.6° and 21.3° and effective cohesion values (c') of 0.6 kPa and 3.6 kPa. The pore pressure coefficients at failure (A_f) are 0.61 and 0.78. The S ratios are 0.22 and 0.24. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slopes and intercepts of 0.6990 and 0.4971 m/s kPa^{-1} and 48.12 and 70.14 m/s (Figure 9.74a and 9.74b).

Slope Stability

The *FS* was calculated at slope angles of 1°, 2.5°, 5°, 10°, 15°, and 20° (Figure 9.75). The slope is stable under static conditions with a minimum *FS* of 6.7 at a core depth of 555 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 28 m, 18.3°, and 0.09, respectively.

Geotechnical Profile

The sediment consists of clay and silt. Gravitational compaction is the dominant feature in this core with uniform density and decrease in water content with depth. (Figure 9.76)

The grain size fractions are uniform with depth with < 10% sand, 40 to 45% silt, 45 to 50% clay. Water content decrease abruptly from 125 to 80% in the upper 1 m, then is uniform from 1 m to the base of the core. The liquidity indices follow the water content trend and range from 2.2 to 0.9.

Consolidation tests at 230 and 701 cm suggests the sediments are normal to slightly over consolidated with OCR values of 1.21 and 1.29, respectively. MV measurements increase uniformly with depth and lie above S values. CIU triaxial tests at 241 and 688 cm consolidated to in-situ effective overburden pressure correlate with MV measurements. The minimum FS of 6.7 suggests the core is stable under gravitational loading.

9.3.9 Piston Core 20098040040

Introduction

Piston core 20098040040 was collected from the shelf in Ajurak Block at a water depth of 74 m and a slope angle of 0.1° (Figure 9.27), determined from multi-beam data. The core recovered 198 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 3 samples (Figure 9.77). The samples were of intermediate to high plasticity. The liquid limits range from 38.0 to 85.9% and the plastic limits range from 23.7 to 34.1%.

Slope Stability

The *FS* was calculated at slope angles of 0.1°, 1°, 5°, 10°, 15°, and 20° (Figure 9.78). The slope is stable under static conditions with a minimum *FS* of 421.2 at a core depth of 126 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 966 m, >45°, and 0.30, respectively.

Geotechnical Profile

The sediments consist of clay and silt to sandy clayey silt. Gravitational compaction is the dominant feature in this core with uniform density and a decrease in water content with depth in the upper 1.25 m. Since core 20098040040 is located on the shelf, the core reaches a distinct high density high shear strength unit below 1.25 m characterized by a large abrupt increase in density and undrained shear strength and abrupt decrease in water content. (Figure 9.79)

The grain size fractions are uniform in the upper 1.25 m with < 15% sand, 40 to 45% silt, and 45 to 50% clay. Below 1.25 m, grain size fractions change to 20% sand, 60% silt, and 20% clay. Water contents decrease linearly from 95 to 75% in the upper 1 m, then decrease abruptly to 30%. The liquidity indices follow a similar trend to water content hovering around 1 in the upper 1 m, then decreasing abruptly to 0.2. The minimum FS > 30 suggests the core is stable under gravitational loading.

9.3.10 Piston Core 2012004PGC0025

Introduction

Piston core 2012004PGC0025 was collected from the headwall in the Ajurak Block by GSC-P at a water depth of 138 m and a slope angle of 1.7° (Figure 9.27), determined from multi-beam data. The core recovered 501 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 2 samples (Figure 9.80). The samples were of high plasticity. The liquid limits were 59.0 and 60.1% and the plastic limits were 26.7 and 28.4%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 384-386.5 cm (Figure 9.81) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.42 and recompression index, C_r, of 0.093. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 9.70E-08 cm/sec. The consolidation test characterizes the sediment as normally consolidated with respect to the effective overburden with OCR value of 0.61.

9.3.11 Piston Core 2012004PGC0026

Introduction

Piston core 2012004PGC0026 was collected from above the headwall in the Ajurak Block by GSC-P at a water depth of 107 m and a slope angle of 2.2° (Figure 9.27), determined from multi-beam data. The core recovered 493 cm of sediment.

Atterberg Limits

Atterberg limit tests were completed on 1 sample (Figure 9.82). The samples was of high plasticity. The liquid limit was 60.1% and the plastic limit was 26.2%.

Consolidation Test Results

Consolidation tests were conducted on 2 samples at a core depths of 133-135.5 cm and 315-317.5 cm (Figure 9.83a and b) to assess the compressibility (C_c), hydraulic conductivities (k) and stress histories (OCR). The compressibility are moderate with compression indices, C_c, of 0.50 and 0.47, and recompression indices, C_r, of 0.107 and 0.087. The hydraulic conductivities, k, at the void ratio equivalent to the P'_c are 1.06E-07 cm/sec and 6.74E-08 cm/sec. The consolidation tests characterize the sediment as normally and over consolidated with respect to the effective overburden with *OCR* values of 0.34 and 1.47.

9.3.12 Summary

The sediments in Region 2 predominantly range from high to very high plasticity and plot above and below the A-line (Figure 9.84a and 9.84b). Two Atterberg tests from 2009804 cores 0013pc and 0040pc taken on the shelf are of intermediate plasticity (CI) and plot within Zone B liquefaction zone identified by Seed et al., 2008. Two Atterberg tests from the upper 25 cm in 2014804 cores 0016pc and 0024pc are of extreme plasticity (CE and ME), likely due to recent sedimentation. Table 9.6 summarizes all Atterberg test results for Region 2.

The colloidal activity ranges from inactive to normal for all cores within Region 2 except for one clay fraction from core 20148040013 located on the shelf identified as an active clay lying just above the A = 1.25 line. Clay fractions and PI are variable in Region 2 with no distinct pattern (Figure 9.85).

A total of 13 consolidation tests were performed on samples within Region 2 and indicate the sediments are apparently over consolidated in the upper 2 m with OCR values decreasing from 5.7 to 1.8, with exception of a consolidation test at 133 cm from core 2012004PGC0026pc suggesting the sediments are under consolidated with an OCR value of 0.3. The sediments are over consolidated from 2 to 3.5 m with OCR > 1, then under consolidated at 3.8 and 4.3 m with OCR < 1, then over consolidated at 7 m with OCR value of 1.3 (Figure 9.86). Table 9.7 summarizes consolidation test results for Region 2.

A total of 9 CIU triaxial tests were performed on samples within Region 2 and indicate S values range from 0.22 to 0.29. Table 9.8 summarizes the CIU triaxial test results of samples from Region 2.

A comparison of MV data to effective overburden pressure (Figure 9.87a and 9.87b) indicates that all sediments from failed and unfailed material within Region 2 are of high strength with MV data plotting between $S_u/\sigma'v$ values of 0.5 and 1.0.

