GEOTECHNICAL CHARACTERIZATION AND STABILIZATION OF MUSKEG SOILS USING THE MICROBIALLY INDUCED CALCITE PRECIPITATION TECHNIQUE (MICP)

by

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GEOTECHNICAL CHARACTERIZATION AND STABILIZATION OF MUSKEG SOILS USING

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Abstract

Muskeg is the landform that describes the organic terrain. Muskeg soils are widespread in Canada. They cover around 15% of the Canadian landscape. Their problematic nature is attributed to their high compressibility and low shear strength when subjected to loads due to their high initial void ratio and water content. This study aims to investigate the problematic nature of Muskeg soils and provide a solution for their stabilization.

Water and organic contents of Muskeg soils are the two primary index properties, and they are easy and inexpensive to determine. Therefore, many studies in the literature correlate the other index and compressibility with these two simply measured properties. An extensive review of the literature was conducted, and new correlations to determine some index and compressibility properties as a function of the organic and water contents were presented. These new proposed correlations would help the design engineers provide reasonable estimations in the concept design phase for projects that deal with this soil.

Extensive laboratory and field testing programs were conducted for Muskeg soil samples retrieved from Bolivar Park, Surrey, British Columbia, Canada, to determine their geotechnical properties. New correlations for determining the maximum shear modulus G_{max} as a function of the undrained shear strength S_u, measured from the electronic vane shear test, and ball net tip resistance q_{b-net}, measured from the ball penetration test, were demonstrated. Moreover, new correlations for the undrained shear strength S_u as a function of the cone tip resistance q_{net}, measured from the seismic cone penetration test (SCPTu), and ball net tip resistance q_{b-net}, measured from test (BPT), were presented.

The excessive settlement of Muskeg soil was treated through the environmentally-friendly technique, the Microbially Induced Calcite Precipitation MICP using the urease active bioslurry approach. The results showed a substantial improvement in the compressibility properties when

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the bioslurry concentration was 0.4, the bioslurry weight was 10%, and cementation solution volume equals twice the pores' volume.

The second round of stabilization was conducted to achieve further improvement by coupling the bioslurry with sand as additives. Further improvement was achieved by adding 10% of sandy soils while keeping the bioslurry concentration at 0.4 Mole/Litre, the bioslurry weight at 10% and cementation solution volume the same as the pores' volume.

Lay Summary

Muskeg is the landform that describes the organic terrain. Muskeg soil is not a good foundation for any construction. The main reason is that it exhibits large settlements when subjected to loads, which may cause failure to the overlying structures. This study aims to help design engineers improve their understanding of this soil's behaviour in the early design and construction stages. Moreover, this study provides a solution for the excessive settlement of these soils using an environmentally-friendly stabilization technique by making use of the inherent biomass inside the soil. This technique improved the weak soil's behaviour and led to a small settlement, which will confirm the overlying structures' safety.

Preface

I, Ahmed Elmouchi, prepared all the content of this dissertation under the supervision of Dr. Sumi Siddiqua, including; literature review, statistical analysis to provide new correlations for the Muskeg soil properties, interpretation of data and analysis of results for the field and laboratory testing, methodology for the microbially induced calcite precipitation treatment using the urease active bioslurry approach, stabilization of Muskeg soils using the bioslurry only, and stabilization by using bioslurry and sand simultaneously. The co-authors of the published and submitted articles are Dr. Sumi Siddiqua and Prof. Dharma Wijewickreme. They provided technical inputs during the manuscript preparation about the articles' scientific contribution. They reviewed the articles' manuscript and provided the necessary comments to improve their quality.

I conducted all the laboratory experiments at the Geomaterials Research Laboratory and High-Head Laboratory at the School of Engineering, University of British Columbia, Okanagan Campus. The field-testing program was carried out by Conetec Investigations Ltd., the industry partner for this project, under the supervision of Stephen Renner (Masters student at the University of British Columbia, Okanagan Campus), Jennifer Liu (Masters student at the University of British Columbia, Vancouver Campus), and myself.

Part of the research contained in this dissertation is already published in a peer-reviewed journal paper and a peer-reviewed conference paper. Three journal papers are submitted for possible publication in peer-reviewed journals, as well. These papers are listed below:

 A portion of Chapter (3) has been published in the conference proceeding of the Geo-Virtual 2020 and titled "Effect of water and organic contents on the index and compressibility properties of organic soils." Canadian Geotechnical Society (CGS), September 2020.

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- A version of Chapter (3) has been published in the Geotechnical and Geological Engineering Journal (Springer Nature) in March 2021 and titled "A review to develop new correlations for geotechnical properties of organic soils." <u>https://doi.org/10.1007/s10706-021-01723-0</u>. This part is included in this dissertation after getting permission from Springer Nature.
- A version of Chapter (4) has been submitted for possible publication in the ASCE International Journal of Geomechanics and titled "Characterization of an organic soil deposit using multi-faceted geotechnical field and laboratory investigations." This part does not need permission because it is not published until the publication of this dissertation.
- A version of Chapter (6) has been submitted for possible publication in the ASCE Geotechnical and Geoenvironmental Journal and titled "Muskeg soil stabilization using the Microbially Induced Calcite Precipitation MICP technique by the Urease Active Bioslurry Approach." This part does not need permission because it is not published until the publication of this dissertation.
- A version of Chapter (7) has been submitted for possible publication in the Canadian Geotechnical Journal and titled " Stabilization of Muskeg soils using two additives: sand and urease active bioslurry." This part does not need permission because it is not published until the publication of this dissertation.

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List of Notations

ALA	American Lifelines Alliance
ANOVA	Analysis Of Variance
ASTM	American Standard Testing Method
ATCC	American Type Culture Collection
BPT	Ball Penetration Testing
с'	Effective Cohesion
Cc	Compression Index
Cr	Recompression Index
Cs	Swelling Index
Cv	Coefficient of Consolidation
Cα	Secondary Compression Index
DSS	Direct Simple Shear Testing
DST	Direct Shear Testing
е	Void Ratio
EDS	Energy Dispersive X-Ray Spectroscopy
Eur	Unload-Reload Stiffness
eVST	Electronic Vane Shear Testing
FC	Fibre Content
FHWA	Federal Highway Administration
fs	Sleeve Friction
$G_{max} Or G_0$	Maximum Shear Modulus
Gs	Specific Gravity
IL	Incremental Loading
LSD	Least Significant Difference

Μ	Constrained Modulus
MICP	Microbially Induced Calcite Precipitation
m _v	Coefficient of Volume Compressibility
OC	Organic Content
OPC	Ordinary Portland Cement
PRCI	Pipeline Research Council International
q _b	Ball Penetration Resistance
q _{b-net}	Ball Net Tip Resistance
q _{net}	Cone Net Tip Resistance
qt	Cone Tip Resistance
rpm	Revolutions Per Minute
S	Degree of Saturation
SCPTu	Seismic Cone Penetration Test
SEM	Scanning Electron Microscope
Su	Undrained Shear Strength Parameter
S _{u-peak}	Peak Undrained Shear Strength
Su-remoulded	Remoulded Undrained Shear Strength
U2	Pore Water Pressure
UCS	Unconfined Compressive Strength
Vs	Seismic Shear Wave Velocity
Wc	Water Content
wL	Liqiud Limit
XRD	X-Ray Diffraction
$ ho_{dry}$	Dry Density
ρ_{wet}	Wet Density
σ _p '	Preconsolidation Pressure

- $\sigma_{v'}$ Vertical Effective Stress
- φ' Effective Angle of Shearing Resistance

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Lastly, thanks to the person who inspired me a lot throughout my life and passed away in the second year of my Ph.D. degree. Thanks, Prof. Sherif Wissa Agaiby. Dedication

Dedicated to my father, Mohamed

My Mother, Fatma

My Sister, Soha

My Nieces, Lojy and Marwan

They are the true meaning of life, and they are the

main reason to start, continue and accomplish my

dream

Chapter 1: Introduction

In this chapter, the impetus behind this research will be presented. This impetus raised some important research questions, which this study aims at answering. The objectives of this study are listed to guide the answer to these research questions. The answers to these questions are presented throughout the thesis under separate chapters. The thesis structure and scope of this research are demonstrated by the end of this chapter.

1.1 Background

Energy pipelines are considered one of the safest and fastest ways of transporting oil and gas over vast distances. The energy industry is crucial to the Canadian economy. Buried pipeline systems form a vital part of the oil and gas infrastructure. Energy pipelines are mostly buried below the ground surface of highly variable topographic and geotechnical conditions for considerable distances. The variability of underground conditions may disrupt the performance of the energy pipeline systems, which, in turn, translates into undesirable impacts on citizens' life, communities' services, and country's economic growth. It is necessary to ensure that energy pipelines are secure and safe to protect the communities and environment, ensure our health and quality of life, and promote the country's economic growth.

Over 1.5M km² of the Canadian landscape is covered with Muskeg (organic) soils (MacFarlane, 1969; Tarnocai, 2006; Xu et al., 2018). Therefore, many of the energy pipelines have to pass in Muskeg soils, which requires a thorough understanding of Muskeg's behaviour when interacting with these pipelines. The soil-pipe interaction's simple simulation is the simulation of the soil mass around the pipeline as springs according to the American Lifelines Alliance's procedures (ALA, 2000) and Pipeline Research Council International (PRCI, 2009). The guidelines for designing the soil-pipe interactions between various pipeline materials and surrounding soils are well established for the conventional soil types such as sandy and clayey

soils. However, the soil-pipe interaction in Muskeg (organic) soils is not well defined in the literature due to the lack of understanding of these soils' mechanical properties. The current engineering design practice treats the soil-pipe interactions in Muskeg soils the same as soft inorganic soils while adding a more significant safety factor to account for this assumption's uncertainty.

For these reasons, there is a vital need to advance our understanding of the behaviour of Muskeg soils and their mechanical properties that would affect the energy pipeline design. That is why the first phase of this research is dedicated to the Muskeg soil characterization and comprises two separate sections.

The first section involves revisiting the available data in the literature and using these data to develop new correlations that enable the determination of organic soils' index and compressibility properties in the concept design stage, only by measuring their water and organic contents. The water and organic contents are inexpensive and straightforward to measure. Although there are similar relations in the literature that estimate the organic soil properties based on the water content, the new proposed correlations would cover a wide range of the water contents that have not been covered before. Moreover, the new correlations add another factor for better interpretation of the organic soil properties, which is the organic content.

The second section is the thorough geotechnical characterization of Muskeg soils retrieved from Bolivar Park, Surrey, British Columbia, Canada. The characterization comprises multi-faceted field and laboratory tests, which compare the actual intact soil behaviour in the field and disturbed samples' behaviour in the laboratory.

Muskeg soils are distinguished by their high initial water content, high initial void ratio, high compressibility, high hydraulic conductivity, high angle of internal friction, low bulk density, and low specific gravity (Adams, 1965; MacFarlane, 1969; Dhowian & Edil, 1980; Landva, 1980; Lefebvre et al., 1984; Mesri & Ajlouni, 2007). Therefore, they exhibit different mechanical properties than inorganic soils' conventional behaviour (Edil & Wang, 2000). Due to the

unfavourable engineering properties of Muskeg soils, any construction that involves dealing with Muskeg soils requires many precautions in both the design and construction phases. That is the reason why organic soils are considered problematic.

Enhancing the geotechnical properties of Muskeg soils would help in solving the problems related to the soil-pipe interaction. It would decrease the pipe's differential settlement and induced stresses, which would decrease the cost and anticipated risk of the energy pipeline. Various methods have been used to stabilize and enhance the geotechnical properties of organic soils. The most common method is chemical grouting using cement, epoxy, lime, and fly ash. However, chemical grouting requires a high cost for energy used in production and installation. It increases the carbon dioxide CO_2 emissions in the surrounding environment. Also, it might affect the quality of underground water (DeJong et al., 2010). The current universal trend for soil stabilization is heading to use environmentally-friendly techniques to overcome the drawbacks of chemical grouting techniques. One of these new techniques relies on using the bacteria as a catalyst to achieve soil stabilization through Microbially Induced Calcite Precipitation (MICP), which is commonly known as "Bio-cementation" or "Bio-grout." Much research has applied this technique to sandy soils and checked the sand performance before and after treatment (Cheng & Shahin, 2016; DeJong et al., 2006). The results showed a significant improvement in the sand properties. The main reason for using this technique for sandy soil is the bacteria's ability to live and work as a catalyst in relatively large pores compared to the tiny pores of clayey and silty soils. Their high initial void ratio distinguishes Muskeg soils. Therefore, this technique is considered a promising technique for organic soil stabilization.

With this impetus, the second phase of this research encompasses two different organic soil stabilization methods using the MICP technique.

The first method comprises the mechanical mixing of Muskeg soil with the urease active bioslurry. The urease active bioslurry is one of the MICP approaches proposed by Cheng & Shahin, 2016.

The second method aims to further improve the Muskeg soil's behaviour by mechanical mixing of the soil with bioslurry and sand such that the sand would work as a filling material. Specific changes in the urease active bioslurry's proposed methodology for the preparation and application into Muskeg soils have been considered to account for the natural variation between sandy and Muskeg soils. Comparing the organic soil behaviour before and after treatment would help better understand the additives' optimum combination to stabilize these soils.

1.2 **Problem Statement**

Due to the increasing number of populations worldwide, there is an increasing demand for extending and delivering energy sources for many recent locations. Constructing energy pipelines buried in various soil deposits is a challenge for the oil and gas industry. Geo-hazards such as slope movement, landslides, and subsidence would cause a permanent deformation for a specific segment of the energy pipelines (Daiyan et al., 2011). The relative displacement between the energy pipelines and surrounding soils will impose extra stresses in the energy pipelines' bodies. If stresses are increased to a specific level that was not accounted for in the design stage, severe problems can occur.

Muskeg soils cover around 15 % of the Canadian landscape. The energy pipelines in certain situations would have to pass through these Muskeg soils. Muskeg soils' problem is that there are no guidelines on how the soil would behave under different loadings around the energy pipelines. However, there are guidelines and a better understanding of the pipe behaviour around sandy and clayey soils (ALA, 2000; PRCI, 2009). Further and thorough investigation of these soils is the first step for better understanding these soil behaviour around pipelines.

Muskeg soils are known for their high initial water content and void ratio, making them weak in strength and induce very high settlement under various loading conditions. Burying energy pipelines in these weak soils would expose the energy pipelines to high risks. Therefore, before constructing over these soils, a soil stabilization technique needs to be applied to enhance

these soils' geotechnical properties. The most common technique for stabilizing the soil mass is the cement grouting technique. This technique's major problems are the high cost of production and installation, the harmful effect on human health due to changing the underground water quality, and it is not an environmentally-friendly technique as it emits a large amount of CO₂ into the surrounding environment. This problem's solution has been incorporated using an environmentally-friendly additive to the soil mass like the microorganisms used in the Microbially Induced Calcite Precipitation (MICP) technique.

Most of the previous research focused on applying the MICP technique to sandy soils due to their high initial void ratio compared to clayey soils. Adding the MICP to Muskeg soil is a challenge due to the various natural properties between sandy and Muskeg soils. Developing a new method for applying this technique in Muskeg soils would help figure out the optimum components to enhance these soils' properties using the MICP technique.

1.3 Research Questions

Based on the problems presented above, the following research questions have been selected to be answered throughout the current research:

- (1) Based on the available data of many organic soils worldwide, can the easily measurable water and organic contents be used to determine the index and compressibility properties of any organic soil?
- (2) Are the available correlations for mineral soils to interpret the maximum shear modulus G_{max} and undrained shear strength S_u from cone and ball penetration field tests considered applicable for Muskeg soils? If not, what are the new correlations to be used for Muskeg soils?
- (3) What is the effect of the sample disturbance on the compressibility properties of Muskeg soils?

- (4) What are the optimum bioslurry weight, bioslurry concentration, and cementation solution volume to achieve a satisfactory stabilization level for Muskeg soils?
- (5) What is the optimum sand percentage by weight to be added to the optimum bioslurry weight that would further enhance organic soils' mechanical properties?

1.4 Objectives

This study's overall objective is to present a thorough geotechnical investigation of Muskeg soils and enhance their performance using the Microbially Induced Calcite Precipitation MICP stabilization technique. This could be achieved through the pursuit of the following short-term objectives:

- Develop new correlations to determine the index properties and compression index tentative values of organic soils in the concept design stage from the easily measurable water and organic contents.
- Present new correlations to interpret the maximum shear modulus G_{max} and Undrained shear strength S_u for Muskeg soils from field testing.
- Evaluate the effect of sample disturbance on the compressibility properties of Muskeg soils.
- Determine the optimum urease active bioslurry weight, concentration and cementation solution volume for the Muskeg soils' stabilization.
- 5) Determine the optimum sand weight by percentage to be added to the optimum bioslurry combination to enhance the compressibility properties of Muskeg soils further.

1.5 Thesis Organization

This dissertation is prepared based on articles submitted or published in peer-reviewed journals and conference proceedings to achieve the pre-listed objectives. It consists of eight chapters in total, and a summary of each chapter is described below: **Chapter 1** presents the background and impetus for conducting this research. Clear objectives are determined to answer the raised research questions. The thesis organization is also clarified.

Chapter 2 expounds on the literature review for the two phases of this study. The first part reviews the geotechnical properties of organic soils, including; their classification, index properties, compressibility properties, and shear strength properties. The second part presents the review for the Microbially Induced Calcite Precipitation MICP technique, including; the developed strength mechanism, factors affecting the calcite precipitation, effect of MICP on properties of treated Soils, applications, advantages, and limitations of the technique.

Chapter 3 shows the developed correlations for the tentative determination of the index and compressibility properties of organic soils as a function of the easily measured water and organic contents.

Chapter 4 demonstrates the geotechnical characterization of the Muskeg soil deposit collected from Bolivar Park, Surrey, British Columbia, Canada, using multi-faceted field and laboratory testing. The various tests used and their methodology are presented along with the results and interpretations. New correlations for determining the maximum shear modulus G_{max} from the measured undrained shear strength S_u from the electronic vane shear test and ball net-tip resistance q_{b-net} from the ball penetration test are presented. Furthermore, the constants usually used for determining the undrained shear strength S_u from the net tip resistance q_{net} of the seismic cone penetration test and ball net tip resistance q_{b-net} from the ball penetration test are interpreted for Muskeg soils.

Chapter 5 describes the followed methodology for Muskeg soil stabilization using the Microbially Induced Calcite Precipitation MICP technique by the urease active bioslurry approach.

Chapter 6 introduces the results of stabilizing Muskeg soil by mechanically mixing with the urease active bioslurry. The effect of changing the bioslurry concentration, the bioslurry weight and cementation solution volume on the compressibility properties are presented. The effect of stabilization on the shear strength behaviour is presented as well.

Chapter 7 elucidates the results of the stabilization of Muskeg soils with bioslurry and sand simultaneously. The effects of changing the sand percentage and injected cementation solution volume on the mechanical properties are demonstrated.

Chapter 8 demonstrates the summary, significant findings and conclusions of this study. The originality and major contributions are presented as well. Finally, the limitations of this study and recommendations for future work are discussed.

Chapter 2: Literature Review

In the first part, this chapter presents a review of the basic mechanical properties of Muskeg (organic) soils, including index, compressibility, and shear strength properties. The second part aims to demonstrate a review for the Microbially Induced Calcite Technique MICP that enhances the soil properties using environmentally-friendly organisms.

2.1 Geotechnical Properties of Organic Soils

In this part, a review of organic soil classifications will be presented. Afterward, a description of the geotechnical properties of organic soils would be demonstrated.

2.1.1 Geotechnical Classification

The particle size distribution, texture, and Atterberg limits are the primary components for mineral soil classification as per the Unified Soil Classification System (USCS) (ASTM-D2487, 2017). However, these components do not apply to the organic soil classification, as Atterberg limits cannot be measured in organic soils. Besides, it is not feasible to determine the grain size distribution in organic soil for classification. Texture, mineral content, organic content, fibre content, degree of humification, and pH are the properties that can be used for effectively classifying organic soils (Ajlouni, 2000).

Von Post (1922) presented a scale to determine the degree of humification for organic soils, ranging between H1 for the un-decomposed peat to H10 for the completely decomposed peat. The Von Post determination for the degree of humification is used and standardized by the ASTM-D5715 (2014). MacFarlane (1969) proposed a classification of Canadian peat into seventeen categories based on the fibres' nature and divide them into three major groups (amorphous granular, fine fibrous, and coarse fibrous).

The ASTM-D4427 (2018) proposed the classification of peaty soils based on five characteristics as follows: (i) Fibre Content: fibric, hemic, and sapric (ASTM-D1997, 2013), (ii) Ash Content: low, medium and high ash (ASTM-D2974, 2014). The ash content is a direct measurement of the organic content, such that the organic content equals 100% minus the ash content percentage (i.e., 25% of ash content indicates 75% organic content). Moreover, the ASTM defined the peaty soils as soils that have organic content higher than 75%, (iii) Acidity pH: highly acidic, moderately acidic, slightly acidic, and basic peat (ASTM-D2976, 2015) (iv) Absorbency: extremely, highly, moderately, and slightly absorbent (ASTM-D2980, 2017), and (v) Botanical Composition (if single or two names of the botanical designations should be added to the peat naming, at least 75% of the fibre content of this peat should be derived from these botanical designations).

As per the ASTM-D1997 (2013) and ASTM-D5715 (2014), the degree of humification is related to the fibre content. In other words, fibric peat has a degree of humification between H1 to H3, the hemic peat has a degree of humification between H4 to H7, and sapric peat has a degree of humification between H8 to H10. The ASTM classification for peat is preferred over other classification systems because it includes different index properties that can distinguish most peaty soils concerning their geotechnical engineering properties. However, the ASTM-D4427 (2018) classification does not mention how to deal with the soil's fabric or texture when calculating the different index properties.

From a geotechnical perspective, organic content's cut-off value distinguishes between peaty and organic soils is not universally agreed upon. **Figure 2.1** shows the different cut-off values between peat and organic soils based on the soil's organic content in different countries (after Anderjko et al., 1983; Wolski et al., 1988). The figure reveals three significant categories: peat, which has high organic content; muck, which has moderate organic content; and mineral organics, which has low organic content. The fixed limits between these three categories are not unique, as the transition in behaviour between the three categories is relatively subtle (Farrell et
al.,1994). According to McVay & Nugyen (2004), a tentative standard classification system to distinguish between these three categories was considered by the ASTM.



Figure 2.1 Comparison of classification systems used for peat and organic soils in various countries (after Anderjko et al., 1983; Woliski et al., 1988). *Reprinted with permission of ASTM International.*

2.1.2 Index Properties

To use organic soils in any engineering application, the various soil properties should be interpreted. The following physical properties are of great importance to organic soils' engineering behaviour: water content, organic content, fibre content, void ratio, bulk density, and specific gravity.

2.1.2.1 Water Content (w_c)

One of the distinctive properties of organic soils is their high-water content (Mesri & Ajlouni, 2007). The high-water content is attributed to the large voids formed by the fibre structure and high cation exchange capacity of the organic matter, which increases the attraction of water molecules (FHWA, 2009). Approximate estimation of the organic content, bulk density, specific gravity, compressibility, and shear strength parameters can be obtained using available correlations by measuring the water content. Ajlouni (2000) concluded that the origin, degree of decomposition, and chemical composition of organic soils are the main factors responsible for their wide range of water contents.

The laboratory determination for the water content of organic soils is similar to mineral soils. A representative sample of organic soils is dried in the oven at 110° C, and weight of the water expelled from the wet sample divided by the weight of the dry sample provides the water content, as per (ASTM-D2974, 2014). Miyakawa (1959), Goodman and Lee (1962), and MacFarlane and Allen (1964) proposed an oven temperature of 85°C to avoid burning the organic content. Afterward, Skempton and Petley (1970) investigated the effect of drying the organic soil samples using different temperatures and concluded that low temperatures retain an amount of water within the organic soil particles, whereas at 110°C, the loss of organic material was minimized. O'Kelly and Sivakumar (2014) concluded that ASTM-D2974 (2014) suggested the proper method of determining the water content.

The amount of water held in fibrous peat is more than the amount of water held in amorphous peat. There are five sources for the water content in organic soils (Ajlouni, 2000): (i) free water in pores between particles, (ii) free water in pores inside particles, (iii) double layer (osmotically bond) water around all surface areas, (iv) adsorbed water covering all the surface areas and (v) water within and between components due to chemical bonds.

