Sampling disturbance in fine-grained soils

By

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BEng.(Hons)

This thesis is presented for the degree of
Doctor of Philosophy of
The University of Western Australia

Centre for Offshore Foundation Systems
School of Civil, Environmental and Mining Engineering

October 2018
Dedication

To

my family and Hoa Pham
I, Guan Tor Lim, hereby declare that:

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PUBLICATIONS ARISING FROM THIS THESIS

Conference paper


Journal papers


Lim, GT, Pineda, JA, Boukpeti, N & Carraro, JAH, Fourie, A 2018a, “Effects of sampling disturbance in geotechnical designs”. Canadian Geotechnical Journal, ISSN:0008-3674


Lim, GT, Pineda, JA, Boukpeti, N & Carraro, JAH, Fourie, A 2018c, “Mechanical behaviour of intact and reconstituted calcareous silts”. Submitted as a conference paper in 13th Australia New Zealand Conference on Geomechanics 2019

Lim, GT, Pineda, JA, Boukpeti, N & Carraro, JAH. Fourie, A 2018d, “Physical modelling of tube sampling in calcareous silty soils”. Prepared to be submitted
AUTHORSHIP

The thesis is presented as a series of papers, according to the Graduate Research School regulations.

The thesis contains work that has been published or prepared to be submitted for publication, some of which has been co-authored.

The following statements illustrate the contribution of the candidate to each of the co-authored work.

Details of the work:
Location in the thesis: Chapter 2
Candidate contribution to work:
The candidate conducted the experimental work, analysed the data and had the main role in drafting the manuscript.

Details of the work:
Lim, GT, Pineda, JA, Boukpeti, N & Carraro, JAH, Fourie, A 2018a, “Effects of sampling disturbance in geotechnical designs”, Canadian Geotechnical Journal, ISSN:0008-3674
Location in the thesis: Chapter 3
Candidate contribution to work:
The candidate carried out the site study, experimental work, analysed the data and had the main role in drafting the manuscript.
**Details of the work:**

*Location in the thesis:* Chapter 4

*Candidate contribution to work:*  
The candidate carried out the parametric study, analysed the data and had the main role in drafting the paper.

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**Details of the work:**
Lim, GT, Pineda, JA, Boukpeti, N & Carraro, JAH, Fourie, A 2018c, “Mechanical behaviour of intact and reconstituted calcareous silts”. *Submitted as a conference paper in 13th Australia New Zealand Conference on Geomechanics 2019*

*Location in the thesis:* Chapter 5

*Candidate contribution to work:*  
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**Details of the work:**

*Location in the thesis:* Chapter 6

*Candidate contribution to work:*  
The candidate designed and built the physical model, carried out experimental work, analysed the data and had the main role in drafting the paper.

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**Details of the work:**
Lim, GT, Pineda, JA, Boukpeti, N & Carraro, JAH. Fourie, A 2018d, “Physical modelling of tube sampling in calcareous silty soils”. *In preparation to be submitted*

*Location in the thesis:* Chapter 8

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ABSTRACT

Sampling disturbance is an important consideration when interpreting geotechnical parameters from laboratory data to ensure reliable and economical design of geotechnical structures. In this study, sampling disturbance in fine-grained soils such as soft clay and silty soils are investigated. The investigation is divided into two approaches addressing two different soils; a soft estuarine clay, hereafter referred to as Ballina clay and offshore calcareous silts retrieved at various water depths from the North West Shelf of Western Australia. The study includes extensive experimental tests such as one-dimensional consolidation tests, triaxial tests, simple shear tests with cell pressure confinement and small strain stiffness measurement using bender elements and a resonant column apparatus. Additionally, two sets of physical models at different scales were conducted; one small-scale to study soil deformations in the sampling tube using digital image correlation techniques and another, large-scale physical model to simulate tube sampling in a well-controlled environment to provide new insights of tube sampling.

An experimental study was carried out to evaluate the quality of tube specimens from an estuarine soft clay deposit containing natural heterogeneities such as shell fragments. The effects of volumetric shell fractions on compressibility parameters were investigated, with sampling disturbance taken into consideration. Additionally, different sampling methods such as open push tube sampler (Shelby tube), fixed and free piston sampler and block sampler (Sherbrooke sampler) were used, to retrieve soil samples to investigate the effects of sampling methods in geotechnical designs. The consequences of sampling disturbance were illustrated using the design of an embankment and shallow foundation and how different outcomes were obtained using measured data from different sampling methods.

Experimental studies were also performed to characterise the offshore fine-grained calcareous silts to be used in physical modelling. Element tests such as one-dimensional consolidation tests and triaxial tests were performed to study the mechanical properties of these soils while a scanning electron microscope was used to evaluate the resulting microstructure and fabrics. The effects of tube penetration rates and corresponding drainage conditions were study using two different physical models, a small-scale physical model and a large-scale physical model. The small-scale physical model studied the effects of penetration rates on shear strains in the sampled soil using only a digital image correlation technique. Conversely, the large-scale model allowed the measurement of excess pore pressure development, retrieval of tube and block specimens to be tested.
in the laboratory as well as shear strain measurement during tube sampling. The additional information provided in the model allowed new understandings of tube sampling in the soil.

Different aspects of sampling disturbance on fine-grained soils have been studied and new insights of tube sampling are presented. Some recommendations are made to improve the sampling process to retrieve undisturbed specimens for element tests.
ACKNOWLEDGEMENTS

First and foremost, I would like to express my heartfelt appreciation to my supervisors, Professor Andy Fourie, Dr. Nathalie Boukpeti, Dr Jubert Pineda and Assoc. Prof Antonio Carraro for their consistent guidance, support and encouragement during my PhD candidature. Andy has been supportive and helpful in guiding and coordinating my PhD journey during the last final year. Not only is he always reachable despite his busy schedule, he is always ready to provide constructive feedback. My initial milestones will not be reached if not because of Nathalie and Jubert who continue to inspire and be supportive in developing my research skills. They have shaped me personally as well as academically as a researcher. Without Antonio, I will not fall in love with soil testings and have high standards in doing experiments in the soil laboratory. His attention to details and depth in knowledge had guided myself to be proficient in soil laboratory testings.

My sincere thanks to all technicians in the soil labs, centrifuge labs, electrical and electronic workshop and mechanical workshops for providing technical assistance in setting up experimental works, physical models development and physical modelling. They include Yaurel Guadalupe Torres, Behnaz Abdollahzadeh, Usha Mani and Satoko Ishigami who helped in soil experimental testing, Manuel Palacios, Adam Stubbs and Kelvin Leong for physical modelling, John Breen, Guido Wager and Khin Seint for my electrical, electronic and software development for the physical model and Matt Arpin and Frank Tan for guiding and building the physical model for testings. I am deeply grateful for all of their help, time and friendship during these years.

I am deeply indebted for genuine friendships from my fellow colleagues throughout these few years. They have created a wonderful, pleasant and lively atmosphere and their presence will continue to be missed. They are Tianyuan, Dengfeng, Minh Tri, Rafael, Dunja, Manuel, Guy, Pauline, Jit Kheng, Stefanus, Mark, Adriano, Yining, Wensong, Fuming, JoonMo, Dmitra, Nicole and Fillippo.

Throughout my candidature, I was financially supported by Australian Government Research Training Program (RTP) Scholarship and Adhoc-Scholarship from COFS, which are gratefully acknowledged. A huge gratitude to all the administration staffs,
Monica, Lisa, Rochelle, Dana and Kirstin for their consistent help, guidance and continuous support on administration work.

My sincere appreciations to my family for their love and care during stressful times especially my parents and my siblings Jackson, Regine, Cherlyn and Vincent who continue to provide comfort with multiple calls and video calls even when we are far away from each other.