There is good correlation between cores taken from failed material (Figure 9.87a). Cores 20148040016pc and 20098040019pc have peak shear strengths below 15 kPa effective overburden pressure with $S_u/\sigma'v$ values > 1. Core 20148040016pc has a distinct boundary between 12 and 15 kPa effective overburden pressure indicating the potential for significant overburden removal. The depth of overburden removal using 20148040016pc peak MV shear strength was estimated as 15.0 to 23.7 m.

There is good correlation between cores taken from unfailed material (Figure 9.87b). MV data suggests the sediments are apparently over consolidated in the upper 5 kPa of effective overburden pressure with S_u/σ'_v values < 1. Core 20098040036pc MV data trend correlates with Region 1 core 2013HEALY0001pc, with a reduction in the relative strength of the sediment to $S_u/\sigma'v$ value of 0.3 and would suggest the sediments are normally consolidated below 15 kPa effective overburden pressure.

FS calculated at various slope angles for cores within Region 2 indicate that sediments at all core sites are presently stable under gravitational loading (Figure 9.88a and 9.88b) in the upper 7.2 m with a minimum critical slope angle of 15°. Table 9.9 summarizes the stability analysis under gravitational loading for all core sites within Region 2.

9.4 Region 3 – Eastern

Region 3 is located on the central Kugmallit Fan (Figure 9.89) and consists of smooth seafloor features from the shaft break to a depth of 1500 m where multiple failures are present. The 3.5 kHz sub-bottom CHRIP profiles illustrating the acoustic stratigraphy and position (if possible) for the cores in region 1 are presented in Figure 9.90.

9.4.1 Piston Core 20108040019

Introduction

Piston core 20108040019 was collected from the Pokak Block at a water depth of 80 m and a slope angle of 0.2° (Figure 9.89), determined from multi-beam data. The core recovered 327 cm of sediment. The 3.5 kHz sub-bottom CHRIP profiles illustrating the acoustic stratigraphy and position for core 20108040019 is presented in Figure 9.90.

Atterberg Limits

Atterberg limit tests were completed on 4 samples (Figure 9.91). The samples were of intermediate to very high plasticity. The liquid limits range from 46.1 to 86.1% and the plastic limits range from 24.3 to 30.2%.

Slope Stability

The *FS* was calculated at slope angles of 0.2°, 1°, 5°, 10°, 15°, and 20° (Figure 9.92). The slope is stable under static conditions with a minimum *FS* > 30 at a core depth of 220 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 274 m, 23.1°, and 0.18, respectively.

Geotechnical Profile

The sediment consists of silty clay with some sand. The core is characterized by a linear increase in density from 1.75 g/cm3 at the seafloor to 2.2 g/cm3 at the base of the core. The grain size fractions are uniform in the upper 2 m with 8 to 20% sand, 50% silt, and 30 to 40% clay. Below 2 m, grain size fractions are 60% sand, 32% silt, and 8% clay. Water content decrease sharply from 100 to 60% in the upper 1.5m, then is uniform with depth from 1.5 m to the base of the core. Liquidity indices are uniform at 1 in the upper 1.5 m, then drops to 0.6 below 1.5 m. (Figure 9.93)

MV measurements increase linearly with depth and lie above S values. MV measurements become variable from 1.75 m to the base of the core, and range up to 37 kPa. The increase and variability in MV measurements at the 1.75 m contact could be a result of erosion or ice loading. An erosion calculation was performed using peak MV measurements and estimates the thickness of sediment removed to be between 22.7 (S = 0.2) and 15.2 m (S = 0.3) of sediment. The minimum FS > 30 suggests the core is stable under gravitational loading.

9.4.2 Piston Core 20108040024

Introduction

Piston core 20108040024 was collected from the Pokak Block at a water depth of 182 m and a slope angle of 2.8° (Figure 9.89), determined from multi-beam data. The core recovered 440 cm of sediment. The 3.5 kHz sub-bottom CHRIP profiles illustrating the acoustic stratigraphy and position for core 20108040024 is presented in Figure 9.90.

Atterberg Limits

Atterberg limit tests were completed on 5 samples (Figure 9.94). The samples were of intermediate to high plasticity. The liquid limits range from 43.4 to 69.2% and the plastic limits range from 22.8 to 28.0%.

Consolidation Test Results

Consolidation tests were conducted on 2 samples at a core depths of 70-72.5 cm and 255-257.5 cm (Figure 9.95a and 9.95b) to assess the compressibility (C_c), hydraulic conductivities (k) and stress histories (*OCR*). The compressibility are moderate with compression indices, C_c, of 0.51 and 0.36, and recompression indices, C_r, of 0.082 and 0.057. The hydraulic conductivities, k, at the void ratio equivalent to the P'_c are 5.26E-08 cm/sec and 5.36E-07 cm/sec. The consolidation tests characterize the sediment as over consolidated with respect to the effective overburden with *OCR* values of 5.68 and 3.04.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 261-273 cm (Figure 9.96). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 23.6° and effective cohesion value (c') of 5.9 kPa. The pore pressure coefficient at failure (A_f) is 0.37. The S ratio is 0.34. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.6516 m/s kPa^{-1} and 64.01 m/s (Figure 9.97).

Slope Stability

The *FS* was calculated at slope angles of 1°, 2.8°, 5°, 10°, 15°, and 20° (Figure 9.98). The slope is stable under static conditions with a minimum *FS* of 12.0 at a core depth of 380 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 33 m, N/A°, and 0.26, respectively.

Geotechnical Profile

The sediment consists of clay and silt. Gravitational compaction is the dominant feature in this core with increase in density and decrease in water content with depth. (Figure 9.99)

Grain size fractions are uniform with depth with < 5% sand, 50 to 55% silt, and 40 to 50% clay. Water content decrease gradually from 75 to 40%. The liquidity indices are uniform with depth ranging from 1 to 0.5.

Consolidation tests at 70 and 255 cm suggest the sediments are over consolidated with OCR values of 5.68 and 3.04, respectively. MV measurements increase with depth and are significantly higher than S values. The higher MV data below 2.25 m may over-estimate the sediment's shear strength due to the presence of clasts. A CIU triaxial test at 261 cm consolidated to in-situ effective overburden pressure lies between MV measurements and S values. This may be a result of the influence on MV data from numerous clasts within the sediment. The depth of overburden removal using peak MV measurements was estimated as 10.8 to 17.6 m. The minimum FS of 12 suggests the core is stable under gravitational loading.

9.4.3 Piston Core 20108040036

Introduction

Piston core 20108040036 was collected from the Pokak Block at a water depth of 255 m and a slope angle of 0.9° (Figure 9.89), determined from multi-beam data. The core recovered 725 cm of sediment. The 3.5 kHz sub-bottom CHRIP profiles illustrating the acoustic stratigraphy and position for core 20108040036 is presented in Figure 9.90.