2.1.2.2 Ash Content and Organic Content (OC)

Organic soils, in general, have two major components: the organic component and mineral component. The organic component is carbonaceous and combustible. Whereas the mineral component is incombustible and ash forming (MacFarlane, 1969). The organic content plays a

vital role in organic soil's classification and its mechanical properties. The higher the organic content, the higher the water content, void ratio, and compressibility (Abdel Kader, 2010).

Measuring the organic content requires measuring the ash content in advance. According to ASTM-D2974 (2014), an organic soil sample's ash content is defined as the percent of the remaining substance's weight after igniting the oven-dried samples in a furnace at $440^{\circ} \pm 40^{\circ}$ C to the weight of the oven-dried sample. Afterward, the organic content is determined by subtracting the ash content percentage from one hundred percent.

2.1.2.3 Fibre Content (FC)

The fibre content of organic soils is determined in the laboratory following the ASTM-D1997 (2013) standard. The fibre content is the ratio between the mass of fibres, which is retained on the 150 µm sieve (No. 100 sieve), to the solids' weight in the sample. Organic soils are classified according to their fibre content. High fibre content will result in less decomposition. According to the ASTM-D4427 (2018) classification, organic soils are classified as fibrous peat if the fibre content is higher than 67%, corresponding to a degree of humification between H1 to H3 (less decomposed peat) on the Von Post scale. The amorphous peat contains a fibre content of less than 67%. Organic soils with fibre content between 33% and 67% are defined as amorphous hemic soil, which corresponds to a degree of humification between H4 to H6 (moderately decomposed) on the Von Post scale. Fibre content less than 33% refers to highly decomposed soils, H7 to H10 on the Von Post scale, and is defined as amorphous sapric soil.

Fibre content affects other organic soil properties such as water content, organic content, bulk density, void ratio, hydraulic conductivity, shear strength, and compressibility.

More organic material in the soil leads to higher water content, leading to less favourable engineering parameters. Higher fibre content also results in higher water content but results in more favourable soil properties (Den Haan, 1997; Edil & Wang, 2000; Al Adili, 2013). Price et al. (2005) failed to determine a correlation between bulk density and fibre content or degree of

humification and fibre content, although the Von Post humification indicates the loss of any organic soil structure (Mesri & Ajlouni, 2007; O'Kelly & Pichan, 2014). This could be attributed to the measurement of fibre content without considering the soil's fabric dimensions. However, MacFarlane (1969) correlated the bulk density with high fibre content greater than 50% and indicated a decreasing trend for the density with increasing fibre content. Landva & Pheeney (1980) concluded that knowing the type of fibres, like sedge or leave, would be advantageous for estimating the soil suitability for engineering purposes. Correlations based on fibre content, the soil's fabric, and associated engineering properties should be further investigated.

2.1.2.4 Void Ratio (e)

Organic soils are known for their high initial void ratio. A high void ratio reflects high water content, high hydraulic conductivity, and high compressibility. Fibrous organic soils have a higher void ratio than amorphous soils due to the fibre structures. MacFarlane (1969) mentioned that the void ratio of organic soils ranges between 5 and 15. The void ratio may reach as high as 25 in fibrous soils and as low as 2 in amorphous peat. The void ratio is correlated to the water and organic contents. There is an increasing linear relation between the water content and void ratio, as both the water content and void ratio are mathematically linked in the phase diagram calculations (MacFarlane 1969). McVay & Nugyen (2004) reported an increasing relation between the void ratio and organic content for void ratios ranging between 1 to 9.

2.1.2.5 Bulk Density (ρ)

Their low densities feature organic soils compared to mineral soils. ASTM-D4531(2015) describes the procedures for measuring the bulk and dry densities of organic soils using the Paraffin wax method. The bulk density of organic soils is close to water density, and any significant increase in the bulk density would be attributed to the mineral components (MacFarlane, 1969).

The bulk density of organic soils is correlated to both the water and organic contents. The bulk density of organics soils equals roughly the water density for organic soils of higher than 500% water content. This could be attributed to the high fibre and organic contents at high water content (MacFarlane, 1969; Hobbs, 1986; Bell, 2000). The bulk density is frequently correlated to the water content in the literature. However, Den Hann & El Amir (1994) proposed a correlation between the bulk density and organic content for Dutch peat of high organic contents.

2.1.2.6 Specific Gravity (G_s)

The specific gravity of organic soils is smaller than the specific gravity of mineral soils. Higher organic content leads to smaller specific gravity, as the specific gravity mainly depends on the inorganic components (MacFarlane, 1969; Ajlouni, 2000). Specific gravity for organic soils ranges between 1 and 2 (MacFarlane, 1969; Lechowicz et al., 1996; Bell, 2000). Higher G_s reflects high mineral contamination (FHWA, 2009). Specific gravity is measured according to ASTM-D854 (2014) using a water pycnometer. However, more accurate results could be achieved using the gas (kerosene) pycnometer as per ASTM-D5550 (2014). Measuring G_s of organic soils is a reasonably difficult process as it might not be possible to remove all entrapped air or gases inside and between particles (Ajlouni, 2000).

Organic content is correlated to specific gravity. Lea & Brawner (1963), Skempton and Petley (1970), and Den Haan (1997) proposed different equations for calculating the specific gravity of organic soils as a function of the organic content. Landva & La Rochelle (1983) proposed the extreme limits for organic soils' specific gravity as a function of the organic content.

2.1.3 Compressibility Properties

Their high compressibility distinguishes organic soils, and they exhibit high volume changes under loading (MacFarlane, 1969; Kogure, 1999; O'Kelly, 2015; Dehghanbanadaki et al., 2017). Organics soils undergo significant and rapid short-term primary consolidation (until the complete

dissipation of the pore water pressure) and continuous long-term secondary consolidation due to the particle compressions and plastic yielding of the solids (Adams, 1965; MacFarlane, 1969; Samson & La Rochelle, 1972; Kogure et al., 1993; Fox, 2003; Govar, 2007; O'Kelly, 2009; Long & Boylan, 2015). Some researchers argued that organic soils exhibit tertiary consolidation after secondary consolidation (Candler & Chartres, 1988; Kazemian et al., 2009). However, this amount of compression can be neglected because it generally occurs after the structures' design life (Dhowian & Edil, 1980; Fox & Edil, 1994).

The different compressibility indices of organic soils, such as compression index C_c , recompression index C_r , swelling index C_s , coefficient of consolidation C_v , preconsolidation pressure P_c , and secondary compression index C_α , are discussed in the following paragraphs.

2.1.3.1 Recompression (C_r), Compression (C_c), and Swelling (C_s) Indices

The recompression index C_r represents the soil behaviour under loading until the preconsolidation pressure is achieved. The preconsolidation pressure is the highest pressure the soil has been subjected to throughout its history. Many of the consolidation tests for organic soils do not show the recompression component in the e - log σ' curve due to the small values of the preconsolidation pressure σ_p' and sample disturbance (Barden, 1969; Berry, 1983). However, the reported values for the C_r/C_c ratio for organic soils range between 0.1 to 0.3, and this range is close to the range for clayey soils (Ajlouni, 2000; Abdel Kader, 2010).

The compression index C_c shows the soil deformation behaviour under increased applied loads to simulate new loads that the soil is expected to be exposed. The compression index of organic soils is relatively high compared to clayey soils, such that the C_c ranges between 0.5 to 18 for organics soils compared to 0.2 to 0.8 for clay. (Lefebvre et al., 1984; Yamaguchi et al., 1985; Ajlouni, 2000; Abdel Kader, 2010). The relatively high C_c for organic soils is attributed to the high initial void ratio and high water content. C_c is not constant for organic soils and is dependent on the applied consolidation pressure. C_c increases sharply in the normal consolidation range and decreases after twice the preconsolidation pressure while increasing the applied consolidation pressure (Kogure & Ohira 1977; Yamaguchi et al., 1985; Mesri et al., 1997; Santagata et al., 2008).

The swelling (rebound) index C_s of organic soils is determined in the unloading stages through the unloading/reloading cycle in the consolidation test. There is not much data in the literature for the swelling index C_s . The swelling index C_s is related to both the organic content and preconsolidation pressures, such that high organic content and high preconsolidation pressure lead to a high swelling index (Lefebvre et al., 1984; Yamaguchi et al., 1985). The C_s/C_c for organic soils is reported between 0.1 and 0.3, which is the same as clayey soils (Hobbs, 1986; Ajlouni, 2000).

2.1.3.2 Coefficient of Consolidation (C_v)

According to Terzaghi et al. (1996), the coefficient of consolidation of clayey soils is almost constant while increasing the applied consolidation pressure, as the primary consolidation of clayey soils may take years or even decades to complete. The long period required for the primary consolidation is due to the small hydraulic conductivity of clayey soils. On the other hand, organic soils are distinguished by their high initial void ratio and hydraulic conductivity. Hence, the primary consolidation is rapid and may require only a few weeks or months to complete. The rapid primary consolidation of organic soils due to the decrease of hydraulic conductivity while increasing the applied consolidation pressure is the main reason for the variable coefficient of consolidation C_v (Lea & Brawner, 1963; Berry & Vickers, 1975; Mesri, 1997). However, the log time method proposed by Casagrande & Fadum (1940) and square root time method presented by Taylor (1942) is generally used to determine C_v for organic soils (Huat et al., 2011).

Santagata et al. (2008) presented the variation of the coefficient of consolidation C_v with changing the applied stress level for soft clay, muck peaty soils and fibrous peaty soils, and confirmed the change of C_v with stress level for organic soils and stability of C_v with stress level

for clayey soils. Moreover, the decrease in C_v while increasing the stress level is significantly marked in soils with high organic content (Farrell et al., 1994). Oikawa and Igarashi (1997) proposed an equation to calculate the C_v directly without conducting the consolidation test based on the initial water content and applied consolidation pressure.

2.1.3.3 Preconsolidation Pressure (σ'_p)

Their low preconsolidation pressure values generally characterize organic soils due to their small unit weight and location close to the ground surface. The primary source of preconsolidation pressure for organic soil is the structure of the composing planet. Other natural activities, such as drainage, age, and decomposition, may increase or decrease the σ'_{p} (Hobbs, 1986; Ajlouni, 2000).

Based on the consolidation test's quality, the preconsolidation pressure is determined using the same method for the clayey soils. However, in many consolidation tests carried out on organic soils, the point of maximum curvature in the e - log σ ' curve was not marked, and determination of the preconsolidation pressure was difficult (Mesri et al., 1997). The preconsolidation pressure of organic soils is correlated to the void ratio (Kogure & Ohira, 1977; Ajlouni, 2000). The general trend showed a linear decrease of the preconsolidation pressure with the increase in the void ratio.

2.1.3.4 Secondary Compression Index (C_α)

The primary consolidation of organic soils is rapid, significant, and always occurs in the early stages of construction (MacFarlane, 1969; Mesri et al., 1997). Hence, the estimation of settlement due to the secondary consolidation of organic soil deposits is essential, as it occurs during the structure's design life (Ajlouni, 2000). For mineral soils, C_{α} is generally related to C_{c} by the (C_{α} / C_{c}) ratio. The same ratio (C_{α} / C_{c}) is presented for organic soils (Samson & La Rochelle, 1972; Lefebvre et al., 1984; Jorgenson, 1987; Mesri & Ajlouni, 2007). The typical range of C_{α} / C_{c} for

mineral clayey soils is 0.02 ± 0.01 (Mesri et al., 1997; Abdel Kader, 2010). Whereas for organic soils, the C_{α} / C_{c} range is 0.05 ± 0.01 (Mesri & Ajlouni, 2007). The value of C_{α} of organic soils increases by increasing the applied consolidation pressure (Lea & Brawner, 1963; Adams, 1965; Barden, 1969).

2.1.4 Shear Strength of Organic Soils

The suitability of the soil mass, upon which an engineering project should be rested, requires the determination of both the compressibility and shear strength properties. Organic soils are distinguished by their high compressibility and low shear strength properties. The compressibility and shear strength properties are improved when these soils are subjected to loads and consolidated (Lea & Brawner, 1963; Samson & la Rochelle, 1972; Lechowicz, 1994: Edil & Wang, 2000). For amorphous organic soils, they exhibit low shear strength properties from the inorganic component's apparent cohesion. For the fibrous peat, they gain strength from the friction resistance due to the fibre interaction such that the fibres act as reinforcement to the soil sample. However, for the fibrous peat in the loose state, the fibres do not interact together, and corresponding shear strength is small, like the amorphous peat (Arman, 1969; Amaryan et al., 1973; Landva & La Rochelle, 1983; Kazemian et al., 2012).

The main three shear strength parameters for organic soils are the undrained shear strength parameter S_u , effective cohesion c', and effective angle of internal friction ϕ '.

The undrained shear strength parameter S_u is determined either in the laboratory or field. The laboratory tests used to determine the undrained shear strength parameter are the simple shear test or the triaxial test. The S_u could be determined in the field by the full-flow penetrometer test, pressuremeter test, field vane test, the cone penetration test. Field vane tests are preferred over the cone penetration test by many researchers (MacFarlane, 1969; Huang, 1982; Landva & La Rochelle, 1983; Ajlouni, 2000). However, recent research prefers the full-flow penetrometer test as it is more suitable for weak soils that flow like offshore soils, and it does not disturb the soil mass to the same extent as the field vane test and cone penetration test (Boylan et al., 2011). The undrained shear strength of organic soils is generally small and is correlated to the effective overburden pressure on-site or to the preconsolidation pressure such that the S_u/σ'_o ranges between 0.36 to 0.68 (Abdel Kader, 2010).

The effective strength parameters c' and φ' are determined using the laboratory tests such as the triaxial and direct shear tests, which are the two standard laboratory tests for the shear strength determination of organic soils. It is essential to choose the appropriate test for determining the shear strength parameters of organic soils based on the engineering application. For most applications, such as road embankments, bearing capacity, and dikes, the triaxial compression test is the most reasonable as it simulates these applications' same state conditions. The direct shear test should be adopted for the slope stability analysis, as the failure plan is forced to be horizontal to simulate the horizontal part of the slope stability slip surface. O'Kelly (2017) presented a detailed review of fibrous peat's shear strength and recommendations for the measurements and interpretations of the shear strength properties.

There are several trials to correlate the water and organic contents with the shear strength parameters. However, as long as the fibre content and fibres' texture are not presented in the correlations, these correlations should only be used as rough guidance in the project's preliminary design stage.

2.1.4.1 Undrained Shear Strength Parameter Su

The undrained shear strength parameter of organic soils is better to be determined by in-situ testing. MacFarlane (1969) correlated the measured undrained shear strength parameter S_u using the field vane testing with the water content. There are no earlier trials that correlated the undrained shear strength parameter S_u with the organic content. The data available in the literature for measuring undrained shear strength parameter and organic content are not adequate for empirical correlations, as the available data only covers the high organic content.

2.1.4.2 Effective Cohesion c'

Their small cohesion distinguishes organic soils. The triaxial compression and direct shear test are the two most commonly used laboratory tests for determining the effective shear strength parameters. The Consolidated Undrained CU test is the most suitable test for organic soils. The organic soil behaviour starts as drained behaviour at minimal stresses due to the high initial void ratio and hydraulic conductivity, but while the load increases, the behaviour rapidly transfers to the undrained behaviour when the soil consolidates (Dhowian & Edil, 1980).

2.1.4.3 Effective Angle of Shearing Resistance φ'

The primary source of the induced φ' in organic soils is the fibre interactions among each other, or with minerals, and that explains why the φ' is small at small strains. There are many trials to correlate the effective angle of shearing resistance φ' with water content, but the range is extensive. Landva & Pheeney (1980) stated that there is no strong empirical relationship that has been developed yet for the shear strength of organic soils based on water content.

2.2 Microbially Induced Calcite Precipitation (MICP)

The increasing number of populations worldwide made it necessary for massive infrastructure construction to accommodate the populations' basic needs, especially in big cities. The competency of the underlying soils governs the further expansion and renovation of infrastructure in any city. That is why the ground improvement projects reached up to 40,000 projects worldwide, with a total cost of around US\$6 billion/year (DeJong et al., 2010).

On the other hand, the environmental conditions are degrading worldwide due to the global warming problems, which caused an increase in the sea level due to the increasing carbon emission percentage. One of the major contributors to the increasing percentage of carbon emission is cement manufacturing, which is used intensively in ground improvements.

There is a vital need for an environmentally-friendly technique to enable the cement's dispensation for these two main reasons. The interdisciplinary research between microbiology, geochemistry, and geotechnical engineering resulted in using biological processes to improve the soil properties by the precipitation of the calcium carbonate $CaCO_3$ in the soil mass, which binds the soil particles together.

2.2.1 Current Soil Improvement Practice and the Need for Sustainable Technique

The current soil improvement practice involves one of these three techniques: (1) Mixing the soil with a synthetic or human-made material such as geo-synthetics, recycled glass fibres or tires, and fruit brunches (Mujah et al., 2013, 2015); (2) Chemical and cement/lime grouting using the injection method (Bahmani et al., 2014; Di Sante et al., 2015); (3) use of sand or stone columns (Dash and Bora, 2013; Zhang et al., 2013). There are some significant problems associated with these techniques as these techniques depend on either mechanical energy or human-made materials, which require high energy for production and installation.

Chemical grouting is the most common technique out of these techniques. Chemical grouting usually decreases the soil hydraulic conductivity, which limits the treatment to the top layers. Moreover, usually, chemical grouting increases the pH of the underground water to high alkaline levels and may cause many environmental problems (Mujah et al., 2017). As a result, some countries banned the use of chemical grouting due to the severe health problems associated with it, such as Japan (Karol, 2003).

Due to the mentioned limitations and problems in the current engineering practice for soil improvements, exploring a new stabilization technique that is environmentally-friendly and sustainable is necessary to fulfill the increasing demand for soil improvement projects related to the infrastructure. Hence, the Microbially Induced Calcite Precipitation is presented by many researchers to make use of the metabolic pathway of the bacteria to help in hydrolyzing the urea and precipitate the calcium carbonate CaCO₃ in the soil mass, which works as a bonding agent

between the soil particles (DeJong et al., 2006; Whiffin et al., 2007; Harkes et al., 2008; van Paassen et al., 2008). Most of the research associated with this technique is limited to sandy soils due to their large initial void ratio compared to the clayey soils, enabling the bacteria to attach to the particle surface and work as a nucleation site for the calcite precipitation.

2.2.2 Overview of the MICP Technique

The MICP technique mainly depends on urea hydrolysis through a specific type of bacteria and precipitation of calcium carbonate CaCO₃ (Calcite) in the soil mass. The main advantage of using the urea hydrolysis technique is that it is considered an easily controlled process, straightforward, and could generate up to 90 % chemical conversion efficiency over a short period (Al-Thawadi 2011).

Bacillus pasteurii or *Sporosarcina pasteurii* are bacterial strains distinguished by their high urease activity, which precipitates a large amount of calcium carbonate within a short period. That is why they are the most used and preferred bacterial strain for the MICP stabilization technique (Bang et al., 2001; Wei et al., 2015).

The bacteria should be grown and cultivated using a pure strain under sterile conditions to prevent contamination and growing of negative-urease bacteria in the culture (Al-Thawadi, 2008). That is why Cheng & Cord-Ruwisch (2013) concluded that the production of highly active ureolytic bacteria is one of the significant factors in the cost of upscaling and field applications for the MICP technique. They succeeded in producing highly active urease bacteria on-site using a non-sterile chemostat culture at specific conditions, which enabled them to reproduce continuous bacteria on site.

The process of precipitating the MICP within the soil mass comprises two stages: (1) urea hydrolysis: at which the bacteria works as a catalyst to speed the urea hydrolysis process to produce ammonium and carbonate as stated in **Equations [2.1, 2.2, and 2.3]**, (2) calcite precipitation: in this stage, the produced carbonate from stage one reacts with the calcium ions

derived from the calcium chloride CaCl₂ to form the CaCO₃ crystals, as shown in **Equation [2.4]**. The process and chemical reactions are shown in **Figure 2.2**.



The bacteria play an important role in this process such that they catalyze the urea hydrolysis, increase the pH, provide the preferred environment, and work as nucleation sites for the CaCO₃ precipitation. Ferris et al. (2004) explained the process of the CaCO₃ formation in three stages: (1) Ensuring reaching the supersaturation state of the solution, (2) nucleation at the point of critical supersaturation, (3) spontaneous crystal growth.



Net Urea Hydrolysis Reaction: $NH_2-CO-NH_2 + 3H_2O \rightarrow 2NH_4^+ + HCO_3^- + OH^-$ Net pH increase: [OH⁻] generated from NH_4^+ production >> [Ca²⁺]

Figure 2.2 Overview of MICP technique (after DeJong et al., 2010). *Reprinted with permission of ELSEVIER*.

2.2.3 Spatial Distribution of Calcite and Failure Mechanisms

The produced calcite (CaCO₃) is distributed through the soil pores, particularly in the particle to particle contact, and helps in bonding the soil particles together. The calcite particles' distribution is of great importance as a considerable amount of the CaCO₃ could be precipitated in the soil mass but at unfavourable locations such as around the particle and do not form a bridge between the consecutive particles. In this case, regardless of how big is the amount of the produced calcite, the effect of this precipitation on the soil behaviour is limited to decreasing the soil hydraulic conductivity and will not enhance the soil stiffness or shear strength.

DeJong et al. (2010) presented a figure that reveals the three different configurations of the Calcite precipitation around the soil particle and three different shapes of these bonds' failure, as shown in **Figure 2.3**.



Figure 2.3 (a) Spatial distribution of the precipitated calcite between two particles, (b) alternatives failure mechanisms of the connection between the precipitated calcite and soil particles (after DeJong et al., 2010). *Reprinted with permission of ELSEVIER.*

The spatial distribution is shown in **Figure (2.3a)** and shows two extremes: uniform distribution and preferential distribution. The uniform distribution shows that the calcite covers the perimeter of each particle in thin layers. Therefore, the bond between the particles is minimal, and no significant change occurs in the soil properties. The preferential contact is when all the calcite crystals precipitate between the two particles, which give more bonds and better strength properties for the soil mass. Both of these two distributions are extremes and not realistic. The actual distribution is a mixture of these two distributions. Some calcite precipitates around the particle perimeter and others in the particle to particle contact in the actual distribution. The important note is that the amount of calcite precipitates between the soil particles is significant and helps enhance the soil's geotechnical properties.

The failure mechanism of the calcite-particle bond under various loading schemes (compression, shearing, and tension) shown in Figure **(2.3b)** reveals that the actual failure mechanism comprises a portion due to the failure of the calcite-particle bond line and a portion due to the failure of the weak structure of the calcite itself.

2.2.4 Soil Stabilization Using the MICP technique

As previously mentioned, the MICP technique comprises two stages: (1) adding the bacteria to the soil matrix, (2) adding the cementation solution that contains the calcium cations to form the calcite. Introducing the bacteria to the soil mass is crucial and of high importance to ensure the retention of the bacteria in the soil mass during the subsequent injections of the cementation solution. Improper bacterial retention in the soil mass, which would be avoided as it would lead to uneven distribution of calcite in the soil mass, which would cause unfavourable engineering properties. There are three different methods to add the bacteria to the soil mass: (1) injection method, (2) surface percolation method, and (3) premixing method.

In this research, the soil is different than what is usually used in the literature. The soil used in the literature for the various techniques is sandy soils. The soil used in the current research is organic. The mechanical mixing method using the urease active bioslurry approach is adopted in this research.

2.2.4.1 Injection Method

The first step of this method is to introduce the bacterial culture to the soil mass via injection. The bacterial culture usually includes bacteria + urea as the two main components. Other chemicals could be added to help the bacteria survive, such as the broth, or help in the ammonium reaction and stabilization of the pH, such as the sodium bicarbonate and ammonium chloride (DeJong et al., 2011; Al Qabany et al., 2012). Afterward, a retention period between 4 to 6 hours is allowed to help the bacteria diffuse through the soil pores and attach to the soil particles.

The second step comprises injecting the cementation solution, which is the same solution used in the first step with CaCl₂ and without the bacteria. The cementation solution should be injected every three hours until reaching the desired cementation level. The desired cementation level could be determined based on the shear wave velocity measurements and determination of

the CaCO₃ amount (DeJong et al., 2011; Al Qabany et al., 2012). The volume of the injected bacterial culture and cementations solutions are multiple voids' volumes. Usually ranges between one and two pores' volumes (DeJong et al., 2011).