To my partner, Hoa Pham whom this thesis is dedicated to, I cannot thank you enough for your unconditional love, persistence support, continuous encouragement and understanding for my busy times. Your bubbly character and positive energy fuelled my life with motivation and inspiration to finish this thesis on timely manner.
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NOTATION

Roman

AR .......... area ratio
B .......... width of the strip footing
BE .......... bender element
BH .......... Barron and Hansbro's method
CaCO3 ................................ calcium carbonate
CAU ...... anisotropically-consolidated undrained triaxial tests
Cc ........... compression index
CRS ...... one dimensional constant rate of strain
Cs ........... swelling index
cv .......... coefficient of consolidation
cv .......... coefficient of consolidation
cv,1 ........... coefficient of consolidation of top layer at the interface between two layers
D .......... damping ratio
dc .......... depth correction factor
De .......... outer diameter
de .......... diameter of influence with square pattern
dw .......... equivalent diameter of the PVD
DIC ........ digital image correlation
dm .......... equivalent diameter of mandrel
ds .......... diameter of smear zone
e0 .......... initial void ratio
E0 .......... elastic soil modulus
f .......... torsional resonant frequency
fapp ........ apparent frequency
FDM .......... finite difference method
G(\nu) .......... small-strain shear modulus
G\nu·sec .......... secant shear modulus
HPG .......... hydraulic profile gauges
i_c .......... simplified Meyerhof inclination factor
IL ............. incremental loading specimen

Ip ............. plasticity index

SG ............. specific gravity

ISA ............. ideal sampling approach

K ............. cone factor that depends on the conical angle

k1 ............. hydraulic conductivity of top layer at the interface between two layers

k2 ............. hydraulic conductivity of bottom layer at interface between two layers

kh ............. hydraulic conductivity in horizontal direction

kv ............. hydraulic conductivity in vertical direction

L ............. length

leff ............. effective length

lm ............. length of mandrel

LVDT ......... linear variable displacement transducer

M ............. oedometric modulus

MEX ......... magnetic extensometers

NC ............. normal consolidation state

OC ............. over consolidation state

OCA ......... outer cutting angle

OCR ......... over consolidation ratio

PIV ......... particle image velocimetry

PSA ......... perfect sampling approach

PSD ......... particle size distribution

PVD ......... prefabricated vertical drains

p_{in situ} ......... in situ mean effective stress

q ............. deviator stress

q0 ............. net applied pressure on the foundation

qf ............. undrained bearing capacity

RC/RCA ......... resonant column apparatus

RH ......... Relative humidity

sc ............. shape correction factor
SEM...... ..... scanning electron microscope
SP ............. ..... settlement plates
SPM...... ..... strain path method
SS............. ..... simple shear test
s_r .......... ..... settlement at any time, t
s_u .......... ..... undrained shear strength
t .......... ..... thickness
t_p .......... ..... time at which yielding occurs
ts .......... ..... time interval between the first peak of the transmitted and received signal
U .......... ..... average degree of consolidation in the particular sublayer
u(i,j).......... ..... excess pore pressure at node i at time j
u_0 .......... ..... initial excess pore pressure
USCS .... ..... Unified Soil Classification System
V_pp ............. ..... peak to peak voltage
V_SF ............. ..... volumetric shell fraction
V_s(hh) ...... ..... shear wave velocity with horizontal polarization measured in horizontal direction
V_s(ij) ...... ..... shear wave velocity where subscripts i and j refer to the wave propagation and polarization directions
V_s(vh) ...... ..... shear wave velocity with horizontal polarization measured in vertical direction
VWP ...... ..... vibrating wire piezometers
w_L .......... ..... liquid limit
y_m .......... ..... maximum lateral displacement
YSR ...... ..... yield stress ratio

Symbol
σ_v-in situ .. ..... vertical in situ effective stresses
σ_h-in situ .. ..... horizontal in situ effective stresses
β .......... ..... device’s calibration factor
δ .......... ..... logarithmic decrement of free vibration decay curve
σ’ \text{axial} \ldots \text{effective axial stress}

σ’ \text{radial} \ldots \text{effective radial stress}

σ’ \text{v} \ldots \text{effective vertical stress}

γ_{\text{ref}} \ldots \text{reference shear strain}

σ’ \text{yield} \ldots \text{yield stress}

φ’ \text{sec} \ldots \text{secant friction angle}

τ_{\text{xy}} \ldots \text{undrained shear strength from simple shear test}

γ \ldots \text{shear strain}

μ_1 \ldots \text{correction factor for finite thickness of an elastic soil layer}

μ_2 \ldots \text{correction factor for depth of embedment of foundation}

α \ldots \text{ratio of maximum lateral displacement to vertical displacement}
CHAPTER 1. INTRODUCTION

1.1 BACKGROUND

Soil parameters used in geotechnical design and analysis are obtained through laboratory and in situ tests. In situ tests can provide information on the variation of soil properties with minimal disturbance but are plagued by interpretation issues. Despite this, laboratory tests are still preferred to in situ tests, because the boundary and drainage conditions of the tests can be well controlled, as well as the soil stress path. In order for laboratory tests to be carried out, soil samples need to be retrieved from the ground and transported to the laboratory. Among the available sampling techniques, tube sampling is still the most common choice because of its cost and time effectiveness. Additionally, it is only feasible to obtain soil samples from tube sampling in offshore environments instead of onshore block sampling techniques such as the Sherbrooke sampler. Good quality specimens can be obtained in clayey soils due to their low hydraulic conductivity and corresponding pore pressure changes imparted by sample unloading, in combination with the improvements of the sampling techniques (Ladd & DeGroot 2003). These improvements are based on the vast amount of research on sampling disturbance effects in clays carried out during the last few decades. In contrast to this, in situ sandy materials cannot be sampled cost-effectively without causing major disturbance to the soil. Hence, reconstituted specimens of sands are usually used for laboratory testing on those soils. To sample silty materials, such as those found in some parts of the North-West Shelf of Western Australia, sampling techniques similar to those used for clayey soils are commonly adopted. However, there is no clear understanding of the actual soil changes in state during sampling and their effect on the determination of geotechnical parameters for design of offshore structures resting on such deposits.

Soil disturbance caused by sampling is unavoidable, even when the best recommended procedure such as using block sampling technique instead of tube sampling is practiced. From the process of drilling into the ground to specimen preparation in the laboratory, soil is subjected to different disturbances. The displacement of the soil mass during tube penetration, the reduction of shear stresses due to the soil removal from the ground, as well as vibrations induced during transportation are unavoidable sources of soil
disturbance. During the last six decades, substantial research has been carried out on the assessment of sampling disturbance in clayey materials (Yang et al. 2018; Pineda, Liu & Sloan 2016; Lim et al. 2015; Pineda, McConnell & Kelly 2014; Berre 2014; Horng, Tanaka & Obara 2010; Prasad et al. 2007; DeGroot, Poirier & Landon 2005; Nagaraj et al. 2003; Santagata & Germaine 2002; Clayton, Siddique & Hopper 1998; Baligh, Azzouz & Chin 1987; Lacasse, Berre & Lefebvre 1986; La Rochelle & Lefebvre 1971; Hvorslev 1949). While most studies have focussed on clayey materials, for which undrained behaviour can be assumed, not much work has been conducted on intermediate soils, in particular offshore calcareous silts, which may respond in a partially drained manner during sampling. Silty soils are located between clean sands and clays and their mechanical behaviour is still not fully understood from a fundamental standpoint.

Major studies carried out on sampling disturbance were focussed on clayey materials assuming an undrained behaviour and not much work has been conducted in silty soils which can respond in a partially drained manner. The effect of sampling disturbance on silty soils is an understudied topic compared to clayey materials. At present, it is not known whether sampling disturbance on these materials reduces the measured shear strength by destroying the soil structure or, conversely, whether their measured strength is increased by densifying the soil. Hight and Leroueil (2003) showed that sampling disturbance due to piston tube sampling densified loose silts and sand, which led to increase in stiffness and strength. Additionally, initial work by Long (2006) has provided some evidence on the densification of Athlone laminated clay/silt due to tube sampling disturbance that increase the measured stiffness and undrained shear strength of silty material. The recent work by Carroll and Long (2017) on silty materials obtained using piston tube samplers and block samplers from three different sites in Ireland and Norway has shown contrasting responses between samples from each site. Two soil samples (Letterkenny in Ireland and Refnevien in Norway) show similar quality between specimens obtained using piston tube samplers and block samplers whereas another soil sample (Skibbereen in Ireland) shows densification in piston sampling compared to block sampling.
1.2 SOURCES OF SAMPLE DISTURBANCE AND ITS EFFECTS ON MEASURED PROPERTIES

Soil is subjected to various disturbances from drilling to transportation as well as due to specimen set-up. In Figure 1-1, Ladd & DeGroot (2003) show the effective stress path followed by a center-line element in a clay sample during a tube sampling campaign.

![Figure 1-1: Hypothetical stress path during tube sampling and specimen preparation of center-line element of low OCR clay (modified after Ladd & DeGroot 2003)](image)

<table>
<thead>
<tr>
<th>Path</th>
<th>Event</th>
</tr>
</thead>
<tbody>
<tr>
<td>1-2</td>
<td>Drilling</td>
</tr>
<tr>
<td>2-3-4-5</td>
<td>Tube Sampling</td>
</tr>
<tr>
<td>5-6</td>
<td>Tube Extraction</td>
</tr>
<tr>
<td>6-7</td>
<td>Transportation &amp; Storage</td>
</tr>
<tr>
<td>7-8</td>
<td>Sample Extrusion</td>
</tr>
<tr>
<td>8-8</td>
<td>Specimen Set-Up</td>
</tr>
</tbody>
</table>

Point 1 represents the initial effective stresses of the soil sample in the ground, while Point 9 represents the final effective stresses of the soil sample before being tested in the laboratory. From Point 1 to Point 9, the effective stresses fluctuate according to the various events of the sampling process. Effective stress path 2-3-4-5 represents the changes of stresses due to tube sampling, which depends on tube sampler geometry of the tube sampler. Paths 1-2, 5-6 and 7-8 represent the effective stress paths of the sample when subjected to stress relief disturbance (i.e. removal of the in situ stresses acting on the sample). These unavoidable disturbances affect the state and structure of the soil, which results in changes of soil’s stress-strain response and mechanical properties such as shear strength, volume compressibility and shear stiffness (Ladd & DeGroot 2003). Effects of sampling disturbance on clay properties have been studied extensively since the 1950s. Generally, the studies are based on two main events: (1) the tube penetration process using different types of sampler with different geometries and (2) effects of shear stress relief disturbance. Past research on clays (Baligh et al. 1987; Atkinson et al. 1992; Hight et al. 1992; Santagata 1994; Bashar et al. 1997; DeGroot et al. 2005; Pineda et al.,
2016) has shown the following effects with increasing degree of disturbance: reduction of effective stress, undrained shear strength and soil stiffness; increase of shearing strain at peak, loss of the strain softening (increase of ductility) as well as irreversible modifications in soil fabric and structure. However, no clear conclusion can be drawn on the effects of sampling disturbance in intermediate soils because far too little attention has been paid to the behavior of these soils during sampling.