Atterberg Limits

Atterberg limit tests were completed on 7 samples (Figure 9.100). The samples were of high plasticity. The liquid limits range from 61.5 to 68.6% and the plastic limits range from 29.3 to 38.1%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 389-391.5 cm (Figure 9.101) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate to high with compression index, C_c, of 0.67 and recompression index, C_r, of 0.079. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 6.79E-08 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value of 1.35.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 698-710 cm (Figure 9.102). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 22.7° and effective cohesion value (c') of 2.5 kPa. The pore pressure coefficient at failure (A_f) is 0.70.

The S ratio is 0.26. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.4446 m/s kPa^{-1} and 76.95 m/s (Figure 9.103).

Slope Stability

The *FS* was calculated at slope angles of 0.9°, 1°, 5°, 10°, 15°, and 20° (Figure 9.104). The slope is stable under static conditions with a minimum *FS* of 3.5 at a core depth of 430 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 30 m, 3.2°, and 0.02, respectively.

Geotechnical Profile

The sediment consists of silty clay. The uniform density with depth is the dominant feature in this core with decrease in water content. (Figure 9.105)

The grain size fractions are uniform with < 5% sand, 40 to 45% silt, and 50 to 60% clay. Water contents decrease linearly with depth from 100 to 75% in the upper 1 m, then are uniform with depth. The liquidity indices are uniform with depth, ranging from 1.5 to 1.2.

A consolidation test at 389 cm suggests the sediments are normally consolidated with an OCR value of 1.35. MV measurements increase uniformly with depth plotting within the S value range suggesting the sediment is normally consolidated, with exception at 4.25 m where MV measurements lie below S values. A CIU triaxial test at 698 cm consolidated to in-situ effective overburden pressure correlates with the upper limit of the S value range. The minimum FS of 3.5 suggests the core is stable under gravitational loading.

9.4.4 Piston Core 20108040056

Introduction

Piston core 20108040056 was collected from failed material in the Pokak Block at a water depth of 994 m and a slope angle of 2.1° (Figure 9.89), determined from multi-beam data. The core recovered 619 cm of sediment. The 3.5 kHz sub-bottom CHRIP profiles illustrating the acoustic stratigraphy and position for core 20108040056 is presented in Figure 9.90.

Atterberg Limits

Atterberg limit tests were completed on 6 samples (Figure 9.106). The samples were of intermediate to high plasticity. The liquid limits range from 45.4 to 68.4% and the plastic limits range from 27.7 to 45.4%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 427-429.5 cm (Figure 9.107) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate to high with compression index, C_c, of 0.57 and recompression index, C_r, of 0.076. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 1.03E-07 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value of 1.58.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 409-421 cm (Figure 9.108). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 19.3° and effective cohesion value (c') of 2.3 kPa. The pore pressure coefficient at failure (A_f) is 0.65.

The S ratio is 0.22. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.4088 m/s kPa^{-1} and 49.02 m/s (Figure 9.109).

Slope Stability

The *FS* was calculated at slope angles of 1°, 2.1°, 5°, 10°, 15°, and 20° (Figure 9.110). The slope is stable under static conditions with a minimum *FS* of 9.7 at a core depth of 270 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 23 m, 22.9°, and 0.09, respectively.

Geotechnical Profile

The sediment consists of silty clay. Uniform density is the dominant feature in this core with decrease in water content with depth. (Figure 9.111)

The grain size fractions are uniform with depth with minimal sand, 35 to 45% silt, and 55 to 65% clay. Water contents decrease abruptly from 125 to 55% in the upper 2 m, then decreases linearly from 2 m to the base of the core. The liquidity indices are uniform ranging from 1 to 1.5, except at the base where the liquidity index jumps to 2.6.

A consolidation test at 427 cm suggests the sediments are slightly over consolidated with an OCR value of 1.58. MV measurements increase uniformly with depth and lie above S values. A CIU triaxial test at 409 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS of 9.7 suggests the core is stable under gravitational loading.

9.4.5 Piston Core 20108040069

Introduction

Piston core 20108040069 was collected from the Pokak Block at a water depth of 631 m and a slope angle of 2.0° (Figure 9.89), determined from multi-beam data. The core recovered 441 cm of sediment. The 3.5 kHz sub-bottom CHRIP profiles illustrating the acoustic stratigraphy and position for core 20108040069 is presented in Figure 9.90.

Atterberg Limits

Atterberg limit tests were completed on 4 samples (Figure 9.112). The samples were of high to very high plasticity. The liquid limits range from 54.2 to 75.2% and the plastic limits range from 27.0 to 33.6%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 410-412.5 cm (Figure 9.113) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.37 and recompression index, C_r, of 0.052. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 1.77E-06 cm/sec. The consolidation test characterizes the sediment as normally consolidated with respect to the effective overburden with OCR value of 1.13.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 392-400 cm (Figure 9.114). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 25.9°

and effective cohesion value (*c′*) of 0.0 kPa. The pore pressure coefficient at failure (A_f) is 0.87. The S ratio is 0.27.

Slope Stability

The *FS* was calculated at slope angles of 1°, 2°, 5°, 10°, 15°, and 20° (Figure 9.115). The slope is stable under static conditions with a minimum *FS* of 5.5 at a core depth of 430 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 34 m, 11.5°, and 0.07, respectively.

Geotechnical Profile

The sediment consists of silty clay to clay and silt. Gravitational compaction is the dominant feature in this core with increase in density and decrease in water content with depth. (Figure 9.116)

The grain size fractions are uniform with depth with minimal sand, 35 to 50% silt, 50 to 65% clay. Water contents decrease linearly from 125 to 60%. The liquidity indices are uniform with depth ranging from 1.1 to 1.2, except at 1 m where the liquidity index is 1.8.

A consolidation test at 410 cm suggests the sediments are normally consolidated with an OCR value of 1.13. MV measurements increase with depth and fall within S values below 3.25 m. A CIU triaxial test at 392 cm consolidated to in-situ effective overburden pressure correlates with MV measurements and S values. The minimum FS of 5.5 suggests the core is stable under gravitational loading.

9.4.6 Piston Core 20108040070

Introduction

Piston core 20108040070 was collected from the Pokak Block at a water depth of 879 m and a slope angle of 0.7° (Figure 9.89), determined from multi-beam data. The core recovered 420 cm of sediment. The 3.5 kHz sub-bottom CHRIP profiles illustrating the acoustic stratigraphy and position for core 20108040069 is presented in Figure 9.90.