This method is the best preferred as it mainly depends on the injection conditions, which could be controlled easily, such as the flow rate, pressure, and hydraulic gradient. Moreover, this method allows for both vertical and horizontal injections, which helps in the treatment adjustment. The main disadvantage of this method is the uneven distribution of the produced calcite through the soil mass. The primary reason for that is the injected solution's preferred path through the pore space and microbes' filtration through the sand grains. Besides, clogging around the injection ports is considered one of the major problems of this technique. The uneven distribution has been reported by many authors in various configurations starting from the small laboratory scale (Ginn et al., 2001; Cheng & Cord-Ruwisch, 2014; Harkes et al. 2010; Torkzaban et al., 2008; Whiffin et al., 2007) up to the 100 m³ of full-scale soil (van Paassen et al., 2010).

2.2.4.2 Surface Percolation Method

The surface percolation method comprises spraying the bacterial and cementation solutions alternatively to the soil mass's surface, and they would penetrate through the soil pores due to the gravity force (Cheng & Shahin, 2019).

The main advantage of this technique is that it does not require heavy machinery to control the flow as the flow is governed by gravity. The other side is that the flow is mainly governed by gravity, limiting the improvement depth to just the surface layers for the fine-grained soils due to the low infiltration and hydraulic conductivity of these soils. The free drainage in the surface percolation method allows the solution to penetrate through its preferential path where the flow resistance is less than the other paths. That would lead to a higher content of CaCO₃ along this path, which implies a heterogeneous distribution of CaCO₃ through the soil mass (Cheng & Cord-Ruwisch, 2014).

The best application for the surface percolation method is surface ground improvements, such as erosion resistance and dust control (Gomez et al., 2015; Cheng & Cord-Ruwisch, 2014).

2.2.4.3 Premixing Method

The main shortcoming in the two previous methods is the non-homogeneity of the precipitated calcite through the soil mass. The main reason for that is the filtration of the bacterial through the soil pores during the injection or the surface percolation, which leads to less concentration of calcite at far depths (Ginn et al., 2001). To overcome this shortcoming, the premixing method evolves. Mixing the bacteria with the soil until the desired homogeneity, ensures the homogenous calcite precipitation in the soil matrix. Yasuhara et al. (2012) and Zhao et al. (2014) confirmed the precipitated calcite's homogeneity in the soil mass using this method.

Cheng & Shahin, 2016 presented a novel method for premixing the soil with the bacteria and named it the urease active bioslurry approach. In this approach, the bioslurry is mixed with the soil mass instead of the bacterial culture. The bioslurry consists of bacterial culture, urea, and CaCl₂.

This method has been used for organic soil in the current study and showed a significant improvement in the soil properties. The main drawback of this method is that premixing the soil with bioslurry would disturb the original soil structure and add unmeasured stresses to the soil during the mixing process, which would lead to a pseudo-stress development in the soil sample (Cheng & Shahin, 2019).

2.2.5 Factors Affecting the Precipitation of CaCO₃

One of the main factors that affect the strength of the MICP treated soils is the size, shape, and distribution of the precipitated CaCO₃ crystals (Mujah et al., 2017). In this section, the significant factors that affect the precipitation of CaCO₃ crystals during the MICP treatment in sandy soils,

such as temperature, Urease activity, pH level, degree of saturation, and concentration of cementation solutions, would be discussed.

2.2.5.1 Temperature

The temperature affects the bacteria urease activity, the growth of the calcium carbonate crystals, and solubility. On the one hand, the temperature range between 20°C to 50°C affected the amount and size of the formed crystals (Nemati & Voordouw, 2003). On the other hand, if the temperature is more than 50°C, it would cause the microorganisms' death and ceasing the production of CaCO₃ crystals, according to Rebata-Landa (2007).

Cheng & Cord-Ruwisch (2014) examined the precipitated CaCO₃ crystals in sand columns at room temperature (25°C) and at higher temperatures (50°C) using the scanning electron microscope (SEM). They concluded that despite the higher CaCO₃ content at high temperatures, the size of the formed crystals at high temperatures was small and covered the whole surface area of the grains. However, the formed crystals' size at room temperature was big enough and linked the consecutive grains. That is why the measured unconfined compressive strength UCS of the treated samples in the room temperature was larger than those in the higher temperature.

2.2.5.2 Bacterial Concentration/Urease Activity

One of the primary roles of the bacterial cells in the MICP is to work as nucleation sites that catalyze the reaction between the carbonate ions and calcium cations to precipitate the calcium carbonate crystals, which eventually link the adjacent particles together (Hammes & Verstraete, 2002; DeJong et al., 2010). According to Whiffin (2004), urease activity indicates the rate of urea hydrolysis by ureolytic bacteria. Hence, the amount of the added bacteria to the soil mass is of great importance as it affects the hydrolysis rate of urea and availability of nucleation sites for calcite precipitation (Mujah et al., 2017).

The produced carbonate ions through the urea hydrolysis process can either further grow the existing CaCO₃ crystals or nucleate new CaCO₃ crystals based on the availability of nucleation sites (DeJong et al., 2010). Therefore, the bacterial cells' availability throughout the soil matrix is crucial and would affect the produced crystals' size and amount.

2.2.5.3 pH Level

The favourable environment for the precipitation of the $CaCO_3$ is the alkaline environment such that the pH ranges from 6.5 to 9.4 (Ferris et al., 2004). In the MICP, the hydroxyl ions (OH-) are generated during the production of ammonium (NH₄), as presented in Equation [2.2], which increases the alkalinity of the environment and induces calcite precipitation.

Cheng & Cord-Ruwisch (2014) have investigated the effect of the soil's initial pH on the MICP treatment performance. They outlined that the original soil's acidity and alkalinity have a negative impact on the unconfined compressive strength, and the best values for the UCS have been recorded for soils of neutral pH.

2.2.5.4 Degree of Saturation

On the one hand, the initial studies for the effect of degree of saturation on the $CaCO_3$ precipitation carried out by Whiffin et al. (2007), van Paassen (2009), and Al-Thawadi (2013) claimed that the highest unconfined compressive strength had been achieved for samples of 100%-degree saturation.

On the other hand, Cheng & Cord-Ruwisch (2013) investigated the effect of various degrees of saturations of 20, 40, 60, and 100 % on the unconfined compressive strength of MICP treated sand samples. The results showed that the samples of 20 % degree of saturations exhibited the highest unconfined compressive strength at even small content of precipitated CaCO₃. They confirmed their results by performing the SEM analysis for these samples and figured out that the precipitated calcite crystals have occupied good locations and affected the

bridging between the particles for the low degree of saturation. In contrast, the precipitated calcite for the complete saturation samples has occupied random and ineffective locations.

2.2.5.5 Concentration of Cementation Solution

The concentration of the cementation solution usually affects the shape and distribution of the calcite precipitation. The concentration of the cementation solution means the concentration of the various chemicals used in the cementation solutions. Urea and calcium chloride are the two primary components of the cementation solution.

A higher concentration of cementations solutions in the soil mass means higher calcite precipitation opportunities due to the calcium source's availability. However, condensed research has been concluded that the efficiency of the MICP technique is higher in the case of low cementation solution concentrations (Okwadha & Li, 2010; Al Qabany & Soga, 2013; Ng et al., 2014; Cheng & Cord-Ruwisch, 2014). This phenomenon has been investigated through the SEM of the treated sample. It has been justified by the random distribution of the precipitated calcite in the soil mass at higher concentrations of the cementation solutions.

2.2.6 Effect of the MICP Treatment on Properties of Treated Soils

Precipitating the calcite crystals through the soil mass alters the soil properties. This section discusses the change of the soils' geotechnical properties due to the MICP treatment, such as hydraulic conductivity, stiffness, shear strength, unconfined compressive strength, shear wave velocity, and microstructure configuration.

2.2.6.1 Hydraulic Conductivity

One of the MICP stabilization technique's main advantages is that the treated soil could retain sufficient hydraulic conductivity after treatment more than the treated soil using the Ordinary Portland Cement (OPC). The main reason is that the treated samples' pore space using the OPC is filled with hydrates due to the cement hydration reaction, and these hydrates are not soluble in water. That is why they occupy ample space of the soil pores and reduce the hydraulic conductivity. On the other side, the MICP technique does not fill the pore space with the CaCO₃, but the crystals occupy a small volume of the pore space to link and bond the soil particles (Cheng & Cord-Ruwisch, 2013).

Small concentrations of cementation solutions are preferred over the high concentrations when hydraulic conductivity retention is desired (Mujah et al., 2017). Besides, small concentrations of cementation solutions result in more robust samples of uniformly distributed calcite throughout the soil mass. Therefore, treated samples by high concentrations of cementation solutions (0.5 M to 1 M) are subjected to a significant reduction in hydraulic conductivity more than the treated samples by small concentrations of cementation solutions (0.1 M to 0.5 M), according to Ivanov et al. (2010) and AI Qabany & Soga (2013).

It is worth mentioning that even though the MICP could be used to retain a considerable hydraulic conductivity after treatment, it could be used for clogging the soil pores in specific applications as well. Ivanov & Chu (2008) and Chu et al. (2013) presented the concept of bioclogging through the precipitation of large calcite amounts in the soil mass.

2.2.6.2 Stiffness

The soil stiffness is known by the soil's elastic modulus (E), which is the ratio between the soil's stress and strain. Soil stiffness is mainly governed by the soil grains' bonding strength (Mujah et al., 2017). Stiffness of MICP treated soils increase with the increase of the amount of the precipitated calcite, as concluded by Montoya & DeJong (2015) when they studied the stress-strain relation of the MICP at different cementation level. This finding is similar to what has been reported in the literature for stabilized soils by synthetic materials such as Portland cement, Gypsum, and sodium silicate (Santamarina, 2001; Haeri et al., 2006; Amini & Hamidi, 2014).

One of the advantages of the MICP treated soil compared to the other types of geomaterials such as concrete, gravel, and rock is the flexibility induced in the MICP treated soils (Cheng & Cord-Ruwisch, 2013). The soil's flexibility means the soil mass's ability to maintain significant residual strength, which gives extra time for evacuations for areas prone to earthquakes.

2.2.6.3 Shear Strength

Precipitating the calcite crystals in the soil mass would increase the bonding between the consecutive particles, which would increase the overall shear strength of the soil mass. The shear strength of the soil mass is the magnitude of the shear stress that the soil mass can sustain and rely on the soil's inherent shear strength parameters, such as the cohesion (c) and internal angle of shearing resistance (φ).

Initially, Chou et al. (2011) reported that the primary source of the increased shear strength of the MICP treated soils is the increase in the internal angle of shearing resistance for all the treated samples by three conditions of the *Sporosarcina pasteurii* bacterial strain (growing, resting and dead cells). Besides, he reported that the induced peak shear strength of the MICP treated samples was higher than the untreated samples.

Afterward, Duraisamy & Airey (2012) investigated the effect of the MICP on the induced shear strength of liquefiable sandy soils at different cementations levels, and they concluded that the primary source of enhancement was the increase in the soil cohesion due to the precipitation of the calcite crystals between the soil particles. No significant contribution for the internal angle of shearing resistance was observed.

Cheng & Cord-Ruwisch (2013) checked MICP treatments' effect on the soil's cohesion and friction angle under various saturation degrees. They concluded that the cohesion contribution is improved more than the friction at a lower degree of saturation. However, at higher CaCO₃ content, both the cohesion and friction improved regardless of the degree of saturation.

2.2.6.4 Unconfined Compressive Strength UCS

In literature, UCS is the most commonly used test for the MICP technique as it allows for testing many samples in less time comparing to other shear strength tests as reported by many researchers (Whiffin et al., 2007; Harkes et al., 2010; Cheng & Cord-Ruwisch, 2013; Zhao et al., 2014; Ivanov et al., 2015).

Cheng & Cord-Ruwisch (2014) and van Paassen et al. (2010) revealed an exponential relation between the UCS and amount of the precipitated calcite, which indicated that at the same amount of CaCO₃, the UCS value might be different depending on the effectiveness of the calcite precipitation in the pore space. The reported values for the UCS of MICP treated samples ranged between 150 kPa and 34 MPa based on the different cementation levels, according to Whiffin (2004).

The UCS test does not represent the actual field conditions, especially in far depths, where the confining stress increases. Therefore, direct, simple, and triaxial shear tests are recommended for the MICP treated samples to ensure the stress state during the laboratory test is similar to the stress state in the field (Mujah et al., 2017).

2.2.6.5 Shear Wave Velocity

Shear wave is a mode of wave propagation at which the direction of the particle motion is perpendicular to the direction of propagation, which requires the material to have a small-strain shear stiffness (Santamarina et al., 2001). Besides, the shear wave velocity does not propagate through fluids. Therefore, this technique is imperative in the MICP treated sample as it could capture only the precipitated calcite that helps in binding the soil particles, and it is not affected by the calcite suspended in the pore fluid (DeJong et al., 2010).

The shear wave velocity measurements' main advantage is that it is a non-destructive technique and will not alter the measured soil properties. It also enables measuring the soil

strength with time in the real-time domain, which helps in upscaling the MICP technique to the field applications and measuring the improvement change with time in the field over a long period.

2.2.6.6 Microstructure analysis

The precipitation of the calcite crystals within the soil pores affects the soil behaviour. Microstructure analysis using the scanning electron microscopy technique SEM helps researchers to have a closer and more detailed look at how the calcite precipitation affects the soil behaviour. Many researchers concluded that not all the precipitated calcite improve the soil strength but only those who precipitated in the particle to particle contact and formed a link or bridge between the consecutive particles (DeJong et al., 2010; Cheng & Cord-Ruwisch, 2013; Akiyama & Kawasaki, 2012; Tobler et al., 2012; Park et al., 2014).

2.2.7 Applications of the MICP Technique

The MICP stabilization technique is still under development to optimize the various components and processes to be upscaled and used for various field applications. However, many researchers envisioned the proposed applications of this technique in the field applications. The various applications include self-healing of soils, slope stabilization, settlement reduction, erosion control, and liquefaction prevention. The summary of the envisioned application of the MICP technique is summarized in **Table (2.1)** after Mujah et al. (2017). Table 2.1 Envisioned applications for the MICP (after Mujah et al. 2017). Reprinted with

Envisioned applications	Possible mechanism	References
Self-healing of soils	A portion of bio-cementation bonds degrade when loaded beyond its yield strength. The degraded MICP bonds can be healed by re-initiating the bio- geochemical process, returning the cemented sand properties to pre-shearing levels	Harbottle et al. (2014); Montoya and Dejong (2013)
Slope stabilization	The bio-cemented bonds help to strengthen the failure plane surface to provide additional stability needed to prevent slope failures	DeJong et al. (2010, 2013)
Settlement reduction	The bearing capacity of bio-cemented soils is increased; hence, settlement of foundation is reduced	DeJong et al. (2010, 2011); van Paassen et al. (2010)
Erosion control	MICP increases the bio-cemented soil resistance to the erosive forces of water flow along the sea shores and river banks	CheNg et al. (2014); DeJong et al. (2006)
Liquefaction prevention	Similar to the concept of self-healing, the post-shearing loads could re-initiate the MICP process; hence, preventing further liquefaction damages	Montoya and DeJong (2015); Montoya et al. (2013)

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2.2.8 Advantages of the MICP Technique

The MICP stabilization technique's primary advantage is that it utilizes natural materials and processes to enhance the soil properties. That is why it is considered an environmentally-friendly compared to other techniques. In this section, three significant advantages of the MICP would be discussed: sustainability promotion, system reliability, and cost-effectiveness.

2.2.8.1 Sustainability Promotion

The MICP utilizes natural materials such as microorganisms to bond the soil particles together. Hence, if the system is optimized and utilized through the big-budgets soil stabilization projects, it would be considered a green technology for the future. Besides, the formed crystals permanently alter the soil subsurface conditions.

The MICP utilizes calcium chloride as a source for the calcium cations to help in the calcite crystals' precipitation. However, another green alternative could be used as a calcium source, which is the seawater, as proposed by Cheng & Cord-Ruwisch (2014).

Part of the optimization process for this system that needs to be considered is utilizing the ammonium by-product of this system, which might be harmful to the groundwater, by a good collection of the ammonium and refeeding them to the surroundings as fertilizer. (DeJong et al., 2011; Achal & Mukherjee, 2015).

2.2.8.2 System reliability

The system flexibility and reliability are triggering this system's preference over other systems. According to the soil type, the MICP can be optimized and adjusted both mechanically and biologically to achieve the requested level of cementation. The system, as well, provides flexibility for the timeframe in any project as it mainly depends on the calcite precipitation process, which is fast and does not take long time (Mujah et al., 2017)

2.2.8.3 Cost-effectiveness

The comparison between the cost of applying the MICP and chemical grouting techniques has been presented by Ivanov & Chu (2008). The study revealed that the MICP technique is a little bit more expensive than the chemical grouting, such that the cost of the chemical grouting per cubic meter of soils ranges between \$2 to \$7 but for the MICP ranges between \$0.5 to \$9 based on the stabilization level. Whiffin et al. (2007), Ismail et al. (2002), and AI-Thawadi (2013) stated that the MICP is cost-saving in the long-term as the same bacteria could be used multiple times with just adding new cementation solutions.

To reduce the cost of the MICP, three options might be implemented (1) reproducing the bacteria continuously on-site as proposed by Cheng & Cord-Ruwisch (2013), (2) using the seawater as the calcium source instead of the calcium chloride (Cheng & Cord-Ruwisch, 2014), and (3) stimulating the native microorganisms in the soil mass (Gomez et al., 2018).

2.2.9 Limitations of MICP technique

Despite the various advantages of the MICP technique for soil stabilization, this technique has some limitations. The first limitation is the particle size, such that this technique is valid for large particles of sizes between 0.5 mm to 3 mm, which means that this system is just applicable for sandy soils. However, a new technique called electro-biogrouting has been proposed by Keykha

et al. (2014) for the improvement of fine-grained soils. In the current research, the used soil is mainly organic with a high initial void ratio, making it a suitable environment for the calcite's precipitation.

The second limitation is the spatial distribution of the calcite crystals through the soil pores, which significantly affects the MICP treated soils' performance. This needs to be investigated using further large-scale improvements other than those carried out by van Paassen et al. (2010).

The third limitation is the production of ammonium as by-products, which might be harmful to the groundwater system, and some researchers proposed treating this by-product before discharging or back-feeding the ammonium to be used as a fertilizer for the surrounding plants.

Chapter 3: Development of New Correlations for Organic Soils Properties

3.1 Background

Water and organic contents are simple and easy parameters to measure for organic soils. Therefore, there are many correlations in the literature that correlate the other index and compressibility properties of organic soils to their water and organic contents. In this chapter, some new correlations are proposed for organic soils and compared to the existing ones in the literature. The new correlations cover a wide range of water and organic contents that have not been covered before. Moreover, the proposed correlations relate the index and compressibility parameters of organic soils to the water and organic contents at the same time.

3.2 Methodology for Correlations Development

Organic soils are considered one of the most problematic soils due to their high compressibility and low shear strength properties. Characterizing organic soils based on their simple index properties is beneficial for the preliminary design stages of construction projects. There are three main index parameters used to assess organic soil properties: water content, organic content, and fibre content. Their relatively high water content distinguishes organic soils. The organic content includes carbonaceous and combustible components. The fibre content accounts for fibres' presence in organic soils based on their botanical composition and decomposition degree.

The data available in the literature regarding organic soil parameters were collected and analyzed to obtain new correlations between the various organic soil parameters and simple index parameters (water and organic contents). The available correlations found in the literature depend on relating a specific parameter with either the organic content or water content separately. However, the organic and water contents are related. Hence, this study's proposed correlations aim to connect the particular soil parameter with both the water and organic contents, using the same equation. Unfortunately, there is not much literature about the soil texture or fibre content

and their relation with other parameters. Hence, all the proposed correlations in this study are not considering the fibre content or soil fabric.

The following steps show how the data has been gathered and analyzed:

- (1) Going through the available and authentic research for the organic soil and extract the values of the parameters under consideration. These parameters are the water content, organic content, fibre content (if any), void ratio, bulk density, specific gravity, compression index, and shear strength parameters (if any).
- (2) Checking the available data for the parameters under consideration (void ratio, specific gravity, bulk density, compression index) and excluding any outliers.
- (3) Drawing the relation between the particular parameter and water content to check if it is similar to the literature's available correlations. Moreover, to add a range for the collected data, a 95 % confidence interval has been added around the proposed trendline.
- (4) Repeating step 3 but using the organic content instead of the water content to check the particular parameter's change due to the organic content change.
- (5) List all the soils with reported values for the water content, organic content, and parameter under consideration. The points that are missing one of these three parameters have not been used in the analysis.
- (6) Using Datafit software (statistical software), the correlations between the parameter under consideration (dependent variable) and water and organic contents (independent variables) have been constructed as a 3D-surface relationship.

Despite the availability of the correlations for the index and compressibility properties of organic soils using the measured water content, the new proposed correlations provide a higher level of confidence on the interpreted parameter as the equations rely on two parameters (water and organic content) not only one parameter as previously used in the literature. One more advantage of the proposed correlations is that they cover the whole range of the organic content up to 100 %.

It is worth mentioning that the shear strength data are not enough to statistically interpret new correlations. However, the trendlines of the effect of both water and organic contents on the shear strength parameters have been investigated to add more insights into the soil behaviour.

3.3 Developed Correlations

In this section, the developed correlations for the index properties and compression index of organic soils based on the merely measured water and organic contents would be presented first. The main objective of presenting these correlations is to help design engineers in the preliminary stage of the project to interpret the index properties and compression index using the water and organic contents, which are easy and inexpensive to measure. The index properties comprise the void ratio, bulk density, and specific gravity. The compressibility properties include the compression index. The changes of the shear strength parameters with the water and organic contents are presented, as well.

It is worth mentioning that the proposed correlations are not adequate for small water and organic contents (i.e. for water and organic contents less than 5%).

3.3.1 Void Ratio (e)

In the phase diagram relations, the void ratio and water content are related as a function of the specific gravity (G_s) and degree of saturation (S) such that $e = w_c * G_s / S$. For organic soils, Tessier (1966) and Lea & Brawner (1963) proposed a correlation for the void ratio as a function of water content without measuring the values of the specific gravity or degree of saturation.

Figure (3.1a) depicts the relationship between the void ratio and water content based on the literature's collected data. Besides, the figure shows the 95% confidence interval around the trendline as a shaded area. The 95% confidence interval limits are calculated by adding and subtracting twice the standard deviation. The correlation between the void ratio and water content is presented in **Equation [3.1].** Correlations by Tessier (1966), Lea & Brawner (1963), and Den

Haan (1997) are presented in the same graph to verify the compatibility of the proposed 95% confidence interval. Tessier (1966) and Den Haan (1997) proposed Equations are located in the shaded zone of the 95% confidence interval. However, part of the results of Lea & Brawner (1963) for water contents higher than 500% is located outside it.

$$e = 0.0138 * w_c + 1.1256$$
 : $R^2 = 0.9734$ [3.1]

The value of w_c is in percentage (%).

McVay & Nugyen (2004) correlated the organic content and void ratio by a linear equation. This study's collected data was used to obtain a correlation between the void ratio and organic content. The proposed correlation **[Equation 3.2]**, the 95% confidence interval and McVay & Nugyen (2004) equation are presented in **Figure (3.1b)**. The figure reveals a reasonable agreement regarding the increasing trend between the proposed equation and McVay & Nugyen (2004) equation.

$$e = 1.6355 * e^{0.0187 * OC}$$
 : $R^2 = 0.7291$ [3.2]

The value of OC is in percentage (%).

The organic and water contents are the most straightforward index properties for measurement in organic soils. Equations [3.1] and [3.2] show that the void ratio correlates with the organic and water contents. It is beneficial to obtain one equation that correlates the void ratio with both the water and organic contents simultaneously, as presented in **Equation [3.3]** and **Figure (3.1c)**.

$$e = 1.2539 - 3.255 / OC + 0.01353 * w_c : R^2 = 0.9749$$
 [3.3]

The value of w_c and OC are in percentages (%).



Figure 3.1 Void ratio correlations: (a) void ratio vs. water content, (b) void ratio vs. organic content, and (c) void ratio vs. both water and organic contents. *Reprinted with permission of Springer Nature*.