1.3 LABORATORY SIMULATION OF FIELD SAMPLING AND PHYSICAL MODELS

Laboratory testing using physical modelling is a useful method to study sampling disturbance because simulation and testing can be done in a well-controlled environment, with specified boundary and drainage conditions and uniform stresses within soil specimens (Baligh et al. 1987). Several studies have been carried out in the past on different components of the clay sampling process: simulation of stress relief when the confined sample is extruded from the tube to an isotropic stress environment (Kirkpatrick & Khan 1984; Baligh et al. 1987; Bashar et al. 1997; Siddique et al. 2000a; Siddique et al. 2000b; Santagata & Germaine 2002); simulation of disturbance due to a tube penetration event (Baligh et al. 1987; Clayton et al. 1998a; Tanaka & Tanaka 1999) and other general events such as transportation and preparation of a soil specimen (Siddique 1990; Hight et al. 1992). These simulations were carried out by applying the corresponding stress paths on the specimen before tested in the triaxial machine.

A specifically designed model tube sampling apparatus was developed by Santagata et al. (2006) to study the effects on selected engineering properties of tube sampling with different tip geometries. The whole model allows tube samplers to penetrate into the soil, while the perimeter of the soil remains under consolidation stresses. The retrieved samples were subjected to unconsolidated undrained triaxial tests and constant rate of strain oedometer testing. The study concluded that samples retrieved using the physical model were subjected to far greater disturbance compared with the simple simulation (application of corresponding stress paths) carried out in the triaxial machine. In addition, it was found that tip geometry plays an important role on the magnitude of disturbance.

On the other hand, simulation of the sampling process to identify soil disturbance has been carried out using advanced imaging technology. Yan et al. (2010) used particle
image velocimetry and close-range photogrammetry to study the displacement fields around the sampler during the advancement of a tube. A sampler was pushed into lightly overconsolidated kaolin clay to identify the influence zone during the sampling process. A similar approach was followed by Hover et al. (2013) using particle image velocimetry to obtain images during tube penetration on transparent soil. Hover et al. (2013) showed that the center-line strain path is not symmetrical, with different peak strains in compression and extension. Both studies deduced that the soil flow pattern consists of compression/extension stages and asymmetrical strain paths, which qualitatively agree with the prediction from the strain path method proposed by Baligh et al. (1987).

1.4 AIMS

Sampling disturbance affects the mechanical properties of the soil sampled. Results obtained from subsequent laboratory tests carried out on sampled soil may provide altered soil parameters for use in engineering design. This may lead to erroneous design causing catastrophic failure or excessive costs (Lunne et al. 2006). By knowing the effects of sampling disturbance on fine-grained soil such as soft clay and silty soils, this project will provide ways to mitigate and minimise consequences on estimation of soil parameters used in geotechnical design. Interpretation of laboratory results for engineering design will be done more accurately, allowing some reduction of safety factor. Not only the costs for construction projects can be reduced but also a safer infrastructure can be constructed. In addition, this project will also contribute to the development of refined sampling techniques for offshore calcareous silts, which are frequently encountered in the North-West Shelf, Western Australia.

Two approaches were considered: (1) determination of soil mechanical properties from laboratory tests on specimens obtained with different samplers, and (2) physical modelling of the sampling process which uses the digital image correlation technique to determine displacements and strains in the soil. Approach (1) was applied to Ballina clay due to the access to testing site and sampling campaign including Sherbrooke sampler (block sampler). This type of sampling campaign cannot be performed in offshore soils as it is either too costly or impossible to retrieve block samples in offshore environment. Approach (2) was applied to offshore silty calcareous soils due to the relevance for the design of offshore Australian infrastructure.
Therefore, the main aims of this research are:

1. To identify and quantify the effects of disturbance on the mechanical properties of fine-grained soils such as soft clay and silty soils (e.g. shear strength, stiffness, pre-consolidation stress and compressibility parameters) and on the soil structure (fabric and bonding) and

2. To propose some recommendations for characterisation of fine-grained soils such as soft clay and silty soils related to sampling method, laboratory procedure and determination of engineering properties.

### 1.5 RESEARCH PLAN

Following The University of Western Australia’s regulations regarding Research Higher Degrees, this thesis is presented as a series of papers that has been published (Chapter 2 and Chapter 4), submitted (Chapter 3 and Chapter 6) and in preparation to be submitted (Chapter 5 and 7). The thesis comprises of eight chapters. The flowchart of the thesis structure is illustrated in Figure 1-2. Chapter 1 provides a brief introduction of the research including a literature review on soil sampling disturbance and physical models to evaluate the effects of sampling disturbance.

![Flowchart of the thesis structure](image-url)

**Figure 1-2: Flowchart of the thesis structure**
Chapter 2 presents the results of an experimental study addressing the effects of tube sampling in an estuarine soft clay deposit – Ballina clay –, which contains variable amount of shells. The study was based on shear wave velocity measurement and determination of compressibility parameters from one-dimensional consolidation tests. The effects of volumetric shell fractions (percentage of shell fraction in soil specimen) on compressibility parameters was investigated, with sampling disturbance taken into consideration.

Chapter 3 describes an extensive experimental study of the effects of sampling disturbance in Ballina clay and the consequences of different sample quality (resulting from using different sampler types) on the representativeness of soil parameters used in geotechnical design. The experimental study includes common laboratory tests (such as one-dimensional compression tests and simple shear tests) as well as shear wave velocity measurement and resonant column tests to measure small strain stiffness of soil specimens. Mechanical soil properties derived from specimens retrieved using the different samplers are used in the prediction of two classical problems in soil mechanics: the surface settlement and excess pore pressure response underneath an embankment, as well as the settlement and bearing capacity of a shallow footing.

Chapter 4 presents a systematic parametric study on the interpretation of geotechnical parameters from laboratory results to predict the behaviour of an embankment built on soft Ballina clay improved with prefabricated vertical drains. The characteristics of the embankment are very similar to the embankment analysed in Chapter 3. By predicting and back analysing the behaviour of a real embankment using hand-calculation and finite difference methods, the methodology used for geotechnical design of the embankment in Chapter 3 can be assured to be rigorous.

Chapter 5 presents an experimental study to characterise the offshore calcareous soils to be used in physical modelling, with the aim of identifying the effects of soil reconstitution on soil behaviour. Various element tests such as one-dimensional compression tests and anisotropic consolidation triaxial tests were carried out to study soils from different water depths, in “intact” and reconstituted states. Furthermore, scanning electron microscope was used to observe the microstructure of intact and reconstituted specimens to gain some insights on the mechanical behaviour of these soils.
Chapter 6 presents a small scale physical model developed to determine the displacement and strain fields around tube samplers penetrating in calcareous soils, based on the digital image correlation technique. The movements of soil particles were captured using a digital camera during penetration of a half-tube sampler against a Perspex window. The resulting images were analysed using the program GeoPIV_RG. The effects of different sampler wall thicknesses and sampling penetration rates on soil disturbance were investigated.

Chapter 7 presents a large-scale physical model developed to simulate tube sampling in a controlled environment. It includes the design of the physical model and control system, experimental procedure and the outcomes of the physical modelling. This physical model allowed three different viewpoints to provide new insights on sampling disturbance during tube sampling: (i) strain and displacement along the tube samples identified using particle image velocimetry and digital image correlation, (ii) effective stress during tube sampling determined using measurement of excess pore pressure developed along the tube centreline and (iii) compressibility and undrained shear strength measurement from retrieved tube specimens.

Finally, Chapter 8 discusses the outcomes of the research carried out and presents some recommendations for future work.

1.6 REFERENCES


Hvorslev, MJ 1949, 'Subsurface exploration and sampling of soils for civil engineering purposes'.

La Rochelle, P & Lefebvre, G 1971, 'Sampling disturbance in Champlain clays', in *Sampling of soil and rock*, ASTM International.


Yan, W, Ng, I & Cheuk, C 2010, 'Displacement field around an open-tube sampling', in Proc. 7th Int. Conf. Physical Modelling in Geotechnics, Zurich, Switzerland, pp. 411-416.