Atterberg Limits

Atterberg limit tests were completed on 4 samples (Figure 9.117). The samples were of high plasticity. The liquid limits range from 57.8 to 64.5% and the plastic limits range from 27.3 to 31.5%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 220-222.5 cm (Figure 9.118) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.55 and recompression index, C_r, of 0.049. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 5.55E-08 cm/sec. The consolidation test characterizes the sediment as over consolidated with respect to the effective overburden with OCR value 2.00.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 225-233 cm (Figure 9.119). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 23.5°

and effective cohesion value (c') of 0.0 kPa. The pore pressure coefficient at failure (A_f) is 0.67. The S ratio is 0.27.

Slope Stability

The *FS* was calculated at slope angles of 0.7°, 1°, 5°, 10°, 15°, and 20° (Figure 9.120). The slope is stable under static conditions with a minimum *FS* of 20.5 at a core depth of 370 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 105 m, 15.0°, and 0.08, respectively.

Geotechnical Profile

The sediment consists of silty clay. Gravitational compaction is the dominant feature in this core with increase in density, interrupted by variable density from 200 cm to the base of the core. (Figure 9.121)

The grain size fractions are uniform with minimal sand, 35% silt, 65% clay. Water contents decrease linearly from 125 to 75% in the upper 2 m, then are uniform to the base of the core. The liquidity indices decrease linearly from 1.7 to 1.3.

A consolidation test at 220 cm suggests the sediments are over consolidated with an OCR value of 2.00. MV measurements increase with depth and lie within S values below 3.25 m. A CIU triaxial test at 225 cm consolidated to in-situ effective overburden pressure correlates with MV measurements. The minimum FS > 20 suggests the core is stable under gravitational loading.

9.4.7 Summary

The sediments in Region 3 predominantly range from intermediate to high plasticity and plot above and along the A-line (Figure 9.122). Two Atterberg tests from 2010804 cores 0019pc and 0069pc are of very high plasticity. Table 9.10 summarizes all Atterberg test results for Region 3.

The colloidal activity in Region 3 is predominantly inactive with the exception of core 20108040019 located on the shelf with activity ranging from normal to active with variable *PI* and clay fractions ranging from 20 to 60% and 30 to 40%, respectively. Core 20108040019 generally follows an A = 0.75 linear trend. The rest of the cores off the shelf are inactive with variable clay fractions ranging from 40 to 70% and variable *PI* ranging from 20 to 45% (Figure 9.123). Most cores off the shelf have a more consistent activity with some variability following an A = 0.56 general linear trend.

A total of 6 consolidation tests were performed on samples within Region 3 and indicate the sediments are apparently over consolidated at 0.7 m with OCR value of 5.7. The sediments are over consolidated from 2 to 4.3 m with OCR values ranging from 1.1 to 3.0 (Figure 9.124). A consolidation test at 255 cm from core 2010804 0024pc with OCR value of 3.0 could correlate with a regional unconformity. Table 9.11 summarizes consolidation test results for Region 3.

A total of 5 CIU triaxial tests were performed on samples within Region 3 and indicate S values range from 0.22 to 0.34. Table 9.12 summarizes the CIU triaxial test results of samples from Region 3.

A comparison of MV data to effective overburden pressure (Figure 9.125) and indicates that sediments are predominantly of high strength with S_u/σ'_v values decreasing from >1 to 0.3.

Below 15 kPa effective overburden pressure, sediments from cores 2010804 0036pc, 0069pc, and 0070pc are of normal strength gradually become weaker with increasing effective overburden pressure, with exception of two low strength MV measurements in 0036pc at 28 kPa effective overburden pressure and 1 high strength MV measurement at 40 kPa effective overburden pressure. Sediments of low strength are possibly under consolidated and have the potential for excess pore pressure; however, based on visual core descriptions and core photography core 0036pc has no apparent weak layer. Sediments from core 2010804 0024pc are of abnormally high strength with S_u/σ'_v values > 1 for most of the core, with MV measurements increasing up to 275 cm, then decreasing to the base of the core. High strengths could be associated with glacial till IRD and sand lenses and may represent a regional unconformity.

FS calculated at various slope angles for cores within Region 3 indicate that sediments at all core sites are presently stable under gravitational loading (Figure 9.126) in the upper 7.25 m with a minimum critical slope angle of 12°, with exception of a minimum critical angle of 3° at core site 20108040036pc. Table 9.13 summarizes the stability analysis under gravitational loading for all core sites within Region 3.

9.5 Region 4 – Banks Island

Region 3 is located to the west of Banks Island. central Kugmallit Fan (Figure 9.127) and consists of smooth seafloor features The 3.5 kHz sub-bottom profiles illustrating the acoustic stratigraphy and position for the cores in region 4 are presented in Figure 9.128.

9.5.1 Piston Core 20148040006

Introduction

Piston core 20148040006 was collected from the shelf west of Banks Island in a water depth of 122 m and a slope angle of 1.5° (Figure 9.127), determined from multi-beam data. The core recovered 395 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and position for core 20148040011pc is presented in Figure 9.128a.

Atterberg Limits

Atterberg limit tests were completed on 5 samples (Figure 9.129). The samples were of intermediate to high plasticity. The liquid limits range from 41.5 to 57.9% and the plastic limits range from 20.2 to 25.2%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 357-359.5 cm (Figure 9.130) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is low with compression index, C_c, of 0.19 and recompression index, C_r, of 0.055. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 2.17E-08 cm/sec. The consolidation test characterizes the sediment as normally consolidated with respect to the effective overburden with OCR value of 0.97.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 362-374 cm (Figure 9.131). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 20.36° and effective cohesion value (c') of 8.0 kPa. The pore pressure coefficient at failure (A_f) is

0.42. The S ratio is 0.30. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.0539 m/s kPa^{-1} and 108.54 m/s (Figure 9.132).

Slope Stability

The *FS* was calculated at slope angles of 1°, 1.5°, 5°, 10°, 15°, and 20° (Figure 9.133). The slope is stable under static conditions with a minimum *FS* of 17.1 at a core depth of 160 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 40 m, 32.2°, and 0.18, respectively.

Geotechnical Profile

The sediment consists of silty clay to silt and clay. Gravitational compaction is the dominant feature in this core with increase in density and decrease in water content, with a jump in density below 175 cm. (Figure 9.134)

Grain size fractions are uniform with depth with < 15% sand, 50 to 55% silt, and 30 to 40% clay. Water contents decrease linearly from 90 to 40% in the upper 1.5 m, then are uniform to the base of the core. The liquidity indices decrease from 1.5 to 0.5 in the upper 1.5 m then are uniform to the base of the core.

A consolidation test at 357 cm suggests the sediments are normally consolidated with an OCR value of 0.97. MV measurements increase uniformly with depth in the upper 2 m, then increase variably with depth from 8 to 30 kPa. These high MV measurements correlate with the diamict unit. MV measurements lie above S values. A CIU triaxial test at 362 cm consolidated to in-situ effective overburden pressure appears to correlate with MV measurements. The minimum FS of 17.1 suggests the core is stable under gravitational loading.