3.3.2 Bulk Density (ρ)

Figure (3.2a) and **Equation [3.4]** show the correlation between the bulk density and water content. The collected data from literature are used to obtain the trendline of the correlation. Afterward, a range for the 95% confidence interval is obtained and presented as shaded area.

$$\rho = 2.2951 * w_c^{-0.123}$$
 : $R^2 = 0.6446$ [3.4]

The value of w_c is in percentage (%).

The deduced equation's compatibility with the available equations in literature has been checked by adding Hobbs's (1986) data and Moore's (1962) equation to the same figure. The results are located in the 95% confidence interval range. The figure shows that the bulk density tends to settle around 1 gm/cm³ for water contents higher than 500% (Hobbs, 1986 and MacFarlane, 1969).

Figure (3.2b) and Equation [3.5] present the correlation between the bulk density and organic content. The same concept of the 95% confidence interval used in Figure (3.2a) is used.

$$\rho = 2.3183 * OC^{-0.182}$$
 : $R^2 = 0.8354$ [3.5]

The value of OC is in percentage (%).

The 95% confidence interval range is compared with the Den Hann & El Amir (1994) equation for Dutch peat and shows a good agreement with the deduced equation at high organic content as Dutch peats are characterized by their high organic content. On the other hand, the McVay & Nugyen (2004) equation almost coincides with the proposed equation. The different correlations reveal that the bulk density tends to be close to 1 gm/cm³ at high organic content (peaty soils). As long as the bulk density is correlated to the water and organic contents, then it would be more reasonable to correlate the bulk density with both the organic and water contents in the same equation or graph, as shown in **Figure (3.2c)** and stated in **Equation [3.6]**.

$$\rho$$
 = 1.498 - 0.1135 * ln (OC) + 21.6716 / w_c : R² =0.8907 [3.6]

The value of w_c and OC are in percentages (%).

The bulk density could be calculated by Equations [3.4], [3.5], and [3.6] during the preliminary stages of the project using the simple index properties: the water and organic contents.

3.3.3 Specific Gravity (G_s)

MacFarlane (1969) and MacFarlane and Rutka (1962) carried out previous attempts to correlate the specific gravity with water content. A new correlation between the specific gravity and water content is proposed in **Equation [3.7]**. The 95% confidence interval for the proposed equation is shown as shaded area in **Figure (3.3a)**. Equations from MacFarlane (1969) and MacFarlane and Rutka (1962) are added to the same figure to verify the 95% concept. The two equations are located in the shaded area of the 95% confidence level. Besides, the figure reveals that the specific gravity tends to be constant around 1.5 at high water contents (higher than 500%), similar to the bulk density that tends to be constant at the same water content.

$$G_s = -0.342 * \ln (w_c) + 3.7093$$
 : $R^2 = 0.7006$ [3.7]

The value of w_c is in percentage (%).

The specific gravity of organic soils is correlated to the organic content. **Figure (3.3b)** illustrates the proposed correlation between the specific gravity and organic content **[Equation 3.8]**. The 95% confidence interval around the trendline, Kazemian et al. (2009) equation, and extreme limits of Landva & La Rochelle (1983) are presented in the same figure. Comparing the collected data from literature with the extreme limits of Landva & La Rochelle (1983) yields many points outside these limits. Hence, the 95% confidence interval shaded area is a more suitable representation for most of the collected data. The proposed correlation and available correlations reveal that the specific gravity tends to be constant around 1.5 at high organic contents.


Figure 3.2 Bulk density correlations: **(a)** bulk density vs. water content, **(b)** bulk density vs. organic content, and **(c)** bulk density vs. both water and organic contents. *Reprinted with permission of Springer Nature*.



Figure 3.3 Specific gravity correlations: (a) specific gravity vs. water content, (b) specific gravity vs. organic content, and (c) specific gravity vs. both water and organic contents. *Reprinted with permission of Springer Nature*.

$G_s = -0.422 * \ln (OC) + 3.3875$: $R^2 = 0.8046$ [3.8]

The value of OC is in percentage (%).

Equations [3.7] and [3.8] reveal that organic soils' specific gravity is correlated to both the water and organic contents. Hence, one correlation that combines both the water and organic contents in the same equation, along with the specific gravity, is presented in Figure (3.3c) and Equation [3.9].

$$G_s = 2.8631 * 0.9947 \circ c * w_c^{-0.03512}$$
 : $R^2 = 0.8243$ [3.9]
The value of w_c and OC are in percentages (%).

3.3.4 Compression Index (C_c)

The compression index C_c is correlated to clayey soil's liquid limit (Terzaghi & Peck, 1967). It is difficult to determine the liquid limit of organic soils because of the presence of fibres and amount of intraparticle water. However, there are some correlations available in the literature for the compression index C_c as a function of the liquid limit w_L (Hobbs, 1986; Farrell et al., 1994; Al-Raziqi et al., 2003). Most of the correlations available to determine the compression index C_c for organic soils depend on the natural water content and initial void ratio. (MacFarlane, 1969; Kogure & Ohira, 1977). The correlation between the compression index and all related parameters such as water content w_c and organic content OC might be beneficial.

Figure (3.4a) presents the proposed correlation between the compression index C_c and water content w_c . The correlation is stated in **Equation [3.10]**. The trend shows an increase in the compression index C_c , while the water content w_c increases. The increasing compression could be explained by the large amount of pore water pressure dissipation at high water contents. The figure shows the 95% confidence interval concept, which is shaded by the blue area. The area covers most of the data collected from the literature. The 95% confidence interval concept is verified using the previous correlations of MacFarlane (1969) and Kogure & Ohira (1977). MacFarlane (1969) presented the boundaries of the compression index and water content

relation. However, some of the data collected from recent studies are located outside MacFarlane's (1969) boundaries, but they are located inside the shaded area of the proposed 95% confidence level.

$$C_c = 0.0083 * w_c$$
 : $R^2 = 0.906$ [3.10]

The value of w_c is in percentage (%).

It is not common to correlate the compression index C_c with the organic content OC. Instead, it is usually correlated with the water content directly. In this study, the compression index C_c is correlated to the organic content OC as shown in **Figure (3.4b)**. The figure shows an increasing trend for the compression index C_c while increasing the organic content as the organic component is considered compressible. The correlation is stated in **Equation [3.11]**.

$$C_c = 0.0798 * OC - 1.0704$$
 : $R^2 = 0.5423$ [3.11]

The value of OC is in percentage (%).

Figures (3.4a) and (3.4b) show that the compression index is a function of both the water and organic contents. Figure (3.4c) presents the proposed 3D graph that shows the compression index C_c as a function of the water and organic contents at the same time. The correlation is stated in Equation [3.12] with a least-square error R^2 value around 0.9.

$$C_c = 0.5669 - 0.023 * OC + 0.0097 * w_c$$
 : $R^2 = 0.8939$ [3.12]

The values of w_c and OC are in percentages (%).



Figure 3.4 Compression index C_c correlations: **(a)** compression index vs. water content, **(b)** compression index vs. organic content, and **(c)** compression index vs. both water and organic contents. *Reprinted with permission of Springer Nature*.

3.3.5 Undrained Shear Strength Parameter Su

The undrained shear strength of organic soils is better determined by in-situ testing. MacFarlane (1969) correlated the measured undrained shear strength parameter S_u using the field vane test with the water content. There are no trials that correlated the undrained shear strength parameter S_u with the organic content. The data available in the literature for the undrained shear strength and organic content are not adequate for a good correlation, as the data only covers the high organic content.

Figure (3.5) shows the water and high organic contents' effect on the organic soil's undrained shear strength. The figure demonstrates a decreasing trend for the undrained shear strength parameter while increasing the water content. The same trend is observed by many researchers (MacFarlane, 1969; O'Kelly, 2017). In contrast, the undrained shear strength slightly increases while increasing the organic content. The deduced correlations shown in the figure exhibit high scatter and a small least square error value. The figure reveals that the organic soils' undrained shear strength parameter is small and ranges between 2 kPa to 15 kPa, which is much smaller than the soft clay's undrained shear strength (20 kPa to 40 kPa). This small value of the undrained shear strength is the main reason for the lateral spreading failure mechanism for embankments rested on organic soils (Ajlouni, 2000).



Figure 3.5 Effect of water and organic contents on the organic soils' undrained shear strength parameter. *Reprinted with permission of Springer Nature*.

3.3.6 Effective Cohesion c'

Their small cohesion distinguishes organic soils. The triaxial compression and direct shear tests are the two most commonly used laboratory tests for determining the effective shear strength parameters. The Consolidated Undrained CU test is the most suitable test for organic soil. The organic soil's behaviour starts as drained behaviour at small stresses due to the high initial void ratio and hydraulic conductivity, but while the load increases and the soil consolidates, the behaviour rapidly transfers to the undrained behaviour (Dhowian & Edil, 1980; Edil et al., 1991, 1994). This section presents the correlations between the effective cohesion c' and water and organic contents.

The effect of both the water and organic contents on the effective cohesion of organic soils is presented in **Figure (3.6)**. There is not much data in the literature for the effective cohesion at small organic content. The effective cohesion generally decreases while the water content increases as the water surrounds the particles and decreases the particles' cohesion. Based on the available data for the effective cohesion at the high organic content, the figure demonstrates that the effective cohesion increases while the organic content increases. The figure shows that organic soil's effective cohesion generally decreases as more water is added and increases as more organic matters are added. The presence of fibres in the organic soils would affect both the undrained shear strength parameter and effective cohesion. Hence, the fibre content and fibre texture would significantly affect the soil's cohesion.

3.3.7 Effective Angle of Shearing Resistance φ'

The two main shear strength parameters are the effective cohesion c' and effective angle of shearing resistance φ '. The primary source of the induced φ ' in organic soils is the fibre interactions among each other or with minerals, which explains why the φ ' is negligible at minor strains. There are many trials to correlate the effective angle of shearing resistance φ ' with water

content, but the range is extensive. Landva & Pheeney (1980) stated that there is no reliable empirical relationship that has been developed yet for organic soils' shear strength.

Trendlines for the effect of the water and organic contents on the effective angle of internal friction are presented in **Figure (3.7)**. The figure reveals that the effective angle of shearing resistance increases during both the water and organic contents increase. This could be attributed to the high fibre content at high water content. The data available in the literature are not enough to establish a correlation between the water or organic contents and effective angle of shearing resistance.



Figure 3.6 Effect of water and organic contents on the effective cohesion of organic soils. *Reprinted with permission of Springer Nature*.



Figure 3.7 Effect of water and organic contents on the effective angle of shearing resistance of organic soils. *Reprinted with permission of Springer Nature*.

3.4 Summary

This chapter presented some new correlations for some of the organic soil parameters as a function of the easily measured properties: water and organic contents.

The proposed correlations showed that the void ratio and liquid limit were better estimated based on the water content. The unit weight and the specific gravity showed a better correlation with the organic content. Whenever the water content increased, more than 500 %, the bulk density tended to settle around one, and the specific gravity around 1.5. The correlations revealed that the proposed minimum limit Landva & La Rochelle (1983), for the determination of the specific gravity G_s as a function of the water content, was not in a good agreement with the 95% confidence interval proposed in this study. The 95% confidence interval covered many data points that the minimum limit did not cover.

Furthermore, the study revealed that the lower bound proposed by Macfarlane (1969) for the determination of the compression index C_c as a function of the water content was not applicable for the full range of the organic content. Besides, a high scatter of the compression index was noticed when the organic content was used for the correlations. Therefore, it is recommended for the initial tentative estimate of the compression index C_c to use the proposed equation, in this study, as a function of the water content instead.

Organic soils are distinguished by their low shear strength parameters for both amorphous peat and fibrous peat at small strains. It was difficult to correlate the shear strength parameters as a function of water or organic contents without considering the presence of fibres and their textures. However, the undrained shear strength parameter S_u and the effective cohesion c' showed a decreasing trend while the water content increased. Generally, organic soils' cohesion was small and ranged between 2 to 14 kPa for both the drained and undrained conditions.

Chapter 4: Laboratory and Field Testing of Muskeg Soil Deposit

This chapter presents the findings of extensive field and laboratory investigations for a Muskeg soil deposit. The soil is retrieved from Bolivar Park, Surrey, British Columbia, Canada.

4.1 Background

Organic soils, such as peat and Muskeg, are considered problematic materials from a geotechnical engineering perspective, mainly due to their low stiffness and strength properties. These unfavourable conditions could lead to excessive settlements and/or unacceptable failures in our built environments. Many past failures observed in organic soils, such as those in dykes (Bezuijen et al., 2005; O'Kelly, 2008); slopes (Long & Jennings, 2006; Boylan et al., 2008; Long et al., 2011); and embankments (Den Haan & Feddema, 2013) serve testimony to the difficulties and complexities related to this engineering problem.

It is also relevant to note that a significant portion of Canada's oil and gas pipelines are located in areas underlain by Muskeg soils. Muskeg is the term used to describe the terrain in which large amounts of organic matter is present in various degrees of decomposition (Lee et al., 2015), and such terrain covers over 1.5 million square kilometres of the Canadian landscape (MacFarlane 1969; Tarnocai, 2006; Xu et al., 2018). With increased attention to assure the structural integrity and safety of buried pipeline systems with a collective aim "towards zero incidents," the analysis of soil-pipe interaction in soft and weak Muskeg (organic soil) presents significant challenges to the pipeline design engineers. For example, the guidelines for the design of the soil-pipe interaction between the various pipeline materials and surrounding soils are well-established for conventional soil types such as sandy and clayey soils (ALA, 2001; PRCI, 2009); however, the soil-pipe interaction in organic soils are not well defined. This is primarily due to the lack of understanding of the mechanical behaviour of organic soils. Therefore, pipeline design in soft organic soils is often conducted with significant conservatism.

Organic soils are distinguished by their high initial water content, high void ratio, high compressibility, high hydraulic conductivity, high angle of internal friction, low bulk density, and low specific gravity (Adams, 1965; MacFarlane, 1969; Dhowian & Edil, 1980; Landva, 1980; Lefebvre et al., 1984; Mesri & Ajlouni, 2007). In essence, they exhibit different mechanical behaviour than the typical behaviour of inorganic soils (Edil & Wang, 2000). The natural water content has been noted as the index that mainly reflects/correlates with Muskeg soils' engineering properties (Lea & Brawner, 1963; MacFarlane, 1969; Yamaguchi et al., 1985).

The above background highlights a strong need to advance the understanding of organic soils' mechanical behaviour in the design of buildings/lifeline infrastructure. The requirement to characterize shear strength and deformation parameters as input for soil-pipe interaction analysis of buried pipelines in Muskeg soils is also a specific consideration. With this impetus, systematic geotechnical field and laboratory, experimental research programs were undertaken to obtain stiffness and strength characteristics of organic soils.

The field-testing program comprised: seismic cone penetration testing (SCPTu) with shear wave velocity (V_s) measurements, full-flow ball penetration testing (BPT), electronic vane shear testing (eVST), along with fixed-piston tube sampling that was undertaken in drill holes put down at the site underlain by organic soil. Laboratory testing included: index property testing (i.e., water content, organic content, fibre content, specific gravity, wet and dry densities, degree of humification and liquid limit), one-dimensional consolidation testing, and direct shear testing.

Through the use of a range of experimental tools at the same site, this investigation provided a unique opportunity to generate a comprehensive experimental data set, make logical and systematic comparisons between the results from field and laboratory geotechnical testing, and in turn, provide input parameters for geotechnical design of infrastructure, including buried pipelines, in organic soils.

This chapter presents the findings from this work, with the information provided in three parts as follows. The first part describes the field's factual results and laboratory investigations and a brief description of the experimental methods. In the second part, the factual results from both the field and laboratory tests and analyses and interpretation of these data sets would be presented. For the third part, specific comparisons made on parameters derived from both the field and laboratory tests are summarized.

4.2 Materials and Methods

4.2.1 Field Investigations

Some well-established geotechnical field investigation techniques were used to determine the soil and groundwater conditions at a specific site underlain by organic soil. The site was located in Bolivar Park, City of Surrey, British Columbia, Canada, as shown in **Figure (4.1)**. All field tests were conducted by ConeTec Investigations Ltd. of Richmond, British Columbia, Canada, under the direction and supervision of UBC research team members over four discrete days between November 2017 and January 2018. The plan view of the test holes, along with a schematic crosssection showing target testing depths for the field tests, are shown in **Figures (4.2a) and (4.2b)**, respectively. The work was undertaken in two groups of test holes denoted as Group 18-01 and Group 18-02, with the two groups located in plan areas that are horizontally 4.5 m apart from each other. As noted in **Figure (4.2)**, the following tests were conducted: two SCPTu tests (one in each group), two BPT tests (one in each group), and six eVST tests (three in each group). Besides, auger sampling was performed to collect disturbed grab-soil samples, including relatively undisturbed samples obtained using thin-walled, sharpened-edge, no-inside-clearance, stainless-steel tubes for visual inspection and laboratory testing as appropriate.

4.2.1.1 Seismic Cone Penetration Test (SCPTu)

Seismic cone penetration tests (SCPTu), following the ASTM-D5778 (2012) Standard, were conducted using an integrated electronic piezocone penetrometer and data acquisition system manufactured by Adara Systems Ltd. of Richmond, British Columbia, Canada. The piezocone penetrometer used in the testing had the following specifications: the cone cross-section area = 15 cm^2 ; cone apex angle = 60° ; sleeve area = 150 cm^2 ; and pore water pressure measurement (with the filter located behind) with a net end area ratio of 0.8. Two SCPTu tests were performed at the locations shown in **Figure (4.2a)**, with the standard measurements of the cone tip resistance (q₁) corrected as per Robertson (1990; 2009), sleeve friction (f_s), pore water pressure (u₂) behind the cone tip during the cone penetration. Seismic shear wave velocity (V_s) measurements were performed up to 7 m and 12 m depths during the penetration tests SCPTu-18-01 and SCPTu-18-02, respectively.



Map data ©2021 Google 2 km **⊫____**

Figure 4.1 Location of Bolivar Park site in Surrey, British Columbia, Canada From Google Maps, by Google (https://www.google.ca/maps/place/Bolivar+Park/@49.2109297, 122.8530789,17z)





4.2.1.2 Full flow (Ball) Penetration Testing (BPT)

The ball penetration test is similar to the cone penetration test except that a sphere replaces the bottom cone with a bigger volume and footprint. The BPT has been designed mainly for use in very weak and soft soils that would essentially "flow" around the ball as fluid while mobilizing a relatively large soil volume during the ball penetration (i.e., "full flow" penetrometer); the method is used widely in characterizing offshore and oil-sand tailings soil deposits (DeJong et al., 2011).

It was considered that BPT would be applicable in the present research on organic soils, which is a very weak soil with high initial water content. The device used herein has a ball diameter of 13.8 cm, leading to a plan footprint of 150 cm². Two BPT tests, named BPT-18-01 and BPT-18-02, were performed to a depth of about 6.25 m below the ground surface (see Figure 4.2), providing essentially continuous recording of the ball penetration resistance (q_b) – i.e., average vertical bearing pressure experienced by the ball during penetration. The sandy soil fill material present within the upper 2.5 m below the ground surface at the side had to be drilled out to avoid encountering large resistance forces that may develop due to ball footprint during BPT penetration.

4.2.1.3 Electronic Vane Shear Testing (eVST)

An electronic field shear vane system manufactured by ConeTec Investigations Ltd. of Richmond, British Columbia, Canada, was used to perform in situ shear vane tests. The device consisted of a four-blade, stainless steel, double-tapered vane (diameter D = 75 mm; height H = 150 mm; top and bottom taper angles of 45°) attached to a steel rod. The procedure involved removing the soil to a level of about 0.3 m above the depth of interest using drilling, then pushing the vane through the bottom of the test hole to the target depth, and then conducting the test by rotating the vane at a pre-specified rate to measure the torque. The measured torque allows determining the peak undrained shear strength S_{u-peak} , and then the remoulded shear strength $S_{u-remoulded}$, after further rotating the vane for additional ten revolutions. The test is conducted following the ASTM-D2573 (2015).

Six eVSTs were conducted in two test holes, eVST-18-01 and eVST-18-02, with three eVSTs in each drill hole at testing depths of 3.5 m, 5 m, and 6.5 m below the ground surface (see Figure 4.2). The peak and remoulded values of S_u were measured at each of the selected depths.

4.2.2 Laboratory Testing

Laboratory tests were conducted on samples retrieved from the following depth zones below the ground surface at the site: Zone A - 2 m to 4 m depth; Zone B - 4 m to 5.5 m depth; and Zone C - 5.5 m to 7 m depth (denoted by their average as 3 m, 4.75 m, and 6.25 m depth zones, respectively). The soils retrieved from the three thin-walled tube samples were used to investigate the relatively undisturbed soils' behaviour.

4.2.2.1 Index Property Testing

The remoulded samples retrieved from different depth zones were tested to determine a range of index properties as follows: (i) water content, ash content and organic content tests (ASTM-

D2974, 2014); (ii) fibre content test (ASTM-D1997, 2013); (iii) bulk and dry densities test using the wax method (ASTM-D4531, 2015); (iv) specific gravity test using the water pycnometer method (ASTM-D854, 2014); (v) degree of humification test (ASTM-D5715, 2014); and (vi) liquid limit test. Two test methods were used for the determination of the liquid limit (w_L), the conventional Casagrande method (ASTM-D4318, 2017) and fall cone penetrometer method (as per British standards BS1377 - Part 2, 1990 requirements). Since the determination of the w_L using the conventional Casagrande method posed several difficulties due to the presence of fibres in organic soils, the fall cone penetrometer method provided an alternate/supplementary measure of w_L.

4.2.2.2 One-dimensional consolidation Testing

Standard Incremental Loading (IL) consolidation testing was undertaken according to the ASTM-D2435/D2435M (2020) standards using an automated oedometer system with a specimen diameter of 63 mm. The automated system enabled recording deformation and load measurements in 5-second intervals, allowing more accurate definition of rapid primary consolidation followed by secondary compression in organic soils. Three relatively undisturbed samples retrieved from depths of 3 m, 5 m, and 6 m were tested for comparison with counterpart results from remoulded samples obtained from Depth Zones A, B, and C, respectively. The remoulded specimens were prepared by determining the soil's requested weight to fill a known volume of the consolidation ring to fulfill the measured field density.

The consolidation testing was repeated for samples from each depth at least three times as a way of checking repeatability, with their initial water contents kept at almost a constant value for a given depth. The samples were initially loaded up to effective vertical stress of 150 kPa, then unloaded to a stress of 25 kPa, followed by reloading until a final vertical effective stress of 200 or 250 kPa was reached.

4.2.2.3 Shear Strength Properties

The undrained shear strength parameters of remoulded organic soil were assessed using laboratory unconfined compressive strength (UCS) testing, further supplemented by laboratory fall cone penetrometer testing. The UCS tests were conducted (using 38-mm diameter and 76-mm height specimens) as per the ASTM-D2166/D2166M (2016) standard using an axial strain rate of 1% of the sample height per minute. According to the British standards BS1377 - Part 2 (1990), the laboratory cone penetrometer method was performed. The approach proposed by Tanaka et al. (2012) for soft soils, as a function of penetration depth, cone apex angle, cone weight, and cone factor based on Wood (1990), was used to determine the S_u values from this test.

The effective shear strength parameters for the remoulded organic soils were estimated using direct shear testing (DST) conducted following the ASTM-D3080/D3080M (2011) standard. The device accommodated a specimen having a 60-mm square footprint and 20 mm in initial height. The stresses and shear box displacements were measured using an automatic data acquisition system. The specimens were initially consolidated to vertical effective stress levels of 50, 100, and 150 kPa, and then sheared. A shear displacement rate of 0.05 mm/min was estimated according to the ASTM-D3080/D3080M (2011) standards for fine-grained soils, assuming a shear displacement at failure to be 10 mm, and time for failure as 200 min. The objective herein was to have a shearing rate slow enough to allow for adequate dissipation of the excess pore water pressure during shear and meaningfully represent a drained shear condition.