CHAPTER 2. TESTING TUBE SPECIMENS FROM SOFT CLAY DEPOSITS CONTAINING VARIABLE AMOUNTS OF SHELLS

Abstract: This chapter presents the results of an experimental program aimed at characterizing tube specimens from an estuarine soft clay deposit, which contains variable amounts of shells. Investigations on the influence of shell fragments on the compressibility parameters of natural soft clay and on sample quality were carried out. The investigation involved shear wave velocity measurements using bender elements (which can assist in assessing sample quality), as well as one-dimensional consolidation tests for evaluating soil compressibility. The experimental results show that the volumetric shell fraction influences compressibility of the natural soft clay and sample quality.
2.1 INTRODUCTION

In order to obtain reliable results from laboratory tests, an important aspect of experimental studies is to test good quality specimens that represent the soil’s natural state. Natural soil variability along the tube, e.g. presence of shells or inclusions as well as soil disturbance caused by the sampling process may influence the laboratory test results. Inconsistencies of test results may lead to failure of infrastructure and excessive mitigation costs in projects. Furthermore, the presence of natural heterogeneities, such as inclusions of larger particles, results in additional experimental challenges, particularly in regard to soil extrusion from sampling tubes and specimen trimming, which may lead to additional soil disturbance.

The behavior of clays mixed with granular materials has been studied by a number of researchers (Lupini et al. 1981; Graham et al. 1989; Kumar & Wood 1999; Yin 1999). One-dimensional compression tests on clay-sand or clay-gravel mixtures prepared in the laboratory have shown that the compressibility of the mixture decreases with an increase in granular volume fraction i.e. percentage of granular materials (Kumar & Wood 1999; Yin 1999). In addition, experiments have demonstrated that for a granular volume fraction below a critical value (in the range 0.3 to 0.45) the behaviour is determined by the clay matrix (Lupini et al. 1981; Kumar & Wood 1999). While substantial amount of research has been carried out on the effects of sampling disturbance on the behavior of clay materials (Clayton & Siddique 2001; DeGroot et al. 2005; Berre et al. 2007; Horng et al. 2010), little has been reported on the disturbance caused by the sampling process in mixtures of clay and granular materials. The aims of this study are twofold: (i) investigate the influence of shell fragments on the compressibility parameters of natural soft clay, (ii) assess the quality of samples of this natural clay obtained with sampling tubes having different outer cutting toe angles (OCA) and piston samplers and how it relates to the presence of natural heterogeneities. The paper presents and discusses the results of a series of laboratory tests, namely bender elements tests (to measure shear wave velocity) and incremental loading one-dimensional compression tests (IL) carried out on tube specimens of natural soft clay containing variable amount of shells.
Chapter 2: Testing tube specimens from soft clay deposits containing variable amounts of shells

2.2 MATERIALS TESTED

Tube specimens of soft estuarine Ballina clay were obtained from Australia’s first national facility for soft soil testing located near Ballina, New South Wales. The soil profile comprises an upper alluvium layer (∼1 m thick) overlying a 10 m thick soft clay layer (126% <w< 68%; organic content ≈ 5%) whose index properties (Atterberg limits) are strongly sensitive to previous drying before testing (Bishop 2009). The clayey fraction is mainly composed of smectite, kaolinite and mica/illite with variable percentages (Pineda et al. 2013). The Shelby tube sampler and Osterberg piston sampler were used to retrieve undisturbed tube specimens at a depth between 3.5 and 4.1 m. Both were thin-walled samplers while the Osterberg piston sampler had a piston to assist extraction of the soil. The Shelby tubes (U75) had 75 mm internal diameter with three different OCA (5°, 15° and 90°) while Osterberg (O90) had 85 mm internal diameter with OCA of 5°. The tubes are labelled U75-5, U75-15, U75-90 and O90, respectively.

Characterization tests were carried out to determine the basic properties of the soil. These tests included electrical conductivity (EC), Atterberg limits, specific gravity, and particle size distribution tests. The top and bottom ends of each tube were used for this purpose. To determine the EC of the pore fluid, the squeezing technique by Pineda et al. (2013) was used to squeeze out the pore fluid from the soil. Due to the strong influence of pore fluid salinity on the mechanical behaviour of Ballina clay (Pineda et al. 2013), a synthetic solution prepared at the same EC as that of the natural pore fluid was used to perform the consistency limit tests as well as the IL tests. Atterberg limits were performed using soil that was gently pushed through the 425 µm sieve to separate the fine fraction. Liquid limit was determined using fall cone tests. The first point was taken at natural water content. For the two subsequent points, the synthetic solution described above was added to increase the water content of the paste. The soil was mixed well and left to cure for 24 hours before performing the next point. Plastic limit was determined using the thread rolling method according to ASTM Standard D4318 (2010), while specific gravity of the soil was determined using water pycnometer method according to ASTM Standard D854 (2006). Particle size distribution was determined using Micromeritics Particle Size Analyzer. Table 2-1 summarizes the main characteristics of the tube specimens tested, whereas Figure 2-1 shows the particle size distributions for each tube specimen.
Table 2-1: Basic soil properties of Ballina clay in each tube

<table>
<thead>
<tr>
<th>Soil Properties</th>
<th>U75 -5 °</th>
<th>U75 - 15 °</th>
<th>U75 - 90 °</th>
<th>O90</th>
</tr>
</thead>
<tbody>
<tr>
<td>Electrical conductivity, $E$ (mS/cm)</td>
<td>22.54</td>
<td>24.54</td>
<td>19.28</td>
<td>21.20</td>
</tr>
<tr>
<td>Initial water content, $w_o$ (%)</td>
<td>85-87</td>
<td>77-90</td>
<td>90-93</td>
<td>78-89</td>
</tr>
<tr>
<td>Plastic limit, $w_p$ (%)</td>
<td>35.8</td>
<td>36.5</td>
<td>31.3</td>
<td>37</td>
</tr>
<tr>
<td>Liquid limit, $w_l$ (%)</td>
<td>74.9</td>
<td>95.7</td>
<td>94.3</td>
<td>108.5</td>
</tr>
<tr>
<td>Plasticity index, PI (%)</td>
<td>39.1</td>
<td>59.3</td>
<td>63</td>
<td>71.5</td>
</tr>
<tr>
<td>Specific gravity, $G_s$</td>
<td>2.64</td>
<td>2.52</td>
<td>2.71</td>
<td>2.61</td>
</tr>
<tr>
<td>Initial void ratio, $e_o$</td>
<td>2.27-2.36</td>
<td>2.21-2.60</td>
<td>2.40-2.52</td>
<td>2.10-2.41</td>
</tr>
<tr>
<td>Area ratio, AR (%)</td>
<td>8.4</td>
<td>8.4</td>
<td>8.4</td>
<td>9.6</td>
</tr>
</tbody>
</table>

Figure 2-1: Particle size distribution of Ballina clay in each tube

2.3 EXPERIMENTAL PROCEDURE

Each tube sample was scanned using an X-ray machine to determine the distribution of shell fragments as shown in Figure 2-2. Two slice sections for IL testing were identified in each tube: one with a relatively low volumetric shell fraction, labelled as OD1, and one with a relatively large volumetric shell fraction, labelled as OD2. The specimens are named according to the tube they were obtained from, their OCA and volumetric shell
fraction. For instance, U75-5 OD1 represents a specimen from U75 tube with 5 degrees OCA and relatively low volumetric shell fraction.

![Image of X-rays scan of tube specimens and the nominated location of OD1 and OD2.]

**Figure 2-2: X-rays scan of tube specimens and the nominated location of OD1 and OD2.**

2.3.1 **Determination of the volumetric shell fraction**

To determine the volume of shell fragments, the wet sieving method was used. Before the wet sieving procedure, the dry specimens from the final water content test after IL testing were soaked overnight in de-ionised water to soften the dried clay. The soaked specimens were then placed on a 425 µm sieve and washed with running water. The shell fragments retained on the sieve were then dried in the oven to determine the dry mass. By using the density of shells, determined as \( G_{v,\text{shell}} = 2.89 \text{ g/cm}^3 \), the volume occupied by the shell fragments was determined.

2.3.2 **Bender elements and one-dimensional compression tests**

Bender elements (BE) tests were performed on tube specimens 55 mm in height, prior to the IL tests. A special frame was manufactured to hold the bender elements, which were inserted at the top and base of the specimen. The frame ensured accurate alignment
of the BE transducers, as well as correct determination of the travel length. BE measurements were taken under two configurations: (i) before specimen extrusion (specimen in the tube), and (ii) after specimen extrusion. A sine pulse excitation (20 V peak to peak amplitude) was used with frequencies of 1, 3 and 5 kHz. Shear wave velocity was determined as \( V_s = \frac{l_{\text{eff}}}{t_s} \), where \( l_{\text{eff}} \) is the effective length (measured between the tips of the piezoceramic elements) and \( t_s \) is the travel time, determined from the distance between the first peaks in the transmitted and received signals (Viggiani & Atkinson 1995).

Eight IL tests were carried out using fixed ring consolidometer cells with a diameter of 71 mm and height of 25 mm. The ring was pushed directly into the intact specimen after extrusion from the tube. The synthetic pore fluid discussed previously was used to inundate the consolidometer cells. Each specimen was loaded up to a maximum vertical stress of 500 kPa, including an unloading-reloading cycle at 250 kPa. A minimum consolidation time of 24 hours was allowed between each load increment before the next increment.