9.5.2 Piston Core 20148040011

Introduction

Piston core 20148040011 was collected from the outer shelf west of Banks Island in a water depth of 413 m and a slope angle of 0.4° (Figure 9.127), determined from multi-beam data. The core recovered 515.5 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and position for core 20148040011pc is presented in Figure 9.128b.

Atterberg Limits

Atterberg limit tests was completed on 6 samples (Figure 9.135). The samples were of intermediate to high plasticity. The liquid limits range from 37.9 to 61.8% and the plastic limits range from 19.4 to 27.6%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 304-306.5 cm (Figure 9.136) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is moderate with compression index, C_c, of 0.63 and recompression index, C_r, of 0.100. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 4.86E-08 cm/sec. The consolidation test characterizes the sediment as normally consolidated with respect to the effective overburden with OCR value of 0.81.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 308-316 cm (Figure 9.137). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 28.52° and effective cohesion value (c') of 0.0 kPa. The pore pressure coefficient at failure (A_f) is 0.61. The S ratio is 0.38.

Slope Stability

The *FS* was calculated at slope angles of 0.4°, 1°, 5°, 10°, 15°, and 20° (Figure 9.138). The slope is stable under static conditions with a minimum *FS* of 34.6 at a core depth of 290 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 128 m, 14.4°, and 0.08, respectively.

Geotechnical Profile

The sediments consists of clayey silt with sand. Variable density is the dominant feature in this core with uniform water content with depth. (Figure 9.139)

Grain size fractions are variable ranging from 10 to 20% sand, 40 to 50% silt, and 30 to 45% clay. Water contents are uniform at 50 to 60% in the upper 3.5 m. Below 3.5 m, water contents decrease to 25% correlating with the diamict unit. The liquid and plastic limits are variable with depth ranging from 38 to 62 and 19 to 34 respectfully. The liquidity indices decrease from 1.6 to 1.1.

A consolidation test at 304 cm suggests the sediments are slightly under consolidated with an OCR value of 0.81. MV measurements are variable with depth ranging between 4 and 18 kPa and lie above S values in the upper 2.5 m. Below 2.5 m, MV measurements lie between S values, except for a MV measurement in the diamicton unit at 4.5 m which lies above S values. A CIU triaxial test at 308 cm consolidated to in-situ effective overburden pressure lies above MV measurements and S values. The minimum FS > 30 suggests the core is stable under gravitational loading.

9.5.1 Piston Core 20168050013

Introduction

Piston core 20168050013 was collected from the outer shelf west of Banks Island in a water depth of 44 m and a slope angle of 0.4° (Figure 9.127). The core recovered 152.0 cm of sediment. The 3.5 kHz sub-bottom profile illustrating the acoustic stratigraphy and position for core 20148040011pc is presented in Figure 9.128c.

Atterberg Limits

Atterberg limit tests was completed on 1 sample (Figure 9.140). The sample was of intermediate plasticity. The liquid limit was 37.3% and a plastic limit of 20.4%.

Consolidation Test Results

A consolidation test was conducted on 1 sample at a core depth of 114-116.5 cm (Figure 9.141) to assess the compressibility (C_c), hydraulic conductivity (k) and stress history (OCR). The compressibility is low with a compression index, C_c, of 0.19 and recompression index, C_r, of 0.051. The hydraulic conductivity, k, at the void ratio equivalent to the P'_c is 3.92E-08 cm/sec. The consolidation test characterizes the sediment as over-consolidated with respect to the effective overburden with an OCR value of 7.85.

Triaxial Test Results

An isotropic CIU triaxial test was conducted on 1 sample at a core depth of 120-132 cm (Figure 9.142). The Mohr-Coulomb failure envelopes are defined by effective friction angle (ϕ') of 25.0° and effective cohesion value (c') of 20.43 kPa. The pore pressure coefficient at failure (A_f) is 0.19. The S ratio is 0.38. Bender element shear wave velocities (V_s) versus confining pressure (σ'_c) were defined by a slope and intercept of 0.50 m/s kPa^{-1} and 110.2 m/s (Figure 9.143).

Slope Stability

The *FS* was calculated at slope angles of 0.4°, 1°, 5°, 10°, 15°, and 20° (Figure 9.144). The slope is stable under static conditions with a minimum *FS* of 131.2 at a core depth of 80 cm. The critical height, critical slope angle, and minimum acceleration coefficient are 186 m, >45°, and 2.49, respectively.

Geotechnical Profile

The sediments consists of clayey silt with sand. Variable density is the dominant feature in this core with uniform water content with depth. (Figure 9.145)

One sample at 115 cm has a water content of 27.7% a liquid limit of 37.3%, a plastic limit o 20.4% and a liquidity index of 0.43. A consolidation test at 115 cm suggests the sediments are over-consolidated with an OCR value of 7.85. MV values are variable in the upper 90 cm ranging between 16 and 34 kPa. There is a marked increase in shear strength at 1 meter with values ranging between 70 and 85 kPa for the remainder of the core. The MV values plot well above the S values indicating the sediments are highly over-consolidated. The minimum FS > 30 suggests the core is stable under gravitational loading.

9.5.2 Summary

The sediments in Region 4 range from intermediate to high plasticity and plot above the A-line (Figure 9.146). Three Atterberg tests from core 20148040011pc and one core from 20168050013pc plot within Zone B identified by Seed et al., 2008. Table 9.14 summarizes all Atterberg test results for Region 3.

The colloidal activity in Region 4 ranges from inactive to normal clays with concentrated clay fractions between 30 and 45% and variable *PI* ranging from 15 to 35%. Core 20148040011 consists only of inactive clays and follows an $A = 0.58$ linear trend, whereas core 20148040006 ranges from inactive to normal clays. (Figure 9.147).

Three consolidation tests were performed on samples within Region 4. Two sample from cores 2014804 006 and 0011 are normally consolidated to slightly under consolidated below 3 m with OCR values of < 1 (Figure 9.148). A sample from 201680500013 sampled a denser layer resulting in an OCR value of 7.9. Table 9.15 summarizes consolidation test results for Region 4.

Two CIU triaxial tests were performed on samples within Region 4 and indicate S values range from 0.30 to 0.38. Table 9.16 summarizes the CIU triaxial test results of samples from Region 4.

A comparison of MV data to effective overburden pressure (Figure 9.149) indicates that sediments in Region 4 are of high strength in the upper 20 kPa effective overburden pressure with $S_u/\sigma'v$ *values* > 0.5. Below 20 kPa, core 20148040006pc MV measurements increase to $S_u/\sigma'v$ *values* > 0.8, whereas core 20148040011pc MV measurements decrease to $S_u/\sigma'v$ *values* between 0.25 and 0.4 indicating sediments jump between normal and high strength.