4.3 Interpretation and Analysis of Results

4.3.1 Site Stratigraphy

The profiles of measured q_t , f_s , u_2 , and V_s with depth below the ground surface for the two seismic cone penetration tests conducted at the site (SCPTu-18-01 and SCPTu-18-02) are presented in **Figure (4.3)**. Based on the two SCPTu profiles and visual inspection, it could be inferred that the site is underlain by a relatively uniform deposit of organic soil. An upper layer of sand fill overlies the organic soils with a thickness of about 2 to 2.5 m. The organic soil layer is extended to the whole depth zone investigated using the testing techniques. The visual inspections of the soils retrieved from auger sampling indicate that the soils are mostly amorphous organic soils with the presence of fines. It is relevant to note that the organic soil at the site would have experienced consolidation due to the placement of the upper fill – i.e., the organic soils are expected to be at a higher density than that corresponding to the case of a naturally formed organic soil.

The value of cone tip resistance (qt) in the organic material is essentially below 5 bar (~500 kPa), displaying the significant contrast with the sand fill of the high qt values. The organic soil's tested depth zone has mean shear wave velocities ranging from 20 m/s to 40 m/s. According to Borcherdt (1994), it is noteworthy to compare that typical mean shear wave velocities for soft clays and silty clays would range from 100 m/s to 200 m/s. This measured low shear wave velocity indicates an incredibly soft (porous) soil matrix for organic soils, as expected. The positive induced excess water pressure at top Muskeg layers could be interpreted by the presence of fine-grained particles. The calculated Soil Behaviour Type (SBT) for the soil classification for the SCPTu-18-01 and SPTu-18-02 are presented in **Appendix (A)**.

The measured ball penetration resistance (q_b) versus depth profiles developed from the two BPT tests (BPT-18-01 and BPT-18-02) are presented in **Figure (4.4)**. Again, the observed similar values for the ball resistance (q_b) confirm the organic soil's general uniformity at this site.



Figure 4.3 Results of the seismic cone penetration test with pore water pressure measurements at test locations SCPTu-18-01 and SCPTu-18-02.



Figure 4.4 Ball penetration resistance q_b profiles at test locations BPT-18-01 and BPT-18-02.

4.3.2 Index Properties

The average values, maximum and minimum range of the index properties derived by testing samples from different depth zones are presented in **Table (4.1)**. The results indicate an increasing water content with increasing depth. It may be possible that the upper soils' consolidation due to the upper mineral fills' presence may have contributed to this observation.

Considering the high level of organic contents at the site (more than 75% for Depth Zones B and C), the soil samples were classified according to the ASTM-D4427 (2018) Standards specifically intended for peat. Based on this, the soil in Depth Zone A is classified as Hemic peat (fibre content between 33% to 67%) and high ash (ash content more than 15%) of H4/H5 degree of decomposition - which refers to slightly to moderately decomposed peat. The Depth Zones B and C are classified as Hemic and medium ash (ash content between 5% to 15%), with a degree of decomposition of H4/H5.

The Depth Zone A seems to possess higher specific gravity (more solids), and higher wet and dry densities, lesser content in terms of water, fibre and organics compared to those corresponding to Depth zones B and C. Depth zone A is the first layer of the organic soil that directly underlies the upper sand fill. This may be the reason for having more minerals in this organic zone compared to the underlying zones.

It was found that the liquid limit results obtained from the Casagrande method for all three Depth Zones are similar to those derived from the fall cone penetrometer test, despite the difficulties encountered with the former method.

	Depth Zone A (average depth of 3 m)	Depth Zone B (average depth of 4.75 m)	Depth Zone C (average depth of 6.25 m)
Water Content wc (%)	370 ± 20	570 ± 50	620 ± 30
Ash Content AC (%)	35 ± 4	13 ± 2.5	13 ± 2
Organic Content OC (%)	65 ± 4	87 ± 2.5	87 ± 2
Fall Cone Liquid Limit w⊾(%)	520 ± 20	740 ± 25	880 ± 20
Casagrande Liquid Limit w∟(%)	510 ± 15	770 ± 20	900 ± 10
Fibre Content FC (%)	47 ± 4	51 ± 4	51.5 ± 2.5
Wet Density γ _{wet} (g/cm³)	1.01 ± 0.01	0.95 ± 0.06	0.9 ± 0.04
Dry Density γ _{dry} (g/cm³)	0.75 ± 0.07	0.5 ± 0.1	0.57 ± 0.06
Specific Gravity G _s	1.96 ± 0.1	1.88 ± 0.04	1.8 ± 0.1
Degree of Humification	H4/H5	H4/H5	H4/H5

Table 4.1 Index properties of extracted samples at three different depth zones

Considering the simplicity of obtaining the water content and its ability to serve as an index of organic content, many researchers have tried to interpret many peat properties as a function of the water content using empirical relationships. With this in mind, the measured values of the organic content, specific gravity, and wet density were assessed with respect to some already established correlations in the literature, as shown in **Figure (4.5)**. It appears that the index



properties obtained for the soil in this site are in reasonably good agreement with those from the available correlations.

Figure 4.5 Measured values of (a) organic content OC; (b) specific gravity G_s ; (c) wet density ρ as a function of the water content superimposed on the available correlations in the literature.

4.3.3 Compressibility Characteristics

The observed relationships between the test specimens' settlements with elapsed time during the consolidation testing undertaken on reconstituted organic soil originating from the three Depth Zones A, B, and C are presented in **Figure (4.6)**. Good repeatability of test results can be noted from the three essentially identical tests conducted on samples from each depth. The settlements observed in the samples from Depth Zone A are less than those obtained for the samples from Depth Zones B and C. This could be attributed to the observed lower water contents and presence of more minerals in Depth Zone A.

As expected, organic soils' high compressibility, arising mainly due to the high initial void ratio and water content, is evident from the test results. As noted previously by Mesri & Ajlouni (2007), a significant part of the settlement in organic soils is expected due to secondary compression. In other words, primary consolidation under any applied stresses (due to dissipation of excess pore water pressure) occurs relatively quickly, but the secondary compression involving fibre rearrangements and creep could last over a long period.



Figure 4.6 One-dimensional consolidation settlements observed in specimens prepared from remoulded samples obtained from three different depth zones.

The results from consolidation testing conducted on undisturbed samples are compared with those from the remoulded samples in **Figure (4.7).** The figure reveals that, under a given vertical stress increment, the undisturbed samples exhibit noticeably less settlement than the remoulded samples. The remoulded specimen of Depth Zone A exhibits less settlement than the samples in the other depths due to the presence of more minerals in this depth. However, the undisturbed specimen from Depth Zone A showed more settlement than that from Depth Zone B. The tested undisturbed sample extruded from Shelby tubes of depth Zone A did not show clear signs of soil solids either in the wet or dry conditions. This may be the reason for the above apparent discrepancy.

The void ratio (e) versus log vertical effective stress (σ'_v) curves obtained from the consolidation testing of remoulded and undisturbed samples are compared in **Figure (4.8)**. It can be seen that, in general, the initial void ratio observed in the remoulded specimens is smaller than those derived from the testing of undisturbed organic soil. The specimen corresponding to Depth Zone C had a void ratio value for the remoulded specimens somewhat close to that obtained for the counterpart undisturbed specimen. This may be attributed to the high water content and low specific gravity of Depth Zone C, making the reconstitution in the laboratory simpler than the samples with a high specific gravity of solids.

The results from undisturbed specimens also indicate that the slope of the e - log (σ'_v) slope tends to stay relatively flat until the applied consolidation stress (σ'_v) reaches 25 kPa. When the stress increases beyond this stress level, the soil transitions to become relatively more compressible, leading relatively steeper slope of the e - log (σ'_v) curve – i.e., increased anticipated settlements above 25 kPa.



Figure 4.7 One-dimensional consolidation settlements observed in specimens prepared from remoulded and undisturbed samples obtained from three different depth zones.



Figure 4.8 e - log σ ' curves for comparison between remoulded and undisturbed samples.

The calculated compression index C_c from the consolidation testing (i.e., from the relatively steeper part of the curve), before and after the unload-reload cycle for both the remoulded and undisturbed samples, are presented in **Figure (4.9)**. The figure shows that the compression index C_c increases with depth for both the remoulded samples and undisturbed samples, and this could be attributed to the increase in the noted water content with depth. This contrasts with the increasing stiffness/strength with increasing depth usually found in mineral soil deposits, and it illustrates the genuine difficulties in developing behavioural trends to characterize the organic soil mass.

The data presented in the figure also highlights that the C_c of the undisturbed samples is more significant than those of the remoulded ones at all three depths, which could be attributed to the high initial void ratio for the undisturbed samples even though the remoulded samples suffered more settlement than the undisturbed ones (see **Figure 4.7**).

It is also of interest note from **Figure (4.9)** that for soils tested in this study with water contents ranging between 350% to 650%, the measured compression index C_c after the unload-reload cycle is consistently less steep than the calculated compression index before the unload-reload cycle. In essence, the e - log (σ'_v) flattens if the organic soil mass is unloaded and then reloaded, with the fitted trend-line showing a drop in C_c around 20% of the compression index due to the unload-reload process. This finding is similar to that reported by Ajlouni (2000) from tests on fibrous peat of high initial water content.

It is noted that the C_{α}/C_{c} ratio is ranging between 0.06 to 0.08 for both the remoulded and undisturbed samples. This ratio is slightly higher than the 0.05 ± 0.01 reported by Mesri & Ajlouni (2007) for fibrous peat.



Figure 4.9 Relation between compression index C_c of remoulded and undisturbed samples before and after the unload-reload cycle.

4.3.4 Undrained Shear Strength

The field vane shear tests have been used reasonably to determine S_u of fine-grained and organic soils in engineering practice for many decades. This is due to its ability to obtain a direct measurement of the in situ undrained shear strength using a relatively simple field-testing method, in contrast to the difficulties in obtaining undisturbed samples of these types of soils for laboratory testing.

Figure (4.10) presents the shear stress τ (kPa) versus vane rotation angle derived from the mobilized torque on the vane shear blade for the six different eVSTs conducted at the site. The mobilized τ value during the initial rotation (the peak value corresponding with the peak S_u) and after the soil has been remoulded (the peak torque corresponding with the remoulded S_u) are presented herein. As expected, the results indicate that the peak S_u value estimated from the initial rotation phase of the eVST at a given depth is generally higher than those observed from the remoulded phases. The soil in Depths A and B shows a soil-softening trend similar to dense and over-consolidated clays due to the presence of the fill sandy layer. The effect of the fill layer diminishes for depth C as the soil show a hardening behaviour.



Figure 4.10 Induced Shear stress τ vs. shear vane rotation angle curve during eVST testing.

The results for S_u from eVST measurements conducted at specific depths are presented in **Figure (4.11)**. The results are comparable to the in-situ shear strengths between 5 to 20 kPa reported by MacFarlane (1969) for peaty soils at depths less than 2 m. Considering the wide usage in the past and its accepted reliability, these eVST-based measurements were considered reliable for characterizing the undrained shear strength of the organic soils at the site and for comparison with those derivable from other testing methods such as SCPTu and BPT methods as described below. The undrained shear strength (S_u) of organic soils can be interpreted using available data from SCPTu tests using the empirical **Equation [4.1]** for fine-grained soils silts/clays (Robertson, 2009). Similarly, **Equation [4.2]** can be used to estimate S_u from BPT data (Boylan et al., 2011).

$$S_u = q_{net} / N_{kt}$$
[4.1]

$$S_{u} = q_{b-net} / N_{ball}$$
[4.2]

 N_{kt} and N_{ball} are empirical factors. As an initial step, a factor of 15 proposed by Campanella and Howie (2005) for fine-grained soil was employed in **Equation [4.1]. For Equation [4.2]**, the N_{ball} factor of 11 has been suggested as per available experience from ConeTec Investigations (Weemees et al., 2006) and also suggested by Boylan et al. (2011).

In line with this, the values of N_{kt} were assessed by comparing the net tip resistance q_{net} from the SCPTu measurements and six S_u values from the eVSTs at different depths. Based on this, an average value of 8 was calculated for the N_{kt} , with the 25th percentile and 75th percentile of 6 and 12, respectively. The S_u profiles for the full depth of SCPTu tests were calculated using a N_{kt} value of 8, and results are superimposed on **Figure (4.11)**. Overall, reasonable agreement with the eVST results suggests that N_{kt} value of 8 would be suitable for estimating the S_u using SCPTu results compared to the value of 15 proposed for fine-grained mineral soils by Campanella and Howie (2005).

Similarly, the values of N_{ball} were estimated by comparing the net tip resistance q_{b-net} from the BPT measurements and six eVST-based S_u values at different depths. An average N_{ball} value of 9, with the 25th and 75th percentiles of 8.5 and 10.5, respectively, was estimated based on this assessment. Using N_{ball} value of 9, the S_u profiles for the full depth of BPT tests were calculated, and results are again superimposed on **Figure (4.11)**. Excellent agreement with the eVST results suggests that a N_{ball} value of 9 would be suitable for estimating the S_u using BPT results. It is of interest to note that this estimated value of 9 also compares reasonably with the N_{ball} value of 11 suggested by Weemees et al. (2006) and Boylan et al. (2011).

The results of the BPT show less variation than the results of the SCPTu. It can be noted that the estimated S_u values are larger at shallower depths; this may be due to the compression that has occurred due to the directly overlying sandy fill material, which is also in accord with the relatively lower compressibility observed for these organic soils in the previous section. As noted by Boylan et al. (2011), the interpretation of results from SCPTu in organic soils can be erratic due to the variations in cone interaction with fibres. It is relevant to note that Viergever (1985) found that cones with larger projected areas (50 and 100 cm²) tend to give more accurate measurements of resistance, increasing the sensitivity of measurements and reducing the partial drainage. Boylan et al. (2011) also noted that full-flow penetrometers, with relatively large projected areas displacing a large volume of soils, would improve accuracy in soft soils, giving higher confidence in results than SCPTu.

The S_u values estimated from fall cone testing showed results that are generally in line with those derived from the eVSTs. The correlations used for determining the fall cone S_u values are based on the empirical relationships introduced by Tanaka et al. (2012), which were mainly presented for fine grained soils.

Figure (4.11) also reveals that the measured undrained shear strength parameter S_u using the UCS test is small and not comparable to the measured values using the eVST. This may be due to the sample disturbance and absence of confining stresses in the UCS testing.

Moreover, the testing work results suggest that the ratio of undrained shear strength to vertical effective consolidation stress (S_u/σ'_v) values ranging between 0.8 and 1.3 for this site soils. Mesri & Ajlouni (2007) have also noted high S_u/σ'_v ratios for peaty soils. Boylan and Long (2009) conducted a study of Direct Simple Shear DSS tests on peats over hundred samples collected from sixteen sites and concluded that S_u of peat is strongly influenced by the stress history, water content, and fibre content.

4.3.5 Drained Shear Strength

The results from direct shear testing (DST) provided an opportunity to assess the drained shear strength parameters for this organic soil. The maximum shear resistance (τ) observed under



Undrained shear strength parameter S_u (kPa)

Figure 4.11 Undrained shear strength S_u interpreted from different laboratory and field testing

normal effective stresses (σ ') of 50 kPa, 100 kPa, and 150 kPa are presented in **Figure (4.12)**. Since the data seem to follow a similar trend with a relatively low scatter on the τ versus σ ' graph, a "trendline band" with a 95% confidence interval could be developed, as shown in the figure. Based on this, the effective stress failure envelope for the tested material can be characterized by one failure envelope having a shear strength intercept (c') of 7 kPa and an effective angle of friction (ϕ ') of 31°. The interpreted values of both c' and ϕ ' are similar to the values presented in the literature for the same kind of soils (Edil & Wang, 2000; Mesri & Castro 1987; Lee et al. 2015). The observed value for the angle of shearing resistance is in general conformance with the range between 20° to 40° proposed by Farrell & Hebib (1998) and Hebib (2001), again, based on direct shear testing. Many studies have shown that fibrous organic materials could possess relatively high friction angles because of the potential interlocking action between individual fibres (Adams, 1965; MacFarlane, 1969; Landva & La Rochelle, 1983).



Figure 4.12 $\tau - \sigma$ ' curve of the Direct Shear Test DS results for three different samples in each depth.

4.3.6 Small-Strain Shear Stiffness

Small strain shear stiffness (usually termed as G_0 or G_{max}) becomes a critically crucial geotechnical engineering parameter in applications such as seismic ground response analyses, the design of machine foundations, and advanced constitutive modelling of soils. In current engineering practice, the value of G_{max} is best estimated using shear wave velocity measurements. As such, the in-situ shear wave velocity (V_s) measured using SCPTu testing provides an opportunity to directly determine the G_{max} values using **Equation [4.3]** (Note: ρ = total mass density of soil).

$$G_{\rm max} = \rho \, V_{\rm s}^2 \tag{4.3}$$

Many correlations between G_{max} for fine-grained mineral soils and undrained shear strength S_u through a constant linear proportionality (k_g) given in **Equation [4.4]** can be found in the literature. This is partly because both G_{max} and S_u are primarily dependent on common parameters of void ratio, effective stress, and stress history. The k_g has been determined to be a function of overconsolidation ratio; a value of k_g in the order of 1000 has been suggested for normally consolidated soils (Arango et al., 1978).

$$G_{max} = k_g S_u$$
 [4.4]

With this in mind, the values of G_{max} obtained from shear wave velocity measurements for the organic soils at the site are compared with the counterpart measured S_u from eVST measurements in **Figure (4.13).** The results reveal a value of k_g would equal around only 40 (with 25th and 75th percentiles for this constant are 30 to 55, respectively), much less than the value used for mineral soils.

The G_{max} obtained from V_s is plotted with respect to the net tip resistance q_{b-net} from BPT tests in **Figure (4.14)**. The results suggest that the G_{max} could be approximated by multiplying the value of q_{b-net} by a factor C_g (see **Equation [4.5]**) of about 4.4.

$$G_{max} = C_g q_{b-net}$$
[4.5]



Figure 4.13 Relation between G_{max} calculated from shear wave velocity and peak undrained shear strength S_u calculated from eVST measurements.



Figure 4.14 Relation between G_{max} calculated from shear wave velocity and ball net tip resistance q_{b-net} from BPT.

4.4 Summary

A natural organic soil (peat) deposit located in a research test site in British Columbia, Canada, was characterized through a comprehensive geotechnical investigation involving a range of field and laboratory tests.

A range of in situ and laboratory geotechnical investigations were undertaken with the intent of understanding the compressibility, shear stiffness, and strength of organic soils. The field-testing program comprised: cone penetration testing (SCPTu) with shear wave velocity (V_s) measurements, full-flow ball penetration testing (BPT), electric vane shear testing (eVST), along with fixed-piston tube sampling that was undertaken in drill holes put down at the site underlain by organic soil. Laboratory testing included: index property testing (i.e., water content, organic content, fibre content, specific gravity, wet and dry densities, degree of humification and liquid limit), one-dimensional consolidation testing, and direct shear testing.

The measured laboratory index parameters for the site soils showed good agreement with the available correlations in the literature for index properties (i.e., organic content, specific gravity, and wet density) and water content. The results from one-dimensional consolidation tests indicated that the initial void ratio observed in the remoulded specimens was smaller than those derived from the testing of undisturbed organic soil. The results also showed that the usually found behavioural trends of increasing stiffness with increasing effective confining stress in mineral soils could not be extended to organic soils due to their inherently variable organic/fibre contents and relatively complex heterogeneities.

The availability of data from electric vane shear (eVST) tests from the field allowed assessing the use of available correlations between the undrained shear strength (S_u) and the cone net tip resistance (q_{net}) observed from cone penetration (SCPTu) testing. It was found that reasonable agreement for the correlation between (SCPTu) and S_u could be obtained if a value
of 8 was used for the commonly used N_{kt} factor. The results also showed that the N_{kt} value of 15 proposed for fine-grained mineral soils would not be suitable for use in organic soils.

The ball penetrometer test (BPT) seemed to display more regular traces of penetration resistance versus depth compared to those from CPT tests; it is felt that this is due to the ability of the former to mobilize a larger volume of soil around the probe, and in turn, providing a more representative average response for the tested mass. This is in accord with the previous research that has shown that full-flow penetration tools are more desirable for soils that are very soft consistency. For the tested organic soils, an average N_{ball} factor of 9 was found to provide a suitable correlation between the ball net tip resistance (q_{b-net}) from BPT tests with eVST-based S_u.

In situ shear wave velocity (V_s) measured using SCPTu testing were used to directly estimate the small strain shear modulus (G_{max}), and this provided an opportunity to correlate with S_u . It was found that the G_{max} could be approximately obtainable by direct multiplication of the Su by a factor k_g of about 40. Moreover, the results indicated that the value of G_{max} could be approximated by direct multiplication of by direct multiplication of the value of q_{b-net} by a factor C_g of about 4.5.

The outcomes demonstrated the potential of the ball penetration test to serve as a useful tool to characterize the strength and stiffness properties of natural organic soil deposits. It was noted that organic soils are highly variable due to the presence of organic fibres with various degrees of decomposition. As such, in spite of the observed promising trends, the use of correlations presented should be used with appropriate checks conducted based on site-specific correlation.

Chapter 5: Methodology for Muskeg Soil Stabilization using the Microbially Induced Calcite Precipitation MICP Technique

This chapter presents the followed methods to (i) grow the bacteria, (ii) prepare the bioslurry, (iii) prepare the samples for testing, (iv) test the compressibility and shear strength properties, (v) measure the calcite content, (vi) check the microscale behaviour, and (vii) present the testing programs for stabilization with bioslurry only and bioslurry with sand simultaneously.

5.1 Bacteria and Growth Conditions

Sporosarcina pasteurii ATCC[®] (11859[™]) bacteria used for the urea hydrolysis were received from CEDARLANE, an authorized distributor of the American Type Culture Collection ATCC products in Canada.

The growth medium for these bacteria was the ammonium-yeast extract (NH₄-YE) medium, according to the ATCC 1376. The instructions for making the liquid bacterial culture attached to the *Sporosarcina pasteurii* ATCC[®] (11859TM) data sheet were followed to make stock cultures. The stock cultures were stored in the - 80°C freezer to be used whenever needed to grow the bacteria.

The ingredients of the ATCC 1376 medium were as follows: (i) yeast extract (20 g/L), (ii) ammonium sulphate (NH₄)₂SO₄ (10 g/L), (iii) 0.13 M Tris buffer of pH = 9 (1 L), (iv) Agar (for the solid media) (20 g/L). The ingredients were autoclaved separately to ensure the bacterial growth. These chemicals were obtained through Fisher Scientific, VWR, and Alfa Aesar. They were sterilized separately in the autoclave at 121°C for an average of 30 minutes based on the medium's volume. (a picture of these four chemicals is shown in **Figure 5.1**).



Figure 5.1 Different chemicals used in the bacterial growth, according to ATCC 1376.

5.1.1 Forming Single Colonies of Bacteria

To ensure the purity of the bacterial culture and avoid contamination during the growth process, a single colony of the bacteria should be grown initially and then used to inoculate the liquid media to form the liquid bacterial culture. The following steps were followed to grow single colonies of bacteria (as shown in **Figure 5.2**)

(1) The ATCC 1376 solid media was prepared by adding the agar to the other chemicals.

(2) The total volume of the prepared solid media was distributed between different Petri dishes.

(3) Using the stock culture stored in the - 80°C, the solid media in the plates were streaked with the inoculation loop.

(4) After streaking the plates, the plates were placed in the incubator at 30°C without shaking for 24 to 48 hours.

(5) The bacteria were grown and formed different colonies inside the plates.

5.1.2 Forming Bacterial Culture

After growing the bacteria's single colonies, these single colonies were used to inoculate the liquid media to form the bacterial liquid culture. The following steps were followed to form the bacterial liquid culture, according to Al Qabany et al., 2012 (As shown in **Figure 5.3**)

(1) The requested volume of the ATCC 1376 liquid media (without agar) was prepared.

- (2) The volume was distributed in glass tubes.
- (3) Each tube was inoculated with a single colony of bacteria from the Petri dishes.
- (4) The tubes were incubated at 30 °C and shook at 200 rpm for 24 to 48 hours.

(5) The optical density OD of the grown bacteria was checked at 600 nm intensity, and incubation stopped whenever the OD_{600} was between 0.8 to 1.2. This optical density was chosen to ensure the high urease activity, according to Stocks-Fischer et al. (1999).