### 2.4 RESULTS AND DISCUSSION

#### 2.4.1 Volumetric shell fraction

The volumetric shell fraction \( (V_{sf}) \) of the compression specimens (71 mm diameter) was determined according to the following equation

\[
V_{sf} = \frac{\text{Volume of shell, } V_{\text{shell}}}{\text{Volume of IL specimen, } V_{\text{IL Specimen}}} \times 100 \%
\]  

(2-1)

where \( V_{\text{shell}} \) is the volume of shell in the IL specimen determined by sieving and \( V_{\text{IL}} \) is the initial volume of the IL specimen. The values reported in Table 2-2 indicate that \( V_{sf} \) was small for all the specimens tested and varied in the range 0.21 to 2.03 %. U75-5 OD1 had the least amount of shells while O90 OD2 had the most amounts of shells, approximately 10 times the amount of U75-5 OD1. The size of the shell fragments relative to a two Australian dollars coin with a diameter of 20.5 mm is compared in Figure 2-3. Most of the shell fragments were broken pieces and were relatively small compared to the size of the coin, except for U75-90 OD2, which contained a few full-size shells with size ranging between 14 mm to 21 mm in length.
Chapter 2: Testing tube specimens from soft clay deposits containing variable amounts of shells

Table 2-2: Volumetric shell fraction for each tested specimen

<table>
<thead>
<tr>
<th>Specimens</th>
<th>U75-5 OD1</th>
<th>U75-5 OD2</th>
<th>U75-15 OD1</th>
<th>U75-15 OD2</th>
<th>U75-90 OD1</th>
<th>U75-90 OD2</th>
<th>O90 OD1</th>
<th>O90 OD2</th>
</tr>
</thead>
<tbody>
<tr>
<td>Volumetric shell fraction, Vsf (%)</td>
<td>0.24</td>
<td>0.91</td>
<td>0.76</td>
<td>0.98</td>
<td>0.93</td>
<td>1.58</td>
<td>0.35</td>
<td>2.03</td>
</tr>
</tbody>
</table>

a. U75-5 OD1  
b. U75-15 OD1  
c. U75-90 OD1
d. U75-5 OD2  
e. U75-15 OD2  
f. U75-90 OD2
g. O90 OD1  
h. O90 OD2

Figure 2-3: Shells retained for each specimen
2.4.2 Shear wave velocity

The shear wave velocity results \( (V_s) \) are reported in Table 2-3, except for U75-90 OD1 for which no signal was detected by the receiver. The values obtained for the clay specimens before extrusion and after extrusion are very similar and are close to the \( V_s \) measured \textit{in-situ} using seismic dilatometer (average value of 53 m/s).

The relationship between \( V_s \) and \( V_{sf} \) is examined in Figure 2-4. No obvious correlation can be proposed on the relationship between \( V_s \) and \( V_{sf} \). Figure 2-4 also seems to indicate that \( V_s \) increases with OCA (by comparing U75-5 OD2 and U75-15 OD2 with almost similar \( V_{sf} \)). Between OD1 and OD2 for each tube sampler, it appears that \( V_s \) was lower with a higher \( V_{sf} \). There may be additional factors that are not well addressed to confirm the exact relationship between \( V_s \) and \( V_{sf} \) such as soil homogeneity and distribution of shell fragments including their orientation in the specimen. \( V_s \) measurement has been proposed as a non-destructive method for evaluation of sampling disturbance in homogeneous clay (Donohue & Long 2010). These results indicate that this method may not be suitable for clay containing granular inclusions, as \( V_s \) is influenced by the variations in granular volume fraction.

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Estimated Shear Wave Velocities, m/s</th>
<th>Before extrusion</th>
<th>After extrusion</th>
</tr>
</thead>
<tbody>
<tr>
<td>U75-5OD1</td>
<td>52.46</td>
<td>52.44</td>
<td></td>
</tr>
<tr>
<td>U75-5OD2</td>
<td>40.99</td>
<td>38.55</td>
<td></td>
</tr>
<tr>
<td>U75-15OD1</td>
<td>55.27</td>
<td>56.10</td>
<td></td>
</tr>
<tr>
<td>U75-15OD2</td>
<td>53.21</td>
<td>49.33</td>
<td></td>
</tr>
<tr>
<td>U75-90OD1</td>
<td>Nil</td>
<td>Nil</td>
<td></td>
</tr>
<tr>
<td>U75-90OD2</td>
<td>65.31</td>
<td>66.41</td>
<td></td>
</tr>
<tr>
<td>O90 OD1</td>
<td>61.39</td>
<td>61.94</td>
<td></td>
</tr>
<tr>
<td>O90 OD2</td>
<td>58.52</td>
<td>58.60</td>
<td></td>
</tr>
</tbody>
</table>
Figure 2-4: Shear wave velocity against volumetric shell fraction

2.4.3 Stress-strain response

Figure 2-5 shows the 1D-compression curves of two different specimens for each tube. Even though all the specimens were obtained from the same depth, the initial void ratio and the compression curves were different depending on the types of tube sampler used and the volumetric shell fraction. In each tube, OD1 specimen had a higher void ratio compared to OD2 specimen. It appears that OD2 specimens were relatively denser compared with OD1 specimens even though both specimens were from the same source. The reason for this observation is due to the specific gravity of shell fragments (2.89 g/cm³) that was higher compared to the specific gravity of Ballina clay (2.69 g/cm³). As OD2 specimens had more shell fragments, they were heavier in mass yet it occupied the same volume of the IL ring. OD1 specimens also appeared to have a lower initial slope in over-consolidated (OC) region compared to OD2 specimens. This stiff response by OD1 specimens may due to the structure and fabrics of OD1 specimens remained relatively intact and were not affected as to OD2 specimens. Only U75 90 OD2 showed odd unusual response of the compressibility curve which can be explain by the large size of the shell fragments.
Figure 2-5: Compressibility curves for each sampling tube

Figure 2-6 shows compressibility curves for specimens U75-5 OD2, U75-15 OD2 and U75-90 OD1, which have a similar $V_{sf}$, between 0.91 and 0.98%. The only difference between all these specimens was the difference in the OCA used in the tube sampler. All of them showed very similar compressibility curves, indicating that the OCA did not affect specimen compressibility significantly. The presence of shell fragments may subjugate the effects of OCA on the compressibility curves.

Figure 2-6: Compressibility curves for specimens U75-5 OD2, U75-15 OD2 and U75-90 OD1 with similar $V_{sf}$.
2.4.4 Yield stress, compression index and swelling index

Table 2-4 shows the compressibility parameters obtained for all specimens. As the soil tested was soft clay and may be subjected to various amounts of disturbance due to presence of shell fragments, four different methods of interpreting yield stress, $\sigma'_\text{yield}$, were used to increase the accuracy of the analysis. The four methods were: (i) Casagrande (1936) based on empirical observation; (ii) Becker et al. (1987) based on work per unit volume; (iii) Onitsuka et al. (1995) based on bilogarithmic approaches and; (iv) Boone (2010) based on simple slope-intercept mathematics preventing subjective interpretation. The first three methods (Casagrande 1936), (Onitsuka et al. 1995) and (Becker et al. 1987) yielded reasonably close values of $\sigma'_\text{yield}$, whereas the fourth method (Boone 2010) had a tendency to give higher values.

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>U75-5 OD1</td>
<td></td>
<td>36</td>
<td>45</td>
<td>45</td>
<td>46.3</td>
<td>43.08</td>
</tr>
<tr>
<td>U75-5 OD2</td>
<td></td>
<td>32</td>
<td>35</td>
<td>30</td>
<td>37.9</td>
<td>33.74</td>
</tr>
<tr>
<td>U75-15 OD1</td>
<td></td>
<td>39</td>
<td>35</td>
<td>30</td>
<td>52.1</td>
<td>39.05</td>
</tr>
<tr>
<td>U75-15 OD2</td>
<td></td>
<td>31</td>
<td>30</td>
<td>29</td>
<td>37.3</td>
<td>31.82</td>
</tr>
<tr>
<td>U75-90 OD1</td>
<td></td>
<td>45</td>
<td>44</td>
<td>40</td>
<td>47.5</td>
<td>44.12</td>
</tr>
<tr>
<td>U75-90 OD2</td>
<td></td>
<td>55</td>
<td>50</td>
<td>60</td>
<td>58.6</td>
<td>55.86</td>
</tr>
<tr>
<td>O90 OD1</td>
<td></td>
<td>32</td>
<td>33</td>
<td>32</td>
<td>41.7</td>
<td>34.66</td>
</tr>
<tr>
<td>O90 OD2</td>
<td></td>
<td>31</td>
<td>31</td>
<td>29</td>
<td>40.9</td>
<td>32.98</td>
</tr>
</tbody>
</table>

$\sigma'_\text{yield}$ for OD2 specimens determined using all the methods is less than for OD1 specimens, except for specimens from U75-90 tube. The higher $\sigma'_\text{yield}$ for OD1 specimen may be due to the greater depth. This agrees with observations of higher sensitivity soil which relates to the soil structure by Boukpeti and Lehane (2016). On the other hand, U75-90 OD2 specimen seems to have the highest $\sigma'_\text{yield}$ compared with all the specimens. This can be explained by the higher volumetric shell fraction with larger sizes of shell fragments compared with other specimens as shown in Figure 2-3.
To illustrate the effects of shell volume, $\sigma'$ yield obtained using Casagrande’s method is plotted against $V_{sf}$ in Figure 2-7. Except for two data points with higher $\sigma'$ yield values shown by specimen U75-90, Figure 2-7 shows that $\sigma'$ yield decreases with $V_{sf}$. This may suggest that the structure of the specimens with higher $V_{sf}$ subjected to larger disturbance and have reduction in yield stress.