FS calculated at various slope angles for cores within Region 4 indicate that sediments at all core sites are presently stable under gravitational loading (Figure 9.150) in the upper 5.15 m with a minimum critical angle of 14°. Table 9.17 summarizes the stability analysis under gravitational loading for all core sites within Region 4.

10.0 EVALUATING TRIGGER MECHANISMS

10.1 Introduction

The infinite slope method was used for the slope stability analysis. It assumes the failure surface is parallel to the slope, the slope is planar and of infinite length. The failure length of the slope is significantly greater than the failure thickness. The analysis was done for undrained conditions. The triggering mechanisms analysed were gravitational loading, earthquake loading, and excess pore pressure.

10.2 Gravitational Loading

In Section 9 of this report, each core with discrete MV data underwent an undrained slope stability analysis using the infinite slope method. It is difficult however to perform a comparison of the slope stability results between cores of different lengths using MV data due to the generally high Su/σ'_v in the upper 3 meters. Hence the SHANSEP method was also used to estimate *FS* behaviour for normally consolidated sediment. Triaxial data estimated the *S* values to range between 0.2 and 0.3.

The *FS* derived from Mohr-Coulomb ϕ' and the SHANSEP method are compared to the FS valves obtained using the MV data from 013HEALY0001pc and 20108040036pc) in Figure 10.1. All regions correlate with each other and estimate higher *FS* calculated from Mohr-Coulomb than the SHANSEP method. Mohr-Coulomb results suggest a critical angle of 20° in order to fail the slope, which approximates the effective friction angle. The SHANSEP method suggests sediment becomes unstable with slopes > 13°. In comparison the cores 2013HEALY0001pc and 20108040036pc, have critical slope angles of 5.4° and 3.2°. The low critical slope angles are likely attributed to the low MV measurements in the core (Figures 9.25 and 9.125).

10.3 Earthquake Seismic Loading

Along the Beaufort Sea slope, earthquakes most commonly range from magnitudes 2 to 5 based on historical earthquake events (Figure 10.2) with one large *M*5.6 event. Region 3 lies within the seismically active area and Region 2 is on the border of the zone. Regions 1 and most of Region 4 are removed from the active zone.

S values and shear wave velocities were determined from CIU triaxial and bender element test data. MST bulk density data were used to calculate weight ratios (γ'/γ) for all cores. Morgenstern, 1967 approach (Equation 8.15) was used to calculate the critical horizontal acceleration coefficient (k_{min}). Boore et al. (1997) attenuation relationships enabled the investigation of relating earthquake magnitude and distance to the critical horizontal acceleration coefficient.

The attenuation relationships were used to investigate how earthquake magnitude and distance relate to slope angles on a regional basis. The V_{s30}, k_{min}, S values, and weight ratios (γ'/γ) for each core are listed in Table 10.1. Regional V_{s30}, S values, and weight ratios (γ'/γ) used in the analysis are listed in Table 10.2. Critical horizontal acceleration coefficients were calculated for each region under different slope angles using the minimum, maximum and average values of V_{s30}, γ'/γ, and S parameters. Under set magnitudes, varying slope angles, and subsequently varying k_{min}, were related to distance from the epicenter using Boore et al. (1997). The results of the analysis are presented in Tables 10.3, 10.4 and 10.5 and Figures 10.3, 10.4, 10.5 and 10.6.

Region 1 is located south-west from the seismically active zone. No historical events were recorded within Region 1. The largest event, a *M*5.6 earthquake, is located approximately 90 km northeast of the certain portion of Region 1. This event likely did not trigger slope failures in Region 1 as steep slope angles approaching 8° are required using average V_{s30}, γ'/γ, and S values (Figures 10.3a). The slope angle for failure is reduced to 5.0° when using the minimum values. (Figure 10.3c) The epicenter must be even closer at smaller slope angles. Local *M*4 events within 20 km of 2° slopes could trigger slope failures in this region (Figure 10.3c).

Region 2 is located on the edge of the seismically active zone. Historical events within Region 2 range from *M*2 to *M*4. The *M*5.6 event is located 90 km north-northwest and requires a slope angle of 7.8° to trigger slope failures within Region 2 based on average V_{s30}, γ'/γ, and S values (Figure 10.4a). The slope angle for failure is reduced to 3.5° when using the minimum values. (Figure 10.4c). A local *M4* event within 25 km of a 2° slope in this region could trigger slope failures (Figure 10.4c).

Region 3 is situated within the seismically active zone with historical events ranging from *M*1 to *M*5. Using the minimum V_{s30}, γ'/γ, and *S* values, local *M*4 events must be within 20 km to trigger failures on 2° slopes while a larger event of *M*5.6 must be within 57 km on 2° slopes to cause landslides (Figure 10.5c). The average and maximum data are presented in Figures 10.5a and 10.5b.

Region 4 is located to the east of the seismically active zone. Using average V_{s30}, γ'/γ, and *S* values local *M*4 events must be within 4 km for 2° slopes to fail. A larger event of *M*5.6 must be within 25 km of steeper 6° slopes to trigger failure (Figures 10.6a). Using minimum V_{s30}, γ'/γ, and *S* values, local *M*4 events must be within 8 km for 2° slopes to fail, while a larger event of *M*5.6 must be within 30 km on 2° slopes to cause landslides (Figure 10.6c)

The two cores, 2013HEALY0001 from Region 1, and 20108040036 from Region 3, with the lowest minimum *FS* of the cores analysed were selected for further analysis. The low minimum *FS* are attributed to low discrete MV measurements resulting in low S_u/P'_o values of 0.16 for core 2013 HEALY0001 and 0.10 for core 20108040036. The geotechnical data used in the analysis were obtained from the 2 cores. The distance required from the epicenter to result in a FS of one was related to earthquake magnitude and slope angle.

The analysis for core 2013 HEALY0001 includes slope angles to 6° (Figure 10.7). Note that the critical slope angle (*FS* = 1) for gravitational loading for core 2013 HEALY0001 is 5.4°. The distance to the epicenter ranges between 20 to 80 km for slope angles between 1 and 6° for a magnitude 4.0 earthquake. The range is 60 to 215 km for a 5.5 magnitude earthquake. The analysis for core 20108040036 includes slope angles to 4° (Figure 10.8). Note that the critical slope angle (*FS* = 1) for gravitational loading for core 2013 HEALY0001 is 3.2°. The distance to the epicenter ranges between 30 to 110 km for slope angles between 1 and 4° for a magnitude 4.0 earthquake. The range is 90 to 320 km for a 5.5 magnitude earthquake.