5.2 Bioslurry Preparation

The first stage of the bioslurry method was to prepare the bioslurry and mix it with the soil. The procedure to form the bioslurry was following Cheng & Shahin (2016) and presented in **Figure (5.4).** The bioslurry was formed by adding equ-moles of the urea and calcium chloride to the bacterial culture and stirring them for 12 hours at 600 revolutions per minute (rpm) and letting them settle for 6 hours. Afterwards, the supernatant was disposed of, and bioslurry (precipitated calcite and bacteria) was settled and harvested at the container's bottom. The same procedure was followed in this study except that the mixture was centrifuged at 4000 rpm instead of letting it settle. This made the procedure faster and ensured that most of the bacterial cells would precipitate with the calcite crystals. The percentage of the precipitated calcite and bacteria was 10%.



Figure 5.2 Sequence of growing a single colony of the *Sporosarcina pasteurii* bacteria; **(a)** preparation of solid media, **(b)** distributing the solid media in Petri dishes, **(c)** using the stock culture to inoculate the solid media by streaking with the inoculation loop, **(d)** incubating the Petri dishes at 30 °C for 24 to 48 hours, and **(e)** bacterial growth and formation of single colonies.



Figure 5.3 Procedure of forming the *Sporosarcina pasteurii* bacterial culture; (a) preparation of liquid media, (b) distributing the liquid media in glass tubes, (c) inoculating each tube with a single bacterial colony, (d) incubating the liquid media at 30 °C and shaking at 200 rpm for 24 to 48 hours, (e) turbidity in the solution, reflecting the successful bacterial growth, and (f) measuring the OD₆₀₀ of the bacterial culture using the Spectrophotometer.

The precipitated bioslurry was mixed with the soil only for the stabilization using the bioslurry only and with the soil-sand mixture for the stabilization with sand and bioslurry simultaneously. The second stage of the urease active bioslurry approach was to inject the cementation solution into the soil-bioslurry mixture or soil-sand-bioslurry mixture. The cementation solution was a solution of urea and calcium chloride. The urea and calcium chloride concentrations were equal, same as the concentration used to form the bioslurry.









Figure 5.4 Process of bioslurry preparation: (a) growing the bacterial culture, (b) adding urea and calcium chloride to the bacterial culture and stir for 12 hours at 600 rpm, (c) centrifuging the bioslurry mixture for 20 minutes at 4000 rpm, and (d) getting rid of the supernatant and bioslurry is settled at the bottom of the centrifuge tube.

5.3 Sample Preparation

Various samples were prepared by mixing the bioslurry with soil or mixing the bioslurry with the sand-soil mixture. The soil's field density represented the soil density in the mixture (1.1 g/cm³). The field density was determined based on the weight and volume of the soil samples in the Shelby tubes.

The equivalent density of the mixture was calculated based on the bioslurry and sand percentages. The total weight was calculated based on the known volume of the sample's container and equivalent density. After mixing the bioslurry with soil or the bioslurry with sand and soil, the mixture was left for 4 to 6 hours to allow for the bacterial attachments to the soil particle.



Figure 5.5 3D printed caps for the preparation of the consolidation samples

The 3D printed caps, shown **in Figure (5.5)**, were used to prevent leakage during the cementation solution injection. A similar configuration was prepared for the direct shear cuboid sample. The consolidation ring or direct shear cuboid are placed in between the top and bottom caps, and then the two caps are pressed against each other by the four screws at corners. The rubber gasket around the consolidation ring's or direct shear cuboid's perimeter helped prevent

any leakage. Moreover, the mild slope at the bottom cap helped accumulate the cementation solution before penetrating the soil mass and confirmed the cementation solution's uniform distribution.

Afterward, multiple cementation solutions were injected with a retention period of 24 hours for each cementation solution volume. The volume of one cementation solution was determined as the same pores' volume. A peristaltic pump was used to inject the cementation solution with a low rate of 0.5 millilitres/minute. This slow rate was chosen after several trials with different values to prevent clogging around the injection port.

Figure (5.6) shows the different steps for the sample preparation, starting from the mechanical mixing with bioslurry only or with bioslurry and sand simultaneously, followed by the cementation solution's injection using the peristaltic pump.









Figure 5.6 Procedure for urease active bioslurry approach, **(a)** mixing the predetermined soil mass with the appropriate amount of bioslurry or bioslurry and sand by weight, **(b)** adding the mixture to the consolidation ring, **(c)** placing the consolidation ring between the two 3D printed cabs and tight them together, and **(d)** injection of the cementation solution.

5.4 Testing Programs

5.4.1 Muskeg Soil Stabilization with Bioslurry Only

For the stabilization with bioslurry only, the testing program's schematic diagram is shown in **Figure (5.7)**. The effect of different bioslurry concentrations on the compressibility properties was checked in Phase (1). The cementation solution volume was twice the pores' volume. Each combination was tested three times, in this phase, to confirm the preparation and testing methods' repeatability. The bioslurry weight's effect on the compressibility properties was investigated in Phase (2) while fixing the cementation solution volume to twice the pores' volume and bioslurry optimum concentration (0.4 Mole/Litre). The effect of the injected cementation solution volume on both the compressibility and shear strength properties were investigated in Phase (3) while using the optimum bioslurry concentration from Phase (1) and optimum bioslurry weight from Phase (2).



Figure 5.7 Schematic diagram of the proposed testing program for Muskeg soil stabilization with bioslurry only.

5.4.2 Muskeg Soil Stabilization with Sand and Bioslurry Simultaneously

For stabilization with bioslurry and sand simultaneously, the testing program's schematic diagram is shown in **Figure (5.8)**. In the first part, the effect of changing the sand percentage on the compressibility properties would be investigated. In this part, each combination was tested three times to confirm the preparation and testing methods' repeatability. The microstructure analysis is checked as well in this part through the SEM, EDS, and XRD analysis. The effect of injecting different cementation solution volumes was checked in the second part.



Figure 5.8 Schematic diagram of the proposed testing program for Muskeg soil stabilization with sand and bioslurry simultaneously.

5.5 Measuring the Amount of Precipitated CaCO₃ in the Soil

The amount of the precipitated calcite in the soil mass due to the MICP stabilization technique was measured following the ASTM-D4373 (2014) guidelines. It determines the CaCO₃ percentage by weight by adding 1 N hydrochloric acid HCl to one gram of dried soil in an enclosed reactor. Due to the reaction between CaCO₃ and HCl, carbon dioxide is generated, which increases the pressure inside the reactor. The pressure difference could determine the calcite content by comparing it with pre-calibrated pressure measurements using reagent-grade calcium carbonate.

For each sample, the CaCO₃ weight was calculated at four separate quarters, and the average value was used to express the amount of CaCO₃ in the sample. A similar weight of the CaCO₃ within each sample's four quarters indicated the uniform distribution of the precipitated calcite. s

5.6 Consolidation and Direct Shear Tests

The standard Incremental Loading IL consolidation was used to determine the compressibility properties of both treated and untreated samples following the ASTM-D2435/D2435M (2020) guidelines. The Automated Oedometer System manufactured by GDS was used. The GDS automated system allowed recording both deformation and load measurements in short time intervals. The chosen time interval was five seconds to capture the rapid primary consolidation behaviour of Muskeg soils. The samples' dimensions were fixed to a diameter of 63.33 mm, and a height of 22.5 mm. The loading/unloading scheme for all samples was unified. The chosen loading stages were 6.25, 12.5, 25, 50, 100, and 150 kPa, followed by unloading stages of 100 and 25 kPa, and ended by re-loading stages of 100, 150, 200, 250, and 300 kPa.

The direct shear test was used to determine the effective shear strength parameters following the ASTM-D3080/D3080M (2011) guidelines. The Automated Shear Base System manufactured by GDS was used. The automated system allowed for measuring stresses and deformations automatically every two seconds during the whole test time. The sample dimensions were 60 mm in length, 60 mm in width, and 20 mm in height. The chosen axial stresses are 25, 50, and 75 kPa. An individual specimen was allowed to consolidate under these stresses until the complete primary consolidation occurs and then sheared along a predefined horizontal shearing plane. The shearing rate was calculated and equalled 0.05 mm/min following the related ASTM standards' recommendations.

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5.7 Microstructure Analysis

The microstructure behaviour of untreated and treated samples was investigated to compare the raw and bio-cemented status. The Field Emission Scanning Electron Microscope FESEM (Tescan Mira3 XMU) was used to characterize the crystal shape and size of the precipitated calcite CaCO₃ and investigate the bonding behaviour between the soil particles and precipitated calcite. The FESEM analysis was coupled with the Energy Dispersive X-Ray Spectroscopy EDS (Oxford Aztec X-max system), which investigated the selected area's elemental compositions within the FESEM picture. The sample was dried using the freeze-drying technique for around 48 hours to ensure no change in the pores' volume. The sample was cut into four pieces by hand, and a small representative portion was fixed on the aluminum stub using carbon tape. The specimen was then sputter-coated with palladium (Pd) and platinum (Pt) of 20/80 ratios under high vacuum until a 15 µm coating thickness at 0.002 kPa.

The Rietveld method was used for the X-Ray Diffraction (XRD) analysis. The samples were reduced to the optimum grain-size range for quantitative X-ray analysis (<10 μm) by grinding under ethanol in a vibratory McCrone XRD Mill (Retsch GmbH, Germany) for 5 minutes. Continuous-scan X-ray powder-diffraction data were collected over a range 3-80° 20 with CoKα radiation on a Bruker D8 Advance Bragg-Brentano diffractometer equipped with a Fe filter foil, 0.6 mm (0.3°) divergence slit, incident- and diffracted-beam Soller slits and a LynxEye-XE detector. The long fine-focus Co X-ray tube was operated at 35 kV and 40 mA, using a take-off angle of 6°.

5.8 Summary

This chapter presented the detailed methodology to stabilize Muskeg soils either by adding bioslurry only or sand and bioslurry simultaneously. Sporosarcina pasteurii ATCC® (11859TM) bacteria used for the urea hydrolysis. The bacterial growth and bacterial culture preparation were discussed. The bioslurry preparation was demonstrated with some modifications on the proposed

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methodology by Cheng & Shahin (2016). The process of adding the bioslurry to the soil and injection of the cementation solution were demonstrated. The utilization of the 3D printed caps facilitated the injection of the cementation solution and confirmed the full sealing of the sample without any leakage. The measurement of the precipitated calcite percentage by weight using the related standards was discussed. The macroscale testing methodology by consolidation and direct shear testing and microscale testing by the SEM, EDS, and XRD analyses were demonstrated.

Chapter 6: Results of Muskeg Soil Stabilization Using Bioslurry Only

6.1 Background

The testing program presented in chapter (5) for Muskeg stabilization using bioslurry only demonstrated three phases to check the effect of changing key parameters on the behaviour of treated samples. Therefore, this chapter will present the results of these three phases. In Phase (1), the results for the effect of changing the bioslurry concentration would be discussed. The results of the influence of changing the bioslurry weight would be demonstrated in Phase (2). In Phase (3), the results of the effect of changing the cementation solution volume would be presented.

It is worth mentioning that the soil used for Phase (1) was from Depth Zone C, but the soil used for Phases (2) and (3) were from Depth Zone (B). The soil's index properties in these Depth Zones were presented in Chapter (4) in **Table (4.1)**.

6.2 Phase (1): Effect of Bioslurry Concentration

6.2.1 e - log σ ' Curve

Muskeg soils exhibit high compressibility when subjected to external loads due to their high initial void ratio and water content. The MICP helps decrease the initial void ratio and initial water content by precipitating calcite within the soil pores, which would improve the compressibility properties and decrease the settlement.

The compressibility properties of the different bioslurry concentrations are investigated, and results of e - log σ ' curves are presented in **Figure (6.1)**. The consolidation test is repeated three times for the untreated and treated samples at each bioslurry concentration to confirm the stabilization process's repeatability. However, only one sample from each concentration is shown in **Figure (6.1)** for the sake of readability. The figure reveals that the initial void ratio for treated samples decreases and continues to decrease for higher bioslurry concentrations. The change in the initial void ratio is evident between the untreated and treated samples. The untreated sample's initial void ratio is 9.4, but the initial void ratio equals 7.77 for the 0.2 M/L bioslurry and 5.87 for the 0.4 M/L bioslurry. The difference in the initial void ratio for subsequent bioslurry concentration is not significant. Therefore, the 0.6 M/L, 0.8 M/L, and 1 M/L curves are similar.



Figure 6.1 e - log σ ' curve showing the effect of the bioslurry concentration on the compressibility properties.

6.2.2 Compression Index (C_c) and Swelling Index (C_s)

The compression index is a significant factor in the settlement calculation due to the consolidation process. The changes of both the compression index C_c and swelling index C_s when changing the bioslurry concentration and error bars due to each concentration's repetition are shown in **Figure (6.2)**. The figure indicates that the C_c before and after the unload-reload cycle decrease due to the treatment process.

The calculated C_c before the unload-reload cycle is higher than the C_c calculated after the unload-reload cycle. This means that the e - log σ' curve tends to flatten after the unload-reload cycle, and this behaviour has been noticed by Ajlouni (2000) for the undisturbed samples of

fibrous peat. The precipitated calcite in the soil skeleton does not change this behaviour except for the 1 M/L bioslurry, where the C_c does not change before and after the unload-reload cycle.



Figure 6.2 Change in compression index C_c and swelling index C_s due to the change of the bioslurry concentration.

The calculated C_c of the untreated soil before the unload-reload cycle is 4.5 on average, but for the 0.4 M/L bioslurry, the C_c equals 2.2 on average, which means that the C_c has decreased by 50 %. For bioslurry concentrations higher than 0.4 M/L, the change in the C_c is not remarkable. The same behaviour is observed for the calculated C_c after the unload-reload cycle with a 40% decrease when the bioslurry concentration is 0.4 M/L.

One-way ANOVA (Analysis Of Variance) and Post Hoc tests (Least Significant Difference LSD, Bonferroni, and Tukey) were conducted to check if there is a statistically significant difference in the calculated C_c when changing the bioslurry concentration. The ANOVA leads to a P-value of 1.9×10^{-9} and 0.006 for the C_c change before and after the unload-reload cycle, respectively. These values show that at least one of the observations is different. To check which observation's means are different, the Post Hoc analysis shows a statistically significant difference when the bioslurry concentration increases to 0.4 M/L, and there is no statistically significant difference when the bioslurry concentration is more than 0.4 M/L.

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Figure (6.2) also depicts the decreasing trend of the swelling index C_s when the bioslurry concentration increases to 0.4 M/L. The C_s is decreased from 0.7 for the untreated sample to 0.25 for the 0.4 M/L bioslurry, which means that the C_s has decreased by 65 %. The C_s/C_c ratio for the untreated sample is 0.16, and this ratio has decreased till 0.11 for the different bioslurry concentrations, including the 0.4 M/L bioslurry. One-way ANOVA and Post Hoc analyses were conducted for the C_s as well. The ANOVA results in a P-value of 1.3×10^{-9} , reflecting the statistically significant difference in the measured swelling index due to the addition of bioslurry. Post Hoc analysis for the swelling index confirms the significant change in the swelling index up to a bioslurry concentration of 0.4 M/L, same as the C_c .

6.2.3 Coefficient of Volume Compressibility (m_v)

Muskeg soils are known for their high coefficient of volume compressibility m_v as opposed to mineral soils. Treating Muskeg soils with bioslurry leads to a decrease in the interpreted m_v , as shown in **Figure (6.3).** The figure differentiates between the change of m_v during the loading and re-loading stages of the consolidation test.

Figure (6.3a) shows that m_v for untreated samples range from $12x10^{-4}$ to $2.8x10^{-4}$ (1/kPa), but when the bioslurry concentration equals 0.4 M/L m_v ranges from $5.9x10^{-4}$ to $1.4x10^{-4}$ with a decreasing percentage of around 50% at low and high-stress levels. The decrease in the interpreted m_v values when changing the bioslurry concentration from 0.4 to 1 M/L is not statistically significant as per the two-way ANOVA and Post Hoc analyses. The Post Hoc analysis reveals a P-value of 0.169 between the 0.4 and 1 M/L bioslurry concentration using the Least Significant Difference LSD method.

The same observations are captured for the change of the m_v with the bioslurry concentration during the re-loading stages, as shown in **Figure (6.3b)**. The primary difference between the m_v values interpreted during loading and re-loading stages is the small m_v values

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during re-loading stages for both the untreated and treated samples. This could be attributed to the decrease in the void ratio as the applied stresses increase.





6.2.4 Secondary Compression Index (C_α)

Muskeg soils exhibit higher secondary compression than other mineral soils. The secondary compression index C_{α} is stress-dependent which means that it changes when the applied stress changes. The precipitation of the calcite crystals within the soil pores has improved the secondary compression behaviour of Muskeg soils. **Figure (6.4)** presents the change in the calculated C_{α} during both the loading and re-loading stages due to the bioslurry concentration change.

Figure (6.4a) shows a steep increase in C_{α} for the untreated and treated samples at the low-stress level and reaches a plateau at 50 kPa stress level. The calculated C_{α} equals around 0.25 for the untreated sample at high-stress level and decreases to 0.16 and 0.12 for the 0.4 and 1 M/L bioslurry concentrations, respectively. The C_{α}/C_{c} ratios for untreated samples, 0.4 and 1

M/L bioslurry and equal 0.05, 0.07, and 0.07, respectively. Adding bioslurry to Muskeg soils significantly decreases the C_c and C_α but increases the C_α/C_c ratio as the C_c decreasing rate is less than the decreasing rate of the C_α . Two-way ANOVA and Post Hoc tests investigate the significant difference in the C_α values between the 0.4 and 1 M/L. The results indicate no statistically significant difference, particularly at high-stress levels, as the Tukey method's P-value equals 0.255.





Figure (6.4b) demonstrates the change of C_{α} during re-loading stages when changing the bioslurry concentration. Similar behaviour of the C_{α} change during loading stages is observed during re-loading stages. The C_{α} decreases from 0.24 at 250 kPa for untreated samples to 0.13 and 0.11 for 0.4 and 1 M/L bioslurry concentrations. The decreasing percentage is 50%. The C_{α}/C_{c} is increased from 0.05 for untreated samples to 0.06 for 0.4 and 1 M/L bioslurry concentrations.

6.2.5 Constrained Modulus (M)

One of the advantages of the consolidation test is that it could give some insights into the soil stiffness in the constrained conditions. The effect of the bioslurry concentration on the interpreted constrained modulus M at different strain levels is presented in **Figure (6.5)**.

Figure (6.5a) demonstrates the change in the constrained modulus M before the unloadreload cycle in the consolidation test. The figure reveals that M tends to increase when the strain level increases due to decreased pore sizes at high strains. However, at low strains less than 0.15, the increase in the constrained modulus is slight. At strain levels higher than 0.15, the increase in constrained modulus is remarkable. On the other hand, adding the bioslurry to the soil mass increases M and reduces the induced strains due to the calcite crystals' precipitation in the soil pores. At the highest strain, M increases from 710 kPa in the untreated sample to 910 kPa and 950 kPa for 0.4 and 1 M/L bioslurry, respectively, with an improvement percentage of 18% to 23%. The percentage of improvement when increasing the bioslurry concentration from 0.4 to 1 M/L is not statistically significant as per the two-way ANOVA and Post Hoc tests where the Pvalue of the Tukey's Post Hoc test is 0.475.



Figure 6.5 Influence of changing the bioslurry concentration on the constrained modulus M; (a) before the unload-reload cycle, and (b) after the unload-reload cycle.

The change in M after the unload-reload cycle is presented in **Figure (6.5b)**. The figure reveals a decrease in the induced strain in the soil mass due to the addition of the bioslurry without a significant increase in M.

6.2.6 Unload-Reload Stiffness (Eur)

The unload-reload stiffness E_{ur} could be calculated from the slope of the unload-reload cycle in the stress-strain curve resulted from the consolidation test. **Figure (6.6)** illustrates the E_{ur} increase when changing the bioslurry concentration up to 0.4 M/L and reaches a plateau afterwards. The E_{ur} of untreated sample equals 2200 kPa and increases to 3200 kPa when the bioslurry concentration is 0.4 M/L, with a 45% increase. The increase in the unload re-load stiffness means a flatter unload-reload cycle due to the precipitation of the calcite crystals in the soil pores, which impedes the soil dilation when unloaded. The one-way ANOVA reveals a statistically significant difference between the 0.4 and 1 M/L bioslurry with a P-value of 0.228 using the Least Significant Difference LSD method.



Figure 6.6 Effect of the bioslurry concentration on the unload-reload stiffness Eur.

6.2.7 Calcium Carbonate (CaCO₃) Content

The even distribution of the calcite (CaCO₃) crystals in the tested samples is essential for consistency. **Figure (6.7)** demonstrates the change in the amount of precipitated calcite while increasing the bioslurry concentration. At each bioslurry concentration, the error bars reveal a slight standard deviation between the measurements of four quarter of each treated soil sample. This confirms the uniform distribution of the precipitated calcite in the soil. The figure reveals an increase in the amount of CaCO₃ up to 40 % till the bioslurry concentration reaches a value of 0.4 M/L. When the bioslurry concentration is more than 0.4 M/L, no further increase in the CaCO₃ amount is noticed.

One-way ANOVA accompanied by Post Hoc analysis are conducted to check if there is a statistically significant difference between the different amounts of the precipitated calcite while changing the bioslurry concentration. The ANOVA reveals a significant difference in the various means with a P-value of $3x10^{-7}$, and Post Hoc analysis using Tukey's method reveals no statistically significant difference when the bioslurry concentration is more than 0.4 M/L.



Figure 6.7 Variation of the calcite amount while changing the bioslurry concentration.

6.2.8 Microstructure Change due to the Bioslurry Treatment

The various techniques for microstructure analysis such as XRD, SEM, and EDS have been used to better understand the Muskeg soil behaviour before and after treatment. Figure (6.8) shows the XRD patterns of the untreated sample and treated samples with 0.4 and 1 M/L bioslurry. A hump is noticed in the XRD pattern for the untreated sample between 2θ equals 10° and 30° due to the Muskeg soils' amorphous nature. The percentage of the amorphous component in the untreated sample is about 80%, as presented in **Table (6.1)**. This hump is absent for the treated samples, confirming the transformation of amorphous phases in the soil mass into more crystalline phases due to the bioslurry treatment. The XRD patterns of treated samples also show the formation of calcite and vaterite crystals in the soil mass after treatment. The intensity of calcite peaks increases when the bioslurry concentration increases from 0.4 to 1 M/L. The variation in the weight percentage of different chemical compounds is also determined by Rietveld quantification, as presented in Table (6.1). The calcite content increases from 35% at 0.4 M/L to 57% at 1 M/L, and at the same time, the vaterite content decreases from 47% to 32%. This reflects the phase transformation from vaterite to calcite at high bioslurry concentration. The quantification results also demonstrate no noticeable difference in the total weight of calcite and vaterite crystals in the soil mass when the bioslurry concentration is more than 0.4 M/L. This finding elucidates why Muskeg soil properties have no further improvement when the bioslurry concentration is more than 0.4 M/L.

The EDS results show carbon and oxygen as the dominant elements for the untreated samples and calcium element in the treated samples with both the 0.4 and 1 bioslurry. The SEM analysis provides more insights into the significance of the calcite and vaterite precipitations in the soil mass, as shown in **Figure (6.9)**. **Figures (6.9a) and (6.9b**) show no calcite or vaterite crystals in the untreated samples. The presence of the calcite crystals (with sharp edges) is evident for the treated soil with 1 M/L bioslurry (**Figures 6.9e and 6.9f**), but the vaterite crystals

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(rounded-shape crystals) are well-spread in the treated soil with 0.4 M/L bioslurry (**Figures 6.9c and 6.9d**). Although there is clear evidence of more calcite and vaterite crystals when the soil is treated with 1 M/L bioslurry compared to the 0.4 M/L bioslurry, this difference is not large enough to cause a significant improvement in the soil properties.



Figure 6.8 XRD patterns for untreated sample and treated samples with 0.4 and 1 M/L bioslurry.

Table 6.1 Chemical composition quantification after Rietveld analysis of untreated and treated

	Untreated sample	0.4 M/L	1 M/L
Calcite	-	35	57
Vaterite (%)	-	47	32
Quartz (%)	10	2	2
Albite low, calcian (%)	-	1	-
Amorphous + Background. (%)	80	15	9
Andesine (%)	7	-	-
Clinochlore (%)	1	-	-
Illite/Muscovite 2M1 (%)	2	-	-
Calcite + Vaterite (%)	0	83	89

samples with bioslurry only.