![Figure 2-7: $\sigma'$ yield (Casagrande) with $V_{sf}$](image)

The compression index ($C_c$) and the swelling index ($C_s$) are tabulated in Table 2-5 and plotted in Figure 1.4-6. Small variations of $C_c$ and $C_s$ with high tendency for $C_c$ and $C_s$ to decrease with increasing $V_{sf}$ are observed. $C_c$ and $C_s$ for OD2 specimens are slightly lower than OD1 specimens. This observation maybe due to a higher clay content in OD1 specimens compared to OD2 specimens. The higher clay content resulted in higher compressibility are also shown in other past studies (Larson et al. 1980; Imhoff et al. 2004). Additionally, the depth of specimen in the tube may have influence $C_c$ and $C_s$ such that deeper specimen has higher $C_c$ and $C_s$. The effect of OCA on these parameters can be observed by comparing U75-5 OD2, U75-15 OD2 and U75-90 OD1. No obvious difference of $C_c$ and $C_s$ between U75-5 OD2 and U75-15 OD2 specimen could be observed whilst U75-90 OD1 exhibited higher values of $C_c$ and $C_s$. The effects of OCA on these parameters are inconclusive.
Table 2-5: Compression and recompression indices for all specimens

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Compression Index, $C_c$</th>
<th>Recompression Index, $C_r$</th>
</tr>
</thead>
<tbody>
<tr>
<td>U75-5 OD1</td>
<td>1.03</td>
<td>0.13</td>
</tr>
<tr>
<td>U75-5 OD2</td>
<td>0.93</td>
<td>0.09</td>
</tr>
<tr>
<td>U75-15 OD1</td>
<td>1.08</td>
<td>0.15</td>
</tr>
<tr>
<td>U75-15 OD2</td>
<td>0.89</td>
<td>0.14</td>
</tr>
<tr>
<td>U75-90 OD1</td>
<td>1.12</td>
<td>0.13</td>
</tr>
<tr>
<td>U75-90 OD2</td>
<td>0.91</td>
<td>0.07</td>
</tr>
<tr>
<td>O90 OD1</td>
<td>1.05</td>
<td>0.13</td>
</tr>
<tr>
<td>O90 OD2</td>
<td>0.86</td>
<td>0.13</td>
</tr>
</tbody>
</table>

Figure 2-8: (a) Compression index against volumetric shell fraction and (b) swelling index against volumetric shell fraction

2.4.5 Sample quality based on Lunne et al. (1997)

The sample quality of the specimen tested was determined according to the criteria proposed by Lunne et al. (1997). These criteria are based on the measurement of $\Delta e/e_0$, where $\Delta e$ is the change of void ratio taking place in the specimen during recomconsolidation to the in-situ effective vertical stress and $e_0$ is the initial void ratio of the specimen. Values of $\Delta e/e_0$ for all specimens are reported in Table 2-6 and plotted in Figure 2-9 as a function of $V_{sd}$. All specimens classify as poor quality samples and it could be due to the natural
inclusions that magnified sampling disturbance. Figure 2-9 indicates a trend of decreasing sample quality with an increase in $V_{sf}$. Additionally, when considering the data points with similar value of $V_{sf}$ ($V_{sf} \approx 0.95\%$), the results show an increase in sample quality with an increase in OCA. This observation contradicts to previous findings by others (Clayton et al. 1998) showing an increase in sampling disturbance with increasing OCA. This may suggest that other factors may have caused sampling disturbance (e.g., inclination of tube during penetration) or soil variability. It is also important to note that the criteria by Lunne et al. (1997) is never intended for materials with natural inclusions.

Table 2-6: Sample quality assessment based on Lunne et al. (2006)

<table>
<thead>
<tr>
<th>Specimens</th>
<th>Sample Quality</th>
<th>$\Delta e/e_0$</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>U75-5 OD1</td>
<td>poor</td>
<td>0.074</td>
<td></td>
</tr>
<tr>
<td>U75-5 OD2</td>
<td>poor</td>
<td>0.106</td>
<td></td>
</tr>
<tr>
<td>U75-15 OD1</td>
<td>poor</td>
<td>0.085</td>
<td></td>
</tr>
<tr>
<td>U75-15 OD2</td>
<td>poor</td>
<td>0.096</td>
<td></td>
</tr>
<tr>
<td>U75-90 OD1</td>
<td>poor</td>
<td>0.071</td>
<td></td>
</tr>
<tr>
<td>U75-90 OD2</td>
<td>poor</td>
<td>0.104</td>
<td></td>
</tr>
<tr>
<td>O90 OD1</td>
<td>good</td>
<td>0.065</td>
<td></td>
</tr>
<tr>
<td>O90 OD2</td>
<td>poor</td>
<td>0.091</td>
<td></td>
</tr>
</tbody>
</table>

Figure 2-9: Variation of sample disturbance with volumetric shell fraction
2.5 CONCLUSIONS

The purpose of this study was to investigate the influence of shell fragments on the compressibility parameters of natural soft clay and on sample quality. The study has identified that:

a. For a given tube sampler, specimen with more shell fragments tend to have lower $\sigma'_\text{yield}$, $C_c$ and $C_s$ and may be subjected to larger disturbance compared to specimens with less shell fragments.

b. No obvious difference could be observed in the soil compressibility parameters ($\sigma'_\text{yield}$, $C_c$ and $C_s$) when comparing the different tube OCA for specimens with almost similar $V_{sf}$. Effects on OCA on the soil compressibility may diminish due to presence of natural heterogeneities or other unknown disturbances.

c. Based on the preliminary results, a trend of increasing sampling disturbance with increasing $V_{sf}$ is observed.

Despite its exploratory nature, this study offers some insight into the effects of testing natural soft clay with presence of natural heterogeneities.
2.6 APPENDIX

Two methods were used to determine the volume of shell fragments in the soil specimens, namely CT-scan image analysis and wet sieving for U75 tubes only. After the completion of the IL test, the specimen was scanned using Xradia Versa Micro-CT (XRM) to obtain three dimensional images of the specimen. Due to the failure of the equipment in the middle of the study, Bruker Skyscan 1176 In-Vivo Micro CT (Skyscan) was used to scan the remaining specimens (U75-15 OD2, U75-90 OD1 and U75-90 OD2). The scanning equipment has a maximum field of view of 50 mm, with a resolution of 49 µm for XRM and 18 µm for Skyscan. The specimen was trimmed to 50 mm diameter before scanning and the trimming was used for water content test. The scanned images obtained with XRM and Skyscan were processed using the commercial data visualization and analysis software AVIZO and Brucker CT-Analyser (CTan), respectively. After scanning, the 50 mm diameter specimen was dried to determine its water content.

Image analysis is an operator-dependent tool and is also subjective to the image quality and post-processing steps. A consistent approach was used here to remove the artifacts before segmentation of the images. Segmentation was done manually by thresholding appropriate grey values to remove the clay from the 2D images. The segmented binary images were then used to generate the 3D view of the shell fragments distribution as shown in Figure 2-10. Using the analysis function in Avizo and CTan, the volume of shell in the 50 mm diameter central part of the IL specimen was computed. The values are reported in Table 2-7 together with the values obtained by sieving. It is apparent that the shell volume computed by CT-scan analysis tends to be slightly higher than the one determined by sieving, with the differences between both methods remained relatively small. The differences may be due to the difficulty in identifying the shell material accurately in the images through appropriate thresholding of grey values or inaccuracies of the wet sieving method. Overall, these results indicate that the volume of shell fragments can be obtained using these two methods. The relative difference between the volumes computed with the two methods is small if compared with the total volume of IL specimens.
Chapter 2: Testing tube specimens from soft clay deposits containing variable amounts of shells

Figure 2-10: Example of 3D view of shell fragments distribution in a specimen

Table 2-7: Volume of shell fragments and volumetric shell fraction determined using CT-scans and wet-sieving