10.4 Excess Pore Pressure

The minimum excess pore pressure required for slope failure was calculated using the infinite slope method with Mohr-Coulomb strength parameters (See Equation 8.13). The minimum excess pore pressure required for failure (i.e. when *FS* = 1) was calculated at the minimum *FS* depth. A summary of these results is shown in Table 10.6. Note that only cores with triaxial testing measured Mohr-Coulomb strength parameters are included in these results.

11.0 SUMMARY

A comprehensive geotechnical laboratory testing program was conducted on 23 piston cores and 1 gravity core collected between 2009 and 2016 in water depths from 70 to 1536 m in the Canadian Beaufort Sea. The geotechnical data presented included MSCL bulk density, Atterberg limits, grain size analysis, back-pressured consolidation tests and isotropically consolidated undrained (CIU) triaxial tests. Cores targeted unfailed sediments, deeply failed scar floors, previously failed deposits and fluid or permafrost-affected glacial and post-glacial sediments.

The study area has been divided into 4 regions: Region 1 – Western; Region 2 – Central; Region 3 – Eastern; and, Region 4 – Banks Island (Figure 9.1). The region selection is arbritray and partly reflects the geographic focus of the hydrocarbon industry and the quality and quantity of data collected.

Atterberg limits across all regions range from intermediate to very high plasticity and lie along the A-line. Regions 1, 2 and 3 correlate with each other covering the entire plasticity range described previously. Region 4 is the only region which ranges from intermediate to high plasticity (Figure 11.1). Most of the samples plot outside Seed et al. (2003) zones A and B, indicating limited potential for soil liquefaction. There are 5 samples (Regions 2 and 4) that plot within Seed et al. (2003) zone B, 4 of which are located on the shelf. The 2 samples from Region 2 were taken in cores (20098040013pc and 0040pc) in water depths of < 75 m. In Region 4 one sample from core 20168050013pc was collected in a water depth of 44 m while 2 samples are from core 20148040011pc which was collected in a water depth of 411 m.

The activity for each region (Figure 11.2) overlap with each other. Region 1 activity has a wide range with variable clay fractions and *PI*, however, as noted in Section 9.2.4, each core in the region has uniform clay fractions which are independent of each other. Region 2 also has a large activity range but is predominantly characterized by high *PI* and high clay fractions ranging from inactive to normal clays. Region 3 is characterized by variable clay fractions which are predominantly inactive. Lastly, Region 4 is characterized by low clay fractions and *PI*. Regions 3 and 4 follow a linear activity trend, whereas Regions 1 and 2 show no distinct linear activity trend.

A total of 27 consolidation tests were conduced on samples from 21 cores. The compressibility of the sediments (C_c) is generally typical of marine sediments and range from 0.19 to .89 (Table

11.1). The lower C_c values are from cores collected in Region 4. The OCR values indicate the sediments are generally overconsolidated in the upper 2 to 3 meters and become normally consolidated with depth. (Figure 11.3). It should be noted that 5 consolidation tests suggest the sediments are under consolidated (Table 11.2) at those sites. The 3 underconsolidated samples in Region 2 are associated with the prominent failure featured in that region. One sample was obtained within the failure while 2 samples were from near the edge of the head scape (see Figure 9.27) which shows possible disturbance on the multibeam data. The underconsolidated sample in Region 1 from 2013 HEALY0001pc core (see Figure 9.2) is near the base of the core. Of note the MV shear strength in this core suggests the sediments are underconsolidated below 750 cm (see Figure 9.15). The 5th underconsolidated sample is from Region 4, core 20148040011pc (see Figure 9.127) and corresponds to a drop in MV values. The sediments appears to be undisturbed and are underlain by a glacial diamicton.

A total of 22 CIU triaxial tests were conducted on samples from 19 cores. The effective friction angle values range between 15.9° to 28.5° and the effective cohesion values range between 0.0 kPa and 20.4 kPa with the higher values occurring in Region 4 (Table 11.1). The S values range from 0.2 to 0.3 in regions 1, 2, and 3 with values ranging from 0.3 to 0.4 for region 4.

Slope stability analysis for each core suggests that sediments in the upper 8 m are stable under gravitational loading for all regions. The 2 cores with the lowest factor of safety 2013 HEALY0001pc Region 1 and 20108040036pc Region 3 with the minimum FS occulting at 12.5 m and 4.3 m respectfully.

The 2013 HEALY0001pc site has a critical slope angle of 5.4° (see Figure 10.1). Under earthquake loading distance to the epicenter from the core site ranges between 20 to 80 km for slope angles between 1 and 6° for a magnitude 4.0 earthquake. The range is 60 to 215 km for a 5.5 magnitude earthquake. The 20108040036pc site has a critical slope angle of 3.2°. Under earthquake loading distance to the epicenter from the core site ranges between 30 to 110 km for slope angles between 1 and 4° for a magnitude 4.0 earthquake. The range is 90 to 320 km for a 5.5 magnitude earthquake.

12.0 REFERENCES

Abrahamson, N.A., Shedlock, K.M. (1997). Overview: Attenuation relationships using various models. Seismological Research Letters, 68-1, 9-23.

Adams, J., Halchuk, S. (2004). Fourth-Generation Seismic Hazard Maps for the 2005 National Building Code of Canada. 13th World Conference on Earthquake Engineering, Paper No. 2502.

ASTM Standard D2216. (2010). Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass. ASTM International, West Conshohocken, PA, 2013, DOI: 10.1520/D2216-10.

ASTM D2435/D2435M. (2011). One-Dimensional Consolidation Properties of Soils. Danvers: ASTM International, West Conshohocken, PA, 2013, DOI: 10.1520/D2435_D2435M-11

ASTM Standard D2487. (2011). Classification of Soils for Engineering Purposes (Unified Soil Classification System). ASTM International, West Conshohocken, PA, 2013, DOI: 10.1520/D2487-11.

ASTM Standard D4318 (2010). Liquid Limit, Plastic Limit, and Plasticity Index of Soils. ASTM International, West Conshohocken, PA, 2013, DOI: 10.1520/D4318-10.

ASTM Standard D4648/D4648M, (2010). Laboratory Miniature Vane Shear Test for Saturated Fine-Grained Clayey Soil. ASTM International, West Conshohocken, PA, 2013, DOI: 10.1520/D4648-10.

ASTM Standard 4767 (2011). Consolidated Undrained Triaxial Compression Test for Cohesive Soils. ASTM International, West Conshohocken, PA, 2013, DOI: 10.1520/D4767-11.

Atterberg, A. (1911). The Behaviour of Clays with Water, their Limits of Plasticity and their Degress of Plasticity. Kunglia Lantbruksakademiens Handlingar och Tidskrift, 132-158.