Figure 6.9 SEM images for **(a)** untreated sample at 2000x magnification, **(b)** untreated sample at 5000x magnification, **(c)** treated sample with 0.4 M/L bioslurry at 2000x magnification, **(d)** treated sample with 0.4 M/L bioslurry at 5000x magnification, **(e)** treated sample with 1 M/L bioslurry at 2000x magnification, and **(f)** treated sample with 1 M/L bioslurry at 5000x magnification.

6.3 Phase (2): Influence of Bioslurry Weight

In Phase (1), the bioslurry weight is fixed to 10% of the mixture's total weight as a start point proposed by Cheng & Shahin (2016) for sandy soils. In this phase, the effect of changing the bioslurry weight, from 5% to 20% by an increment of 5%, on the compressibility properties of Muskeg soil is studied. It is worth mentioning that the soil samples used for this phase are from Depth Zone (B), which is why the value of the untreated samples is different from Phase (1).

6.3.1 Compression Index (C_c) and Swelling Index (C_s)

Adding bioslurry to the soil mass is expected to change the compressibility properties of Muskeg soil if the injection of the cementation solution accompanies it. **Figure (6.10)** presents the effect of changing the bioslurry weight on the interpreted compression index C_c before and after the unload-reload cycle and swelling index C_s . The figure shows a substantial improvement in the compressibility properties as the C_c before the unload-reload cycle drops from 3.1 for the untreated sample to 2.1 for the treated sample with 5% bioslurry weight corresponding to a 30% decrease. A similar trend is observed for the C_c after the unload-reload cycle with the same decreasing percentage. Moreover, the figure reveals no further improvement occurs to the soil mass when the bioslurry weight exceeds 5%. The interpreted C_c after the unload-reload cycle equals 90% of the interpreted one before the unload-reload cycle for treated samples with indicates the change in the compressibility properties of treated Muskeg soils with bioslurry to be similar to the mineral soils' behaviour as mineral soil exhibits the same C_c before and after the unload-reload cycle.

The figure also shows a decrease in the swelling index C_s when the soil is treated with 5% bioslurry. No further decrease is observed in the swelling index C_s when the bioslurry weight is

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more than 5%. The C_s/C_c ratio equals 11% for untreated samples and decreases to 8% for treated samples with 5% bioslurry.



Figure 6.10 Change in compression index C_c and swelling index C_s while varying the bioslurry weight.

6.3.2 Secondary Compression Index (C_α)

Although increasing the bioslurry weight by more than 5% does not affect the compression index C_c and swelling index C_s , it decreases the secondary compression index C_{α} as shown in **Figure** (6.11). This could be attributed to the calcite particle re-arrangement in the soil pores when the bioslurry percentage by weight increases. As Muskeg soils are known for their high secondary compression indices compared to mineral soils, improving this behaviour via the calcite precipitation would decrease the soil mass's total settlement.

Figure (6.11a) shows the change in the C_{α} with the various applied stresses in the loading stages while changing the bioslurry percentage by weight. The figure shows that the C_{α} tends to settle when the applied stress is 100 or 150 kPa. Moreover, at these stresses, increasing the bioslurry percentage decreases the C_{α} from 0.25 for the untreated samples to 0.14 for the 20% bioslurry. The decreasing percentage is 45%.



Figure 6.11 Effect of bioslurry percentage of the total sample weight on the secondary consolidation index C_{α} during; (a) loading stage, and (b) reloading stages.

Figure (6.11b) demonstrates the effect of changing the bioslurry percentage by weight on the interpreted C_{α} during the re-loading stages. The figure shows an increasing trend for the C_{α} while increasing the re-loading stresses with a plateau when the re-loading stress equals 200 kPa. The figure also reveals that the C_{α} in the re-loading stages is less than the C_{α} interpreted during loading stages for both the untreated and treated samples. This would emphasize the importance of loading and unloading the Muskeg soil to decrease the secondary compression settlement. C_{α}/C_{c} ratio for untreated samples is 0.08 for loading stages and decreases to 0.065 for the re-loading stages.

6.3.3 Unload-Reload Stiffness (Eur)

The change of the unload-reload stiffness E_{ur} while changing the bioslurry weight to the total sample weight is presented in **Figure (6.12)**. The figure reveals an increase in E_{ur} for treated samples as opposed to untreated ones. The E_{ur} for untreated samples equals 3000 kPa and increases to 3800 kPa for 5% and 10% bioslurry and further increases to 4400 kPa for 15% and

20% bioslurry. The increasing percentage is 25% for the 5% and 10% bioslurry and 45% increase for the 15% and 20% bioslurry. The precipitation of the calcite crystals in the soil pores could justify the increase in the E_{ur} , which decreases the total volume of pores in the soil and resists the soil tendency to expand/dilate when the applied stresses decrease.





6.4 Phase (3): Effect of Cementation Solution Volume

Injecting more cementation solution volume to the soil mass is expected to further enhance sandy soils' geotechnical properties, as discussed in many studies (Cheng et al., 2013; Cheng & Shahin, 2016). However, treated Muskeg soils' behaviour with the urease active bioslurry approach under different cementations solutions volumes is not discussed in the literature. Therefore, this phase addresses the effect of injecting different cementation solution volumes in the soil mass on the compressibility, stiffness and shear strength properties of Muskeg soils.

6.4.1 Compressibility Properties

Changing the volume of the injected cementation solution in the soil to the soil pores' volume $V_{cementation} / V_{pores}$ affects the compressibility properties of Muskeg soil, as shown in **Figure (6.13)**. The figure reveals that the compression index C_c before the unload-reload cycle drops from 3.2 for untreated samples to 2 for $V_{cementation} / V_{pores}$ equals one with a decreasing percentage of 35%. A similar drop occurs for the C_c after the unload-reload cycle as it drops from 2.8 to 2. The swelling index decreases from 0.34 in the untreated sample to 0.18 when $V_{cementation} / V_{pores}$ equals one.

The figure shows that when $V_{cementation} / V_{pores}$ is more than one, there is no remarkable improvement in the interpreted C_c before or after the unload-reload cycle as well as the C_s . This behaviour is different from the behaviour induced for sandy soils, where adding more cementation solution would increase the UCS values due to the precipitation of the large calcite crystals around the soil particle and particle to particle contact (Cheng & Shahin 2016). The main reason for this difference is Muskeg soils' nature, as Muskeg soils are characterized by their high organic content and low mineral content. That is why even if a large amount of calcite would precipitate in the soil mass when adding more cementation solution, they will not cover influential positions in the soil mass to affect its C_c and C_s .

Figure (6.14) shows the effect of changing the V_{cementation} / V_{pores} ratio on the secondary compression index C_a. The same behaviour observed for the change of C_a when changing the bioslurry weight (**Figure 6.11**) is observed when changing the V_{cementation} / V_{pores}. The C_a increases to reach a plateau when the applied loading stress equals 100 kPa and applied re-loading stress equals 200 kPa. The ratio V_{cementation} / V_{pores} equals four shows the lowest value for C_a compared to other ratios as it shows a value of 0.17 for 100 kPa loading stress and 0.13 for 200 kPa re-loading stress.



Figure 6.13 Effect of the cementation solution volume on the compression index C_c and swelling index C_s of treated Muskeg soils.



Figure 6.14 Effect of the cementation solution volume on the secondary compression index C_{α} during; (a) loading stages, and (b) re-loading stages.

6.4.2 Stiffness Properties

Precipitating calcite particles in the soil pores would increase the soil stiffness during the unloadreload state. **Figure (6.15)** presents the change in the unload-reload stiffness E_{ur} of Muskeg soil due to the injection of different cementation solution volumes. The figure reveals an increase in E_{ur} when the cementation solution volume equals the pores' volume. The E_{ur} of untreated sample is 3075 kPa and increases to 3685 kPa for the $V_{cementation}$ / V_{pores} ratio equals one with a 20% increasing percentage. The E_{ur} enhancement is stable for the $V_{cementation}$ / V_{pores} ratio up to three. For the $V_{cementation}$ / V_{pores} ratio of four, the E_{ur} increases to 4200 kPa with an increasing parentage of 35% compared to the untreated sample. This behaviour reflects the soil response to a large amount of precipitated calcite in the soil pores at high $V_{cementation}$ / V_{pores} values.

Figure (6.16) presents the change in constrained modulus M when changing the V_{cementation} / V_{pores} between 1 and 4 compared to the untreated sample. **Figure (6.16a)** shows that the soil mass before the unload-reload cycle exhibits the same constrained modulus but at low strain when treated with a cementation solution of a V_{cementation} / V_{pores} ratio equals four. The same behaviour is observed after the unload-reload cycle, as shown in **Figure (6.16b)**, with a slight increase in the interpreted M from 1550 kPa for the untreated sample to 1680 kPa for the treated sample at 200 kPa re-loading stress.



Figure 6.15 Influence of changing the cementation solution volume on the interpreted unload-reload stiffness E_{ur} .



Figure 6.16 Effect of changing the cementation solution volume on the constrained modulus M; (a) before the unload-reload cycle, and (b) after the unload-reload cycle.

6.4.3 Shear Strength Properties

When the $V_{cementation}$ / V_{pores} equals four, the different compressibility properties were enhanced: it led to small compression index C_c, small secondary compression index C_a, large constrained modulus M at minor strains, and a significant increase in the unload-reload stiffness E_{ur}. Therefore, the V_{cementation} / V_{pores} ratio of four is selected to show the effect of the calcite precipitation on the shear strength properties of Muskeg soils, as shown in **Figure (6.17)**.

The figure reveals that the measured shear stress at different axial stresses while increasing horizontal displacement is higher for treated samples than the untreated ones. The untreated sample behaves similarly to loose soils as it exhibits a continuous increase in the measured shear stress till the maximum applied horizontal displacement. However, it shows a dense soil behaviour for treated samples without a peak as the shear stress reaches a plateau at 6 mm horizontal displacement. This could be attributed to the loose state of the untreated Muskeg samples and treated sample's dense state due to the calcite crystals' precipitation in the soil pores.



Figure 6.17 Shear stress evolution while changing the horizontal displacement under various axial stresses.

Horizontal displacement (mm)	Cohesion (kPa)		Angle of internal friction (degree)	
	Untreated	Treated	Untreated	Treated
2	0	0	26.7	34.3
10	8.5	8.4	32.5	35.7

Table 6.2 Shear strength parameters of untreated and treated samples with cementation solution

 volume equals four times the pores' volume at different horizontal displacement

Table (6.2) presents the calculated shear strength parameter for untreated and treated samples at two different horizontal displacements. The table shows a non-cohesion behaviour at small-displacement (2 mm) with an increase in the internal angle of shearing resistance due to the treatment process from 26.7° to 34.3° with an increasing percentage of 28%. At high displacement, the cohesion intercept increases to 8.5 kPa for treated and untreated soils and angle of internal friction increases from 32.5° to 35.7°. The minor increase in the induced internal angle of friction between the 2 mm and 10 mm horizontal displacements.

6.5 Summary

This chapter showed that the stabilization of Muskeg soil using the emerging environmentallyfriendly technique, Microbially Induced Calcite Precipitation MICP by the urease active bioslurry approach, was promising as it decreased the soil compressibility and increased the soil shear strength.

The bioslurry concentration's effect on the compressibility properties of Muskeg soil was thoroughly investigated by changing the bioslurry concentration from zero percent for the untreated sample to 1 M/L with an increment of 0.2 M/L bioslurry. The 0.4 M/L bioslurry was determined as the optimum concentration as no statistically significant difference in the measured compressibility properties was achieved when the bioslurry concentration was more than 0.4 M/L. The 0.4 M/L bioslurry decreased the compression index C_c , coefficient of volume compressibility
m_v , secondary compression index C_{α} by 50% compared to the untreated sample. Moreover, the constrained modulus M is increased by 18%, and the unload-reload stiffness E_{ur} increased by 45% due to the treatment with 0.4 M/L bioslurry concentration. The microstructure analysis justified why there was no further improvement in the compressibility properties when the bioslurry concentration was more than 0.4M/L by the settlement of the CaCO₃ percentage in the soil mass. The only change was the phase transformation from the vaterite to calcite crystal.

Changing the bioslurry weights from 5% to 20% showed no significant effect on the compression index C_c and swelling index C_s , but the secondary compression index C_{α} at 20% bioslurry weight decreased by 45 %, and the unload-reload stiffness E_{ur} increased by 45%.

The effect of injecting multiple cementation solutions was studied as well. When $V_{cementation}$ / V_{pores} was four, there was a significant improvement in the secondary compression index C_{α} and the unload-reload stiffness E_{ur} but not a remarkable improvement in the compression index C_c or the constrained modulus M. Moreover, it was noticed that the shear strength properties were enhanced due to the precipitation of the calcite and vaterite crystals in the soil mass, and this enhancement was more remarkable at small horizontal displacement before the breakage of the calcite crystals at high values of the horizontal displacement.

Chapter 7: Results of Muskeg Soil Stabilization Using Sand and Bioslurry

7.1 Background

The testing program presented in chapter (5) for Muskeg soil stabilization using sand and bioslurry simultaneously showed two parts to check for the effect of changing some parameters on the behaviour of treated samples. This chapter will present the results of these two parts. In Part (1), the results for the effect of changing the sand percentage would be discussed. The results of the effect of the effect of changing the solution volume would be demonstrated in Part (2).

It is worth mentioning that the soil used for Ppartarts (1) and (2) was the soil from Depth Zone (B). The index properties in this Depth Zone were presented in Chapter (4) in Table (4.1).

7.2 Part (1): Effect of Sand Percentage

7.2.1 e - log σ' Curve

Muskeg soils exhibit high compressibility when subjected to external loads due to their high initial void ratio and water content. Stabilization of Muskeg soil with sand and bioslurry helps fill soil's voids with the added sand particles and precipitated calcite due to the MICP process. This, in turn, helps decrease the initial void ratio and initial water content, which would improve the compressibility properties and decrease the consolidation settlement.

Figure (7.1) presents the change in the void ratio with the applied consolidation stresses of untreated and treated samples with different sand percentages while fixing the bioslurry percentage to 10% of the total sample weight. The figure reveals a downward shifting of the e log σ ' curve due to the increasing percentage of the added sand. Although three samples are tested for each combination, only one sample's results are displayed to ensure the figure's readability. Moreover, the figure shows a dramatic decrease in the treated samples' void ratio compared to the untreated one. At minor stress of 6.25 kPa, the untreated sample's void ratio is 7.75 and decreases to 3.4, 2.7, 2.4 for the 10%, 15%, and 20% sand, respectively. This indicates a decrease in the void ratio compared to the untreated samples by 57%, 65%, and 70%, respectively.



Figure 7.1 e - log σ ' curve showing the effect of changing the sand percentage.

To check the soil's performance due to stabilization using sand only as a filling material, the figure also depicts the change in the e - log σ' curve. The figure shows the significance of coupling sand and MICP technique on the soil behaviour by the remarkable decrease in the void ratio. For instance, the void ratio at 25 kPa for treated sample with 20% sand only is 3 and decreases to 2.4 when sand is coupled with the precipitated calcite from the MICP process with a decreasing percentage of 20%.

It is noted that the change in the first portion of the e -log σ ' curve follows a linear pattern for the untreated samples. This could be attributed to the soil's disturbance. However, the added sand and precipitated calcite show an initial curved portion for the e - log σ ' curve representing the soil's recompression zone.

7.2.2 Compression Index (C_c)

The compression index is a significant factor in the settlement calculations due to the consolidation process. The compression index C_c change, due to the soil stabilization using sand only and when the sand is coupled with the precipitated calcite from the MICP, is presented in **Figure (7.2).** Muskeg soil is distinguished by the decrease in the compression index C_c after the unload-reload cycle compared to the C_c before the unload-reload cycle, as reported by Ajlouni (2000). That is why the figure distinguishes between the compression index calculated before and after the unload-reload cycle.

The figure shows no difference in the calculated C_c before and after the unload-reload cycle due to stabilization with sand only or sand and precipitated calcite. This could be attributed to the presentation of more minerals (sand and calcite) to the stabilized soil mass and untreated samples' organic nature.

Improving the C_c of the soil mass would significantly affect the soil settlement. The figure shows a decrease in the calculated C_c due to stabilization with sand and precipitated calcite. The C_c decreases from 2.85 for untreated samples to 1.35, 1.05, 0.95 for treated samples with 10%, 15%, and 20% sand percentages, respectively. This is reflected as a decreasing percentage of 50%, 60%, and 67%, respectively.

Comparing the C_c of the stabilized soil with sand and precipitated calcite from the MICP process with stabilized soil with sand only reveals a notable improvement in the compressibility properties. For example, the stabilized soil with 20% sand only (without bioslurry) shows a C_c of 1.28. However, the treated soil while coupling the 10% bioslurry and 20% sand shows a C_c of 0.95. The treated soil by coupling the sand and bioslurry reveals a decrease in the C_c by 25% compared to the treated soil with sand only. This finding confirms the C_c's significant improvement when Muskeg soils are treated with sand and bioslurry simultaneously instead of sand only.

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Figure 7.2 Change in the compression index Cc due to the change of the sand percentage

To check the statistical significance of changing the sand percentage on the calculated C_c for the stabilized soil with sand and bioslurry simultaneously, one-way ANOVA (Analysis Of Variance) is conducted. The ANOVA reveals a P-value of 1.4×10^{-6} , reflecting the statistically significant difference between the various observations (including the untreated sample). To determine which observation's mean is different from other observations, the Post Hoc test using the Least Significance Difference LSD method is conducted. The Post Hoc test shows a statistically significant difference between the untreated and treated samples with different sand percentages with a P-value less than 0.05. However, the test reveals no statistically significant difference in the calculated C_c when the sand percentage increases from 15% to 20%. The flat horizontal line in the figure between the 15% and 20% sand reflects the same behaviour.

7.2.3 Swelling Index (C_s)

Muskeg soil can swell when it is unloaded. The swelling index C_s is determined by the slope of the line that connects the two endpoints in the unload-reload cycle in the e - log σ ' curve. **Figure** (7.3) presents the change of swelling index C_s from the untreated to treated samples with sand

and bioslurry while changing the sand percentage and fixing the bioslurry percentage to 10%. The error bars in the figure represent the standard deviation of the calculated C_s of the three tested samples at each sand percentage. The figure shows a decreasing trend for the calculated C_s due to the sand percentage increase. The C_s decreases from 0.33 for untreated samples to 0.11, 0.08, 0.06 for treated samples with 10%, 15%, and 20% sand, respectively. The 20% sand percentage shows a decreasing percentage of C_s by 80%. This behaviour could be attributed to the filling of the soil pores by sand and precipitated calcite crystals due to the MICP process. C_s/C_c ratio equals 0.12 for the untreated sample, close to the 0.1 ratio in mineral soils, and decreases to 0.07 for the treated ones with 20% sand and 10% bioslurry. This could be attributed to the significant decrease in the C_s for the treated soil with 20% sand and 10% bioslurry.



Figure 7.3 Swelling index C_s change when stabilizing the Muskeg soil with different percentages of sand in addition to the 10% bioslurry.

The one-way ANOVA reveals a P-value of 4.87×10^{-6} , reflecting the difference of at least one mean of the observations. The Post Hoc test using the Least Significant Difference LSD method shows a statistically significant difference in the measured C_s between the untreated and all treated samples. However, the test reveals no statistically significant difference when the sand percentage is more than 10%. It reveals a P-value of 0.311 when the sand percentage increases from 10% to 15%. The P-value is 0.095 when the sand percentage increases from 10% to 20%. The significant difference in the P-value could be attributed to the notable large standard deviation in the C_s for the treated samples with 10% sand.

7.2.4 Coefficient of Volume Compressibility (m_v)

The coefficient of volume compressibility m_v is considered an essential factor in the settlement determination due to the consolidation process. It is also considered a stress-dependent parameter as it changes with the applied stresses. **Figure (7.4)** shows the change in calculated m_v for untreated and treated samples with sand and bioslurry. The figure differentiates between the calculated m_v during the loading and reloading stages.



Figure 7.4 Change in the coefficient of volume compressibility m_v at different sand percentages; during; (a) loading stages, and (b) re-loading stages.

Figure (7.4a) presents the decreasing trend for m_v during the loading stages. The figure shows a significant decrease in the calculated m_v due to the stabilization process. At a minor applied average stress of 50 kPa, the m_v decreases from $4x10^{-4}$ (1/kPa) for the untreated sample to 2 $x10^{-4}$ (1/kPa) for treated samples with 10% sand and reaches down to 1 $x10^{-4}$ (1/kPa) for treated samples with 20% sand. This reveals a decreasing percentage of 50% and 75% for treated

samples with 10% and 20% sand, respectively. The two-way ANOVA reveals a statistically significant difference between the m_v calculated for untreated and other treated samples. However, the Post Hoc test using the least significant difference method reveals no statistically significant difference due to the increase of the sand percentage from 10% to 20%.

The change in the m_v during reloading stages is presented in **Figure (7.4b)**. The figure shows a plateau in the calculated m_v at high-stress level of 225 kPa due to the slight change in the corresponding void ratio at these high-stress levels. **Figures (7.4a) and (7.4b)** reveal the importance of preloading Muskeg soil either in the untreated or treated states as it significantly decreases the m_v .

7.2.5 Secondary Compression Index (C_α)

Muskeg soils are known for their rapid primary consolidation because the pore water pressure dissipates quickly due to the high initial void ratio. Therefore, most of their consolidation settlement is attributed to the high secondary compression. The secondary compression index C_{α} is considered a stress-dependent property as it changes with the applied stress. The treatment of Muskeg soil using sand and precipitated calcite from the MICP process enhances the secondary compression, as presented in **Figure (7.5)**.

Figure (7.5a) shows the change in C_{α} during the loading stages in the consolidation test for untreated and treated samples with different sand percentages when the bioslurry weight is fixed to 10% of the total sample weight. The figure depicts an increase in the calculated C_{α} up to a stress of 50 kPa. No further change is noted for C_{α} when the applied loading stress is more than 50 kPa. The figure shows a decrease in the calculated C_{α} when the sand percentage increases. At a high-stress level, the treated soil with 20% sand and 10% bioslurry shows a C_{α} value of 0.08 compared to 0.2 for untreated soil with a decreasing percentage of 60%. The two-way ANOVA and Post Hoc tests reveal no statistically significant difference in the calculated C_{α} when the sand percentage is higher than 10%. At 100 kPa stress level, the C_{α}/C_{c} ratio is 0.07 for untreated samples and increases to 0.08 for treated samples with 20% sand and 10% bioslurry. Despite the improvement in the calculated C_{c} and C_{α} due to the stabilization process with 20% sand and 10% bioslurry, the C_{α}/C_{c} ratio is higher than the untreated sample.





The change in the calculated C_{α} due to the stabilization with different sand percentages during the reloading stages is presented in **Figure (7.5b)**. The Figure reveals a similar trend as the one noted in Figure (7.5a) (i.e. the C_{α} increases while the reloading stress increases and stabilizes at high re-loading stresses). At 200 kPa, the C_{α} decreases from 0.2 for untreated samples to 0.06 for treated samples with 20% sand and 10% bioslurry with a decreasing percentage of 70%.

7.2.6 Unload-reload Stiffness (Eur)

The unload-reload stiffness E_{ur} is essential in numerical simulations for many applications that involve the unloading of the soil mass, such as excavations. The E_{ur} could be calculated from the slope of the unload-reload cycle in the stress-strain curve produced from the consolidation test. **Figure (7.6)** illustrates the change in the calculated E_{ur} due to Muskeg soils' stabilization with sand only and when coupling the sand with bioslurry. The figure shows a better improvement in the calculated E_{ur} when the soil is stabilized with sand and bioslurry simultaneously than improvement when using sand only. This is mainly attributed to the precipitation of the calcite crystals in the soil pores, which increases the soil stiffness. The stabilized soil with 20% sand and 10% bioslurry shows a E_{ur} value of 4650 kPa compared to 3600 kPa for the stabilized soil with sand only. In other words, the MICP process increases E_{ur} by around 30% more than the achieved E_{ur} from stabilization with sand only.



Figure 7.6 Influence of changing the sand percentage on the unload-reload stiffness E_{ur} when the sand is coupled with bioslurry and when no bioslurry is added.