<table>
<thead>
<tr>
<th>Sample</th>
<th>Volume of shell in 50 mm diameter specimen (mm$^3$)</th>
<th>Relative difference (%)</th>
<th>Volumetric shell fraction, Vsf, (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Sieving, $V_{sf}$</td>
<td>CT-Scan, $V_{sf,CT}$</td>
<td>$</td>
</tr>
<tr>
<td>U75-5 OD1</td>
<td>152.25</td>
<td>167.71</td>
<td>10</td>
</tr>
<tr>
<td>U75-5 OD2</td>
<td>737.72</td>
<td>652.57</td>
<td>12</td>
</tr>
<tr>
<td>U75-15 OD1</td>
<td>369.90</td>
<td>447.65</td>
<td>19</td>
</tr>
<tr>
<td>U75-15 OD2</td>
<td>253.98</td>
<td>356.84</td>
<td>18</td>
</tr>
<tr>
<td>U75-90 OD1</td>
<td>591.70</td>
<td>602.60</td>
<td>2</td>
</tr>
<tr>
<td>U75-90 OD2</td>
<td>1151.90</td>
<td>1037.10</td>
<td>11</td>
</tr>
</tbody>
</table>
2.7 REFERENCES


CHAPTER 3. EFFECTS OF SAMPLING DISTURBANCE IN GEOTECHNICAL DESIGNS

Abstract: This chapter describes an experimental study of the effects of sampling disturbance in an Australian natural soft clay and the consequences of different sample quality on the representativeness of soil parameters used in geotechnical designs. The paper is divided into three sections. Laboratory test results obtained from specimens retrieved using three different tube samplers as well as the Sherbrooke (block) sampler are first described. Then, the sample quality assessment, using available indices proposed for soft soils, is presented. It is shown that sample quality varies with the stress paths and boundary conditions applied in laboratory tests. Finally, mechanical soil properties derived from specimens retrieved using the different samplers are used in the prediction of two classical problems in soil mechanics: the settlement and excess pore pressure response underneath an embankment as well as the settlement and bearing capacity of a shallow footing. These two examples are used here to highlight the consequences of poor sampling in practice.
3.1 INTRODUCTION

The Embankment Prediction Symposium (CGSE, 2016), recently organized in Newcastle (NSW, Australia) by the ARC Centre of Excellence for Geotechnical Science and Engineering (CGSE), aimed to predict the behaviour of an embankment built on natural soft clay improved with vertical drains (PVDs) from the Ballina field testing facility, located in northern NSW. Results of detailed in situ and laboratory characterization studies (Pineda et al., 2016a; Kelly et al., 2017) were provided to Australian and international predictors, including practitioners and academics. The laboratory study was carried out on tube specimens retrieved using a fixed piston sampler (89 mm diameter), which provided soil specimens of excellent as well as good to fair quality according to available sample quality indexes for soft clays (Lunne et al., 1997).

The fixed piston sampler was preferred instead of other sampling techniques (including block sampling) for two main reasons. First, block sampling is a cost-prohibitive technique almost never used in common practice. Second, fixed piston samplers are known for providing specimens of better quality compared with other tube sampling techniques such as open samplers (Shelby tubes) and free piston samplers (e.g., Andersen and Kolstand, 1979; Lacasse et al., 1985; Tanaka et al., 1996; Lunne et al., 1997; Ladd & DeGroot, 2003; Lunne et al., 2006; Landon et al., 2007; Donohue & Long, 2010; Pineda et al., 2016b; to name a few). Despite their well-documented advantages, the use of fixed piston samplers is still not well established in practice. In Australia, for instance, Shelby tubes (50 – 75 mm in diameter) are the typical choices, mainly due to their simple operational principle. However, mechanical parameters obtained from laboratory tests performed on Shelby tubes in most cases do not represent the in situ conditions. Laboratory testing is expensive and involves long testing times, so additional sampling and laboratory campaigns tend to be avoided in practice, even if the quality of soil specimens does not fulfil the recommendations given in the literature. Therefore, there is a concern about the influence of soil disturbance as a result of sampling on predicted soil behaviour for typical geotechnical infrastructure.

For this study, samples were obtained from the same depth at the Ballina field testing facility using four different samplers in order to assess the level of sample disturbance and its effect on the mechanical parameters of natural Ballina clay. In addition, the consequences of the sampling disturbance on the predicted settlement and
excess pore pressure underneath an embankment as well as the predicted settlement and bearing capacity of a shallow footing are presented. These two classical problems provide a good benchmark exercise for the quantification of the error(s) caused by poor sampling in geotechnical design.

### 3.2 TESTED SOIL AND SAMPLING PROGRAM

Ballina clay is a structured and lightly overconsolidated (OCR<2) marine clay (Holocene age) encountered at the Ballina field testing facility, in northern New South Wales (Australia). Ballina clay is mainly composed of kaolinite, illite, quartz, illite-smectite and amorphous minerals. It has an activity of 1.0 and a plasticity index that ranges between 30% and 85% (i.e., high to extremely high plasticity). Site characterization studies (Pineda et al., 2016; Kelly et al., 2017) divided the soil profile at Ballina site into four main layers (see Figure 1). Layer 1 corresponds to a sandy clayey silt alluvial crust (~1.5 m thick). Layer 2 is the upper estuarine silty clay strata (~2.5 m thick) for which plasticity and void ratio increases (dry density decreases). Layer 3 corresponds to a lower estuarine silty clay layer (~7 m thick) with nearly constant soil properties which overlies a 4 m thick clayey silty sand transition layer (Layer 4). As observed in Figure 3-1, the in situ water content and dry density vary with depth from 20% to 120% and between 1.50 and 0.70 Mg/m³, respectively. The soft clay layer encountered between 2 m and 11 m depth has a predominant clay fraction with maximum values up to 82%. A non-oriented fabric was distinguished using scanning electron microscopy analysis (SEM), which is consistent with its low permeability anisotropy (see Kelly et al., 2017).
Specimens tested in this study were obtained using three different tube samplers as well as the Sherbrooke (block) sampler (Lefebvre and Poulin 1979). Figure 3-2 shows the plan view of the Ballina testing facility including the location of the boreholes (drilled using mounted rigs) which corresponds to the distribution of the instrumentation used for monitoring the performance of two embankments, named as PVD and No PVD. Also included in this figure are the location of the seismic flat dilatometer test SDMT-8, cone penetration (CPTu) and field vane (FV) tests, which are used in the analysis described below. Samples were retrieved from a depth of 5.5 - 6.1 m as indicated in Figure 3-1. The U75 Shelby tube (75 mm diameter) was used in borehole Inclo 4 whereas the free piston sampler P100 (100 mm diameter) was employed in borehole Inclo 1. The O89 fixed-piston sampler (89 mm diameter) was the choice in borehole Standpipe. Finally, a block specimen was retrieved from borehole BH1 using the Sherbrooke sampler (250 mm diameter) which has been recognized to provide the highest sample quality in soft clays (e.g., Hight et al., 1992; Lunne et al., 1997; Lunne et al., 2006). The dimensions and characteristics of the tube samplers are summarized in Table 3-1. A schematic view is given in Figure 3-3. All samplers have an area ratio (AR) lower than 14 % which is less than a typical thick-walled composite piston sampler. The ratio between external diameter and thickness of the tube wall (D_e/t) varies between 33.3 (P100) to 50.7 (U75). The outer
cutting angle (OCA) for the U75 and O89 tubes is 15° and 5°, respectively. The OCA for the P100 tube is 90° as it represents the standard version available in the market. It can be noted that according to the suggestions for tube sampling given by Ladd and De Groot (2003) (D_e/t > 40; OCA < 10° and AR < 10%), the U75 sampler should provide (a priori) the highest sample quality whereas the lowest quality should be obtained with the P100 free-piston sampler. After sampler retrieval, tube ends were properly sealed in-situ with several layers of plastic film underlying a 10mm-thick polystyrene (porexpan) plate covered externally with wax (10-15mm thickness). The porexpan plate was intended to isolate the clay from the thermal gradient. Silicone grease was applied at the interfaces between the tube sampler and the porexpan plate for improving sealing. Tube ends were finally covered with plastic lids prior to packing for transport. The block specimen was sealed using several layers of plastic film and aluminium foil prior to waxing. Specimens were placed in sealed plastic containers on a layer of wet sand aimed at minimizing moisture losses by creating a high relative humidity environment (RH= 99 %). The tubes and the block were packed using scraps of polystyrene beads to induce lateral confinement and to absorb vibrations during transport.