Batchelor, C.L., Dowdeswell, J.A. and Pietras, J.T., 2013. Variable history of Quaternary ice-sheet advance across the Beaufort Sea margin, Arctic Ocean. Geology, 41(2), pp.131-134.

Batchelor, C.L., Dowdeswell, J.A. and Pietras, J.T., 2014. Evidence for multiple Quaternary ice advances and fan development from the Amundsen Gulf cross-shelf trough and slope, Canadian Beaufort Sea margin. Marine and Petroleum Geology, 52, pp.125-143.

Bishop, A.W. and Henkel, 1962. The measurement of soil propteries in the triaxial test. William Clowes and Sons, Great Britian, 227p.

Boore, D.M, Joyner, W.B., Fumal, T.E. (1997). Equations for Estimating Horizontal Response Spectra and Peak Acceleration from Western North American Earthquakes: A Summary of Recent Work. Seismological Research Letters, 68-1, 128-153.

Camacho-Tauta, J.F., Alvarez, J.D.J., Reyes-Ortiz, O.J. (2012). A procedure to calibrate and perform the bender element test. Dyna 176

Cameron, G.D.M., King, E.L. (2018) Multiple mass failure events in slide valley complexes on the central Beaufort Slope. Geological Survey of Canada Open File 8356. https://geoscan.nrcan.gc.ca/starweb/geoscan/servlet.starweb?path=geoscan/downloade.web&search1=R=314644

Casagrande, A. (1936). "The Determination of Pre-Consolidation Load and Its Practical Significance," Discussion D-34, Proceedings of the First International Conference on Soil Mechanics and Foundation Engineering, Cambridge, Vol. III, pp. 60-64.

Casagrande, A. (1932). Research on the Atterebrg Limits of Soils. *Public Roads*, 121-136.

Casagrande, A. (1948). Classification and Identification of Soils. Transactions, *ASCE*, 901-930.

Dimakis, P., Elverhoi, A., Solheim, A., Harbitz, C., Laberg, J., Vorren, T.O., Marr, J (2000). Submarine slope stability on high-latitude glaciated Svalbard-Barents Sea margin. Marine Geology, 162, 303-316.

Gibson, R.E. (1958). The Progress of Consolidation in a Clay Layer Increasing in Thickness with Time. Geotechnique, 8-4, 171-182.

Gourvenec, S. and White, D (2010). Gulf of Guinea Deepwater Sediments: Geotechnical Properties, Design Issues and Installation Experiences. Frontiers in Offshore Geotechnics II, CRC Press, Boca Raton, 59-86.

Grozic, J.L.H., Lunne, T. and Pande, S. (2003). An oedometer test study on the preconsolidation stress of glaciomarine clays. Canadian Geotechnical Journal, 40, 857-872.

Hance, J.J. (2003) Development of a database and assessment of seafloor slope stability based on published literature, MS Thesis, University of Texas at Austin. USA.

Hansen, E.L., Hemmen, H., Fonseca, D.M., Coutant, C., Knudsen, K.D., Plivelic, T.S., Bonn, D., Fossum, J.O. (2012). Swelling transition of a clay induced by heating. Scientific Reports. 2-618.

Head, K. H. (1992). Manual of Soil Laboratory Testing: Volume 1 Soil Classification and Compactions Tests (Second Edition). In Manual of Soil Laboratory Testing: Volume 1 Soil Classification and Compactions Tests, Pentech Press London, 388p.

Holtz, R.D. and Kovacs, W.D. (1981). An Introduction to Geotechnical Engineering. Prentice-Hall, Inc., Englewood Cliffs, N.J., 733 p.

Ladd, C., DeGoot, D. (2003). Recommended practice for soft ground site characterization: Arthur Casagrande lecture, 12th Panamerican Conference on soil mechanics and geotechnical engineering, Massachusetts Institute of Technology Cambridge, June 22-25,2003, 60 pages.

Ladd, C. and Foot, R. (1974). New Design Procedure for Stability of Soft Clays, Journal of the Geotechnical Engineering Division, ASCE, vol. 100, No. GT7, pp. 763-786.

Lakeman, T.R., Pieńkowski, A.J., Nixon, F.C., Furze, M.F., Blasco, S., Andrews, J.T. and King, E.L., 2018. Collapse of a marine-based ice stream during the early Younger Dryas chronozone, western Canadian Arctic. Geology, 46(3), pp.211-214.

Lambe, W., Whitman, R. (1969). Soil Mechanics. John Wiley & Sons, New York, 552p.

Merzouk, A and Levesque, K., editors. 2009 ArcticNet Amundsen 2009 Expedition Report. www.arcticnet.ulaval.ca › docs › 2009_Amundsen_Expedition_Report

Morgenstern, N.R. (1967). Submarine slumping and the initiation of turbidity currents. Marine Geotechnique, 189-220.

Poulos, H. (1988). Marine Geotechnics. Unwin Hyman Ltd., London, 473p.

Sainte-Ange, F, Kuus, P, Blasco, S, Piper, D J W, Hughes-Clarke, J E, and MacKillop, K. 2014. Multiple failure styles related to shallow gas and fluid venting, upper slope Canadian Beaufort Sea, northern Canada. Marine Geology, vol. 355, 2014 p. 136-149.

Seed, R., Cetin, K. Moss, R. Kammerer, A., Wu, J., Pestana, J., Riemer, M., Sanciol, R., Bray, J., Kayen, R.,and Faris, A. (2003). Recent advances in soil liquefaction engineering. 26th Annual ASCE Los Angeles Geotechnical Spring Seminar, Long Beach, California.

Skempton, A.W. (1953). The colloidal activity of clays. Proceedings of the 3rd International Conference on Soil Mechanics and Foundation Engineering, Zurich, 1, 57-61.

Skempton, A.W. (1954). The pore pressure coefficients A and B. Géotechnique, vol. 4, 143-147.

Skempton, A. W. (1970). The consolidation of clays by gravitational compaction. Quarterly Journal of the Geological Society of London, vol. 125, 373-412.

Woodworth-Lynas, C., Blasco, S, Duff, D., Fowler, J., Işler, E.B., Landva, J., Cumming, E., Caines, A, and Smith, C. 2016. Regional assessment of seabed geohazard conditions, Canadian Beaufort outer shelf and upper slope: Legacy data synthesis: Section 2, Geophysical and geological data compilation, outer shelf and upper slope, Southern Beaufort Sea: A handbook of geohazard conditions. Environmental Studies Research Fund Report 208-2. https://www.esrfunds.org/sites/www.esrfunds.org/files/publications/ESRF208-2-Woodworth-Lynas-Blasco.pdf

Yamashita, S., 2009. International parallel test on the measurement of G_{max} using bender eelements organized by T-29. Stress and strain testing of geomaterials of international society od soil mechanics and geotechnical engineering, 76p.