For the stabilized soil with both sand and bioslurry, the figure reveals an increasing trend for the E_{ur} while increasing the sand percentage. The E_{ur} for untreated samples is 3050 kPa and

increases to 4650 kPa for treated samples with 20% sand and 10% bioslurry with a 50% increasing percentage. The one-way ANOVA reveals a P-Value of 0.003, reflecting the statistically significant difference in E_{ur} between untreated and treated samples. The Post Hoc test with the Least Significant Difference LSD method shows no statistically significant difference in the calculated E_{ur} when the sand percentage is more than 10%. When the sand percentage increases from 10% to 15%, the difference of E_{ur} shows a P-value of 0.665. This P-value decreases to 0.12 when the sand percentage increases from 15% to 20%.

7.2.7 Constrained Modulus (M)

The consolidation test allows for determining the constrained modulus M as it measures the soil's axial strain when subjected to different loads and constrained from lateral movement. **Figure (7.7)** shows the effect of stabilizing Muskeg soil with sand and bioslurry together on the behaviour of the constrained modulus M.

Figure (7.7a) shows the change in M before the unload-reload cycle. The figure shows an increase in M due to the increased applied stresses attributed to the void ratio decrease. Moreover, the figure reveals a decrease in the induced axial strain of treated samples compared to the untreated ones. The untreated sample shows a maximum axial strain of 0.4 compared to 0.26 for the treated samples with 20% sand and 10% bioslurry, which is a 35% decreasing percentage. The addition of sand as a filling material and precipitated calcite in the soil pores through the MICP process increases the M and decreases the corresponding strain. For instance, the treated samples with 20% sand and 10% bioslurry show a maximum M of 1050 kPa compared to 825 kPa for the untreated samples. The two-way ANOVA and Post Hoc tests reveal no statistically significant difference in the calculated M when the sand percentage is more than 10%.



Figure 7.7 Effect of changing sand percentage in the treatment process on the calculated constrained modulus M; (a) before the unload-reload cycle, and (b) after the unload-reload cycle.

The unload-reload cycle's effect on the constrained modulus is significant, as shown in **Figure (7.7b).** The figure reveals an increase in the constrained modulus M from 825 kPa for the untreated samples before the unload-reload cycle to 1500 kPa after the unload-reload cycle, around 80% increase. The same percentage is observed for treated samples with different sand percentages while fixing the bioslurry percentage to 10%. Moreover, Muskeg soil's stabilization using sand and bioslurry shows a decrease in the induced strain after the unload-reload cycle.

7.2.8 Calcium Carbonate (CaCO₃) Content

The uniform distribution of the precipitated calcite in the soil mass due to the MICP process dramatically affects the soil's mechanical behaviour. To confirm the uniformity of the CaCO₃ distribution, the CaCO₃ is measured four times for each sample at four different quadrants. The average measurements of the calcium carbonate content and standard deviation of these measurements are presented in **Figure (7.8**). The slight standard deviation in each measurement confirms the even distribution of the calcite crystals in the soil mass. The figure reveals the precipitation of the calcite crystals by an average of 32% at the different sand percentages. This behaviour is expected as the calcite precipitation process primarily relies on the bioslurry concentration and cementation solution's injected volume. This emphasizes that the soil mixture behaviour's significant improvements at high sand percentage are mainly attributed to the sand material's presence, not the precipitation of more calcite crystals.





7.2.9 Microstructure Variations Before and After Treatment

The microstructure analysis of untreated and treated samples helps figure out the behaviour in the macroscale.

The XRD patterns for the untreated and treated samples with different sand percentages while fixing the bioslurry percentage at 10% are presented in **Figure (7.9)**. The figure shows the untreated sample's amorphous nature through the noticeable hump when 20 ranges between 10° to 30°. However, for the three treated samples, no hump is noticed, which reveals the presence of more crystals in the soil mass due to the addition of the quartz crystals in the sandy soil and precipitated calcite and vaterite crystals from the MICP process. The figure reveals no significant difference when the sand percentage changes from 10% to 20%. The quantification of the various crystals is calculated through the Rietveld analysis and presented in **Table (7.1)**. The table confirms the untreated sample's amorphous nature and transformation to crystallized nature for treated samples. Moreover, the table reveals the constant percent of calcite and vaterite summation when the sample is treated with different sand percentages.



Figure 7.9 XRD patterns for untreated and treated samples with 10% bioslurry while changing the added sand percentage.

The EDS results for the untreated sample reveal that carbon and oxygen are the two dominant elements in the soil mass. For treated samples with 20% sand and 10% bioslurry, the EDS results reflect the calcium formation in the soil and silicate beside the carbon and oxygen. The SEM images for the untreated and treated samples with 20% sand and 10% bioslurry are

presented in **Figure (7.10).** The figure shows the amorphous nature of the untreated sample and precipitated calcite and vaterite crystals within the soil mass for the treated sample. **Figure (7.10b)** shows the sand particle's location, and some of the vaterite and crystals remained after taking the sand particle out. These precipitated crystals help increase the cohesion through the soil mass, reflected in the direct shear results in the next section.

Table 7.1 Chemical composition quantification after Rietveld analysis of untreated and treated

	Untreated sample	10% Sand + 10% Bioslurry	15% Sand + 10% Bioslurry	20% Sand + 10%Bioslurry
Calcite (%)	0	30	28	35
Vaterite (%)	0	50	49	44
Quartz (%)	10	5	6	5
Amorphous + Background (%)	80	9	10	8
Andesine (%)	7	4	5	6
Clinochlore (%)	1	1	1	1
Illite/Muscovite 2M1 (%)	2	1	1	1
Calcite + Vaterite	0	79	77	79

samples with sand and bioslurry simultaneously





Figure 7.10 SEM images for **(a)** untreated Muskeg sample, and **(b)** treated Muskeg sample with 20% sand and 10% bioslurry.

7.3 Part (2): Effect of Cementation Solution Volume

For stabilized sandy soils with the MICP technique, injecting multiple cementation solutions produces more calcite crystals in the soil pores. If the calcite crystals occupy effective locations at the particle to particle contacts, they will enhance the soil's physical properties (Cheng et al., 2013; Cheng & Shahin, 2016). In this part, the effect of Muskeg soil treatment, when the sand and bioslurry weights of the total sample weight are 20% and 10%, respectively, on the physical properties, is discussed due to the change of the cementation solution volume.

7.3.1 Compressibility Properties

The effect of increasing the cementation solution volume as a percentage of the pores' volume on the compression index C_c and swelling index C_s is presented in **Figure (7.11)**. The figure depicts a dramatic decrease in both the compression and swelling indices when the cementation solution volume is the same as the pores' volume. For $V_{cementation} / V_{pores}$ more than 1, no further improvement in C_c is noticed. The figure shows no change in the calculated C_c before and after the unload/reload cycle at different cementation solution volumes, which is consistent with the initial findings in Part (1). C_c decreases from 3.1 for the untreated sample to 0.8 for the treated sample with 20% sand and 10% bioslurry when $V_{cementation} / V_{pores}$ equals one. This represents a decreasing percentage of 75%.

The swelling index C_s decreases from 0.34 for untreated samples to 0.08 at V_{cementation} / V_{pores} equals one and decreases to 0.05 when V_{cementation} / V_{pores} equals four. This could be attributed to the ineffective positions in the soil mass that the newly precipitated calcite occupy that is why it does not contribute to the enhancement of the compression index C_c but improves the swelling index C_s. The treated samples reveal a C_s/C_c ratio equals to 0.05 at the highest V_{cementation} / V_{pores} ratio compared to 0.11 for untreated samples.



Figure 7.11 Change in the compression index C_c and swelling index C_s due to the increase in the cementation solution volume.

The secondary compression index C_{α} reveals no significant improvement when changing the V_{cementation} / V_{pores} ratio. C_{α} for treated samples with different V_{cementation} / V_{pores} is the same and equals 0.08 compared to 0.25 for untreated samples. This exhibits a decreasing percentage of 70%. The C_{α}/C_c ratio for treated samples is the same as the C_{α}/C_c of untreated samples and equals 0.08. The treatment process decreases both C_{α} and C_c. That is why the C_{α}/C_c ratio does not change.

7.3.2 Stiffness Properties

The calcite precipitation inside the soil mass due to injecting multiple cementation solution volumes affects the soil stiffness.

The change in the constrained modulus M and corresponding strain before the unloadreload cycle is presented in **Figure (7.12a)**. At the maximum applied stress (150 kPa), the figure reveals that when the V_{cementation} / V_{pores} is four, M increases to 1200 kPa compared to 800 kPa for the untreated sample with an increasing percentage of 50%. Moreover, this increase in the constrained modulus is conjugated with a decrease in the strain level such that the induced strain decreases from 0.4 for the untreated sample to 0.2 for treated samples with a 50% decreasing percentage. Similar behaviour for the constrained modulus change is noticed after the unload-reload cycle, as shown in **Figure (7.12b)**. The figure shows an increase from 1500 kPa for untreated samples to 2000 kPa for treated samples with different V_{cementation} / V_{pores} ratios.

Figure (7.13) shows the change in the unload-reload stiffness E_{ur} while injecting different cementation solution volumes. The figure reveals an increase in the E_{ur} from 3000 kPa to around 4500 kPa for treated samples with cementation solution volume equals one or twice the pores' volume with a 50% increasing percentage. At high $V_{cementation}$ / V_{pores} ratios (three or four), the E_{ur} increases to 5200 kPa with an increasing percentage of 75 % compared to untreated samples.



Figure 7.12 Change in the constrained modulus M due to the variation of the cementation solution volume; (a) before the unload-reload cycle, and (b) after the unload-reload cycle.



Figure 7.13 Change in the unload-reload stiffness E_{ur} due to the injection of multiple cementation solution volumes.

7.3.3 Shear Strength Properties

Changing the cementation solution volume does not significantly affect the soil's compressibility properties but increases the soil stiffness. When the V_{cementation} / V_{pores} ratio equals four, the treated soil's behaviour during the direct shear test is presented in **Figure (7.14)** compared to the untreated sample. The figure shows an increase in the induced shear stress under various axial stresses when the sample is treated. Both treated and untreated samples show a softening behaviour such that the shear stress continues to increase while the applied horizontal displacement increases. This behaviour is more similar to the loose soil behaviour. Although the soil pores are filled with sand and calcite precipitated during the MICP process, the pores' volume is still significant. That is why the soil is exhibiting loose behaviour.

Table 7.2 Shear strength parameters of untreated and treated samples with cementation solution

 volume equals four times the pores' volume at different horizontal displacement

Horizontal displacement (mm)	Cohesion (kPa)		Angle of internal friction (degree)	
	Untreated	Treated	Untreated	Treated
2	0	6.5	26.7	27.5
10	8.5	13	32.5	33.5



Figure 7.14 Effect of cementation solution volume on the induced shear stresses in the direct shear test DST.

The shear strength parameters are determined at 2 mm and 10 mm horizontal displacements, representing the small and large displacements, respectively, as tabulated in **Table (7.2)**. The treatment process increases the cohesion intercept and angle of internal friction. For small displacement, the cohesion intercept increases from zero to 6.5 kPa and internal angle of shearing resistance increases from 26.7° to 27.5°. However, at large displacement, the internal angle of shearing resistance increases from 32.5° to 33.5° and cohesion intercept increases from 8.5 kPa to 13 kPa. The noticed improvement in the cohesion intercept could be attributed to the precipitated calcite as it helps bond the sand particles to the surrounding amorphous texture in Muskeg soil as presented in the SEM images (**Figure 7.10**).

7.4 Summary

This study investigated the stabilization of Muskeg soils using two additives simultaneously; the sandy soil as a filling material and the calcite crystals precipitated from the Microbially Induced Calcite Precipitation technique by the urease active bioslurry approach.

The effect of changing the sand percentage while fixing the bioslurry percentage to 10% on the soil's compressibility and stiffness properties was studied. Increasing the sand percentage up to 20% by weight enhanced both the compressibility and stiffness properties for Muskeg soils. However, the improvement in these properties was not statistically significant than the achieved improvement when the sand percentage was 10%.

When the sand percentage was 20%, the compressibility properties improved, compared to the untreated samples such that (i) the initial void ratio decreased from 2.4 to 7.75 by 70%, (ii) compression index C_c decreased from 2.85 to 0.95 by 70%, (iii) the swelling index C_s fell from 0.33 to 0.06 by 80%, (iv) The coefficient of volume compressibility m_v decreased from 4x10⁻⁴ to 1x10⁻⁴ (1/kPa) by 75%, and (v) secondary compression index C_a fell from 0.2 to 0.08 by 60%. The stiffness properties were improved as well. The unload-reload stiffness E_{ur} increased from 3050 to 4650 by 50%, and the constrained modulus M increased from 825 kPa to 1050 by 50%. The Microstructure analysis revealed no further formation of calcite or vaterite crystals in the soil mass when the sand percentage was more than 10%. Therefore, any further improvement in the soil behaviour would solely be attributed to the sand particles.

The effect of injecting multiple cementation solutions in the soil mass to stimulate the bioslurry to form more calcite or vaterite crystals was investigated. The results showed that when the cementation solution volume was more than the pores' volume, no further enhancement happened to the compression index C_c , but the swelling index C_s decreased. The constrained modulus M was noted to be fixed when injecting more than one pore's volume but exhibited less strain. The unload/reload stiffness E_{ur} increased to 5000 kPa when the injected volume was four

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times the pores' volume. The increase in the cohesion intercept reflected the improvement in the shear strength parameters, and no significant improvement was noticed in the angle of shearing resistance.

This chapter concluded that the stabilization of Muskeg soil by coupling the sand and MICP technique would give satisfactory results when the sand percentage equalled 10%, the bioslurry percentage equalled 10%, and the cementation solution volume was the same as the pores' volume.

Chapter 8: Conclusions

This chapter summarizes the conducted research in this thesis, the major conclusions, the significance of contribution, the limitations of this study, and future studies' recommendations.

8.1 Summary

As Muskeg soils are known for their problematic nature due to their high compressibility and low shear strength, thorough investigation to understand their behaviour and interpret their properties in the early and final design stages is mandatory. Moreover, constructing on these soils is not favourable from the geotechnical engineering perspective as they usually exhibit high settlement when loaded, which could lead to severe problems for any construction. Therefore, these soils are either replaced with new competent soil or stabilized to enhance their properties.

With this impetus, this research was conducted to address the problems associated with construction on Muskeg soils. In the first phase of this research, some correlations were developed to interpret the index and compressibility properties in the early design stages by the simple measurements of the water and organic contents. Moreover, detailed and thorough field and laboratory investigations were conducted for a soil deposit retrieved from Bolivar Park, Surrey, British Columbia, Canada. The purpose of these investigations was to give more insights into the Muskeg soils' behaviour in the field and compare it with the laboratory's behaviour. Furthermore, the applicability of the Ball Penetration Test BPT to characterize Muskeg soils properties was studied.

The second phase of this research included stabilizing Muskeg soil with the Microbially Induced Calcite Precipitation MICP technique, the evolving environmentally-friendly technique, by the urease active bioslurry approach. The stabilization was conducted at two levels. In the first

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level, Muskeg soils were stabilized with bioslurry only. In the second level, sandy soil from a local source was added to the bioslurry to further enhance the Muskeg soils' properties.

8.2 Conclusions

The following conclusions were deduced from this study:

- New correlations were proposed to interpret the Muskeg soil index and compressibility parameters as a function of the easily measured parameters: water and organic contents.
- The proposed correlations showed that the void ratio is better estimated based on the water content. The unit weight and specific gravity showed a better correlation with the organic content. Whenever the water content increases, more than 500 %, the bulk density tended to settle around 1 gm/cm³, and specific gravity around 1.5.
- This study revealed that the proposed minimum limit by Landva & La Rochelle (1983) for determining the specific gravity G_s as a function of the organic content was not in good agreement with the 95% confidence interval proposed in this study. The 95% confidence interval covered many data that the minimum limit of Landva & La Rochelle (1983) did not cover.
- The lower bound proposed by Macfarlane (1969) for the determination of the compression index C_c as a function of the water content w_c was not applicable for the full range of the organic content. Besides, a high scatter of the compression index was noticed when the organic content was used for the correlations. Therefore, this study recommended, for the tentative estimate of the compression index, using the proposed equation of the water content instead.
- It was not easy to correlate the shear strength parameters as a function of water or organic contents without considering the fibres' presence and textures. However, the undrained shear strength parameter S_u and effective cohesion c' showed a decreasing trend while

the water content increased. Generally, organic soils' cohesion was small and ranged between 2 to 14 kPa for both the drained and undrained conditions.

- A natural organic soil (peat) deposit located in a research test site in British Columbia,
 Canada, was characterized through a comprehensive geotechnical investigation involving a wide range of field and laboratory tests.
- The initial void ratio observed in the remoulded specimens was smaller than those derived from undisturbed organic soil testing. The results also showed that the behavioural trends of increasing stiffness with increasing effective confining stress in mineral soils could not be extended to organic soils due to their inherently variable organic/fibre contents and relatively complex heterogeneities.
- Reasonable agreement for the correlation between (SCPTu) and S_u could be obtained if a value of 8 was used for the commonly used N_{kt} factor. The results also showed that the N_{kt} value of 15 proposed for fine-grained mineral soils would not be suitable for organic soils.
- An average N_{ball} factor of 9 was found to provide a suitable correlation between the net tip resistance (q_{b-net}) from BPT tests with eVST-based S_u.
- G_{max} could be approximately obtainable by direct multiplication of the S_u by a factor k_g of about 40.
- G_{max} could be approximated by direct multiplication of the value of q_{b-net} by a factor C_g of about 4.5.
- The ball penetration test's potential to serve as a valuable tool to characterize natural organic soil deposits' strength and stiffness properties was concluded.
- Muskeg soils' stabilization using the emerging environmentally-friendly technique,
 Microbially Induced Calcite Precipitation MICP by the urease active bioslurry approach,

was promising as it decreased the soil compressibility and increased the soil shear strength.

- The 0.4 Mole/Litre bioslurry was determined as the optimum bioslurry concentration.
- The 0.4 M/L bioslurry decreased the compression index C_c , the coefficient of volume compressibility m_v , and secondary compression index C_α by 50% compared to the untreated sample. Moreover, the constrained modulus M was increased by 18%, and unload-reload stiffness increased by 45%.
- The microstructure analysis justified why there was no further improvement in the compressibility properties of treated Muskeg soils when the bioslurry concentration was more than 0.4 M/L by the extra amount of the precipitated CaCO₃ in the soil mass. The only change due to the increase in the bioslurry concentration was the phase transformation from vaterite to calcite crystals.
- Varying the bioslurry weight as a percentage from the total sample weight showed no significant effect on the compression index C_c and swelling index C_s, but the secondary compression index C_α at 20% bioslurry weight decreased by 45 %, and unload-reload stiffness E_{ur} increased by 45%.
- The effect of injecting multiple cementation solutions was studied as well. When $V_{cementation}$ / V_{pores} equals four; there was a significant improvement in the secondary compression index C_{α} and unload-reload stiffness E_{ur} but not a remarkable improvement in the compression index C_{c} or the constrained modulus M.
- Shear strength properties were enhanced due to the precipitation of the calcite and vaterite crystals in the soil mass. This enhancement was remarkable at small horizontal displacement before the breakage of the calcite crystals at high values of the horizontal displacement.

- Adding sand to the bioslurry helped in further enhancing the compressibility properties of Muskeg soils.
- When stabilizing Muskeg soil with 10% bioslurry and 20% sand, the compressibility properties were enhanced compared to the untreated samples such that (i) the initial void ratio decreased from 2.4 to 7.75 by 70%, (ii) the compression index C_c decreased from 2.85 to 0.95 by 70%, (iii) the swelling index C_s fell from 0.33 to 0.06 by 80%, (iv) the coefficient of volume compressibility m_v decreased from $4x10^{-4}$ (1/kPa) to $1x10^{-4}$ (1/kPa) by 75%, and (v) the secondary compression index C_a fell from 0.2 to 0.08 by 60%. The stiffness properties improved as well. The unload-reload stiffness E_{ur} increased from 3050 to 4650 kPa by 50%, and constrained modulus M increased from 825 to 1050 kPa by 50%.
- The Microstructure analysis revealed no further formation of calcite or vaterite crystals in the soil mass when the sand percentage was more than 10%. Therefore, any further improvement in the soil behaviour would solely be attributed to the sand percentage increase, not the precipitation of more calcite or vaterite crystals.
- When the cementation solution volume was more than the pores' volume, no further enhancement was noticed for the compression index C_c, but the swelling index C_s decreased. The constrained modulus M was noted to be fixed when injecting more than one pore's volume but exhibited less strain. The unload/reload stiffness increased to 5000 kPa when the injected volume was four times the pores' volume. The increase in the cohesion intercept reflected the improvement in the shear strength parameters, and no significant improvement was noticed in the angle of shearing resistance.

8.3 Significance of Contributions

One of the common engineering challenges for geotechnical engineers is the problems associated with construction on Muskeg soils. This research's primary contribution to the current knowledge is that it provided deep insights for characterizing the geotechnical properties and understanding the mechanical behaviour of Muskeg soils. Moreover, it investigated the applicability of the new environmentally-friendly stabilization technique, the Microbially Induced Calcite Precipitation MICP, in these soils.

This research started with developing new correlations to help geotechnical design engineers in the concept design phase of the project to estimate the index and compressibility properties of Muskeg soils. These correlations provided tentative values that would help draw the boundaries of the parameters under consideration. Although there were similar correlations in the literature, these correlations are the first to cover the entire organic content range.

This research investigated the applicability of the ball penetration test for the geotechnical characterization of Muskeg soils, despite it is known usage for offshore investigations. Moreover, this research proposed new correlations for estimating the undrained shear strength S_u and maximum shear modulus G_{max} , which would open the door for further research on the applicability and significance of the ball penetration test to characterize Muskeg soils' properties.

Soil stabilization using the MICP technique is evolving during the last two decades. The second part of this research comprised introducing this stabilization technique for Muskeg soils, which added a significant contribution to the body of knowledge of this new environmentally friendly technique and Muskeg soils' stabilization.

Furthermore, the stabilization of Muskeg soils using the MICP technique and sand from a local source was not investigated in the literature as per the author's knowledge. This research covered this knowledge gab by checking the Muskeg soils' behaviour before and after treatment with these two additives. The settlement and shear strength properties of Muskeg soils were improved when stabilized by the two additives compared to soils stabilized with the MICP individually.

8.4 Limitation of Study

Although this study contributes significantly to the body of knowledge of Muskeg soils characterization and stabilization, the following limitation out of this study should be considered:

- Although the provided correlations in the first phase of this study provide reasonable estimates for the parameters under considerations, these correlations are only applicable for the project's concept design phase.
- The MICP stabilization technique was introduced to Muskeg soils of 85% organic content.
 Therefore, the stabilization results of Muskeg soils of various organic contents should be compared to the results of this study.
- The studied behaviour of treated and untreated samples in this study was limited to consolidation and direct shear testing.

8.5 Recommendations for Future Studies

The following recommendations are proposed for future research:

- Conduct more field and laboratory testing for soils with different organic and water contents and compare the results with the correlations proposed in this study for the undrained shear strength parameter S_u and maximum shear modulus G_{max}.
- Conduct Constant Strain Rate CRS consolidation testing on treated samples and compare the results with the Incremental loading IL consolidation testing provided in this study. Similar results would indicate uniform and consistent compressibility behaviour of the treated samples.
- 3) Investigate the behaviour of treated Muskeg samples on the triaxial testing as the current study focused only on the direct shear test behaviour. Triaxial testing would provide more insights and increase the understanding of the soil behaviour at different loading conditions.

- 4) Carry out cyclic and dynamic shear testing for untreated and treated Muskeg soils as untreated Muskeg soils are known for their low shear stiffness. The treatment would increase the shear stiffness, and quantification of this enhancement would add an excellent contribution to the body of the knowledge.
- 5) Upscale the treatment process of Muskeg samples with bioslurry only and bioslurry and sand simultaneously and conduct compressibility and shear strength testing on the large-scale samples. This would allow to investigate the scale-effect of the treatment process.
- 6) The effect of changing the cementation solution concentration while fixing the bioslurry concentration should be investigated. Large cementation solution concentrations might lead to improved properties, but the improvement cost should be compared to the higher concentration cost.

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Appendices





Figure A.1. Soil Behaviour Type SBT classification interpreted from the SCPTu-18-01 results.



Figure A.2. Soil Behaviour Type SBT classification interpreted from the SCPTu-18-02 results.