Table 3-1: Dimensions of the samplers used in this study

<table>
<thead>
<tr>
<th>Sampler</th>
<th>Type</th>
<th>Length, L</th>
<th>Outer diameter, D_e</th>
<th>Thickness, t</th>
<th>Outer cutting angle, OCA</th>
<th>D_e/t</th>
<th>Area ratio, AR</th>
</tr>
</thead>
<tbody>
<tr>
<td>U75</td>
<td>Open sampler</td>
<td>600</td>
<td>76</td>
<td>1.5</td>
<td>15</td>
<td>50.7</td>
<td>8.4</td>
</tr>
<tr>
<td>P100</td>
<td>Free piston</td>
<td>600</td>
<td>100</td>
<td>3.0</td>
<td>90</td>
<td>33.3</td>
<td>13.2</td>
</tr>
<tr>
<td>O89</td>
<td>Fixed piston</td>
<td>700</td>
<td>89</td>
<td>2.0</td>
<td>5</td>
<td>46.8</td>
<td>9.6</td>
</tr>
<tr>
<td>Sherbrooke</td>
<td>Block sampler</td>
<td>350</td>
<td>250</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
3.3 EXPERIMENTAL PROGRAM AND PROCEDURES

Wave propagation tests using bender element (BE) transducers, resonant column (RC) tests, one-dimensional constant rate of strain (CRS) compression tests and simple shear (SS) tests with cell pressure confinement were carried out in this study to evaluate the stiffness, compressibility and strength parameters of Ballina clay specimens. The allocation of specimens is indicated in Figure 3-4. Tube samples were sliced horizontally
before trimming to minimize the adhesion force at the clay-tube interface during specimen extrusion. A brief description of the testing procedures is given below.

Figure 3-4: Specimen allocation in a tube (U75, P100 and O89 tube specimens) and block samples.

### 3.3.1 Bender element tests

Shear wave velocity was evaluated on sectioned tube slices devoted to one-dimensional CRS compression tests, before and after soil extrusion, using BE transducers manufactured at The University of Western Australia (UWA). BE inserts were attached at the top and bottom of the specimen during the measurement of the shear wave velocity. A sine pulse of peak to peak voltage, $V_{pp}$ of 20 V with apparent frequency $f_{app} = 5$ kHz was used as input signal. The effective length, $l_{eff}$, was determined as the tip-to-tip distance between the piezoceramic elements. The travel time, $t_s$, was estimated as the time interval between the first peak of the transmitted and received signal. The testing procedure followed the protocol described in Viggiani and Atkinson (1995). The shear wave velocity was estimated as $V_{s(ij)} = l_{eff}/t_s$, where subscripts $i$ and $j$ refer to the wave propagation and polarization directions, respectively.

### 3.3.2 Resonant column tests

A Stokoe fixed-free type of resonant column apparatus (GDS Instruments Ltd., UK) was used to assess the small-strain shear modulus, $G_{(vh)}$, as well as the damping ratio, $D$, of Ballina clay specimens. The apparatus is similar to that described in Clayton (2011) and was designed to minimize equipment damping. The RC tests were controlled via software using the GDSLAB and GDSRCA packages. Specimens tested were 140 mm in height and 72 mm in diameter. Specimens were saturated under 5 kPa isotropic effective
stress by applying a back pressure of 500 kPa in stages. Low isotropic effective stress was used to prevent over stressing the specimen during saturation. Values of Skempton’s B parameter higher than 0.96 were measured in all specimens after 1.5 days of saturation under the back pressure of 500 kPa. Specimens were then isotropically consolidated to an in situ mean effective stress \( p'_{\text{in situ}} = \left( \sigma'_{v} + 2\sigma'_{h} \right) / 3 = 31 \text{ kPa} \), estimated from vertical and horizontal in situ effective stresses (\( \sigma'_{v,\text{in situ}} = 42 \text{ kPa} \) and \( \sigma'_{h,\text{in situ}} = 25.2 \text{ kPa} \), for \( K_0 = 0.60 \)) according to Kelly et al. (2017a). Consolidation stress increments were applied in 5 kPa increments. The RC testing procedure followed the recommendations outlined by ASTM Standard D4015 (2007). A sinusoidal torsional vibration was applied to the top of the specimen to allow propagation of shear waves. The shear wave velocity was estimated as:

\[
V_{s(vh)} = \frac{2\pi f l}{\beta}
\]

(3-1)

where \( f \) is the torsional resonant frequency, \( l \) is the length of the specimen and \( \beta \) is the device’s calibration factor which accounts for the mass moment of inertia of the specimen as well as the mass moment of inertia of the components mounted on the specimen (i.e., drive system, accelerometer, top platen and counterweight masses). The damping ratio, \( D \) was estimated from the logarithmic decrement of free vibration decay curve, \( \delta \), according to:

\[
D = \sqrt{\frac{\delta^2}{4\pi^2 + \delta^2}}
\]

(3-2)

### 3.3.3 One-dimensional constant rate of strain compression tests

CRS tests were carried out to evaluate the compressive behaviour of Ballina clay. CRS tests followed the recommendations given by ASTM D4186 (2006). Cylindrical specimens (50 mm in diameter and 24 mm in height) were tested using a Trautwein cell in combination with a computer-controlled loading frame. Specimens were saturated for 48 hours under a vertical effective stress of 10 kPa. Back pressure of 20 kPa was applied to saturate the specimen without over-stressing it. Then, the vertical load was applied under a constant strain rate of 1.0 %/h until a maximum vertical effective stress of 500 kPa was reached. Single drainage was allowed during compression while the excess pore water pressure was measured at the specimen base. The adopted strain rate produced
excess pore water pressures lower than 20 % of the total vertical stress in all tests except for one test when a maximum value of 30 % was reached.

### 3.3.4 Simple shear tests with cell pressure confinement

Undrained monotonic simple shear tests with cell pressure confinement were performed to estimate the undrained shear strength of Ballina clay specimens. These SS tests were carried out in the UWA simple shear apparatus similar to that described by Mao and Fahey (2003). In this apparatus, specimens with dimension and height equal to 72 and 40 mm respectively were enclosed in an unreinforced latex membrane. During the shearing phase, the total vertical stress was kept constant by varying the cell pressure via software and the sample height were maintained constant by mechanically locking the vertical actuator. The testing procedure included three stages: saturation, anisotropic consolidation and undrained shearing. A back pressure equal to 400 kPa was ramped during the saturation stage while maintaining a mean effective stress of 10 kPa on the specimen. Skempton’s B parameter was higher than 0.98 in all tests after 1 day of saturation. Specimens were then consolidated to their in situ stress state defined by $\sigma'_\text{axial} = 42$ kPa and $\sigma'_\text{radial} = 25.2$ kPa, as discussed above. An equalization time of 24 h was allowed for all specimens after achieving the in situ stress state. Finally, shearing under undrained conditions was conducted using a horizontal strain rate of 2%/h, following the recommendations by ASTM D6528 (2007) using $c_v$ determined from CRS tests.

### 3.4 RESULTS

#### 3.4.1 Shear wave velocity

Figure 3-5 shows the shear wave velocity, $V_s$ determined from block and tube specimens devoted to CRS testing. $V_s$ with horizontal polarization was measured in both vertical ($V_s(\text{vh})$) and horizontal ($V_s(\text{hh})$) directions. In the case of tube specimens, vertical measurements were made before ($V_s(\text{vh}\text{-tube})$) and after specimen extrusion ($V_s(\text{vh}\text{-extruded})$). The comparison against in situ values obtained from seismic flat dilatometer test SDMT-8 (Figure 3-5 (a)) shows that $V_s(\text{vh})$ is more strongly affected by sampling than $V_s(\text{hh})$. The largest stress relief due to sampling, which takes place in the vertical direction, seems to be responsible for the highest reduction in $V_s(\text{vh})$. The fact that $V_s(\text{hh}\text{block}) \approx V_s(\text{vh}\text{SDMT})$, supports the observation that sampling due to stress relief affects $V_s(\text{vh})$ more than $V_s(\text{hh})$. It is important to note that despite the sedimentary nature of the clay, which could explain
the higher values of $V_{s(hh)}$, scanning electron microscopy (SEM) analysis has shown a non-oriented fabric for natural Ballina clay. In fact, values of $V_{s(vh)}$ similar to those obtained from SDMT have been measured in CRS tests at $\sigma'_v = \sigma'_{v-in situ}$ by Pineda et al. (2016a) on specimens obtained using a similar sampler as O89. Hence, \textit{in situ} values obtained from SDMT can be used as good references as the results have shown consistent agreement with shear wave velocity from bender element measurement in the laboratory (Pineda et al., 2016a). Block and O89 specimens show the highest and second highest shear wave velocities respectively for both directions while the P100 and U75 samples show the lowest $V_{s(vh)}$ and $V_{s(hh)}$, respectively. Soil extrusion produces an additional reduction in Vs as indicated by $V_{s(vh)}$-extruded. This is more evident in the case of the P100 sample. The Vs normalised by $V_{s(vh)}$SDMT is plotted in Figure 3-5 (b) and shows that sampling causes a reduction in $V_{s(vh)}$ between 15% for O89 and 31% for P100 free-piston sampler whereas in the case of $V_{s(hh)}$ it decreases by 28% for U75 Shelby sampler.

![Figure 3-5: Variation of (a) shear wave velocity and (b) normalised shear wave velocity with sampler diameter](image-url)
3.4.2 Small-strain stiffness

Figure 3-6 shows the variation of the small-strain shear modulus measured in the vertical direction, $G_v$, with the shear strain, $\gamma$, obtained from RC tests for block, P100 and U75 specimens. Poor coupling between the pedestal and the O89 specimen led to some erroneous measurements, which are omitted in Figure 3-5 for consistency. The RC test for the P100 specimen was stopped at $\gamma=0.02\%$ due to malfunctioning of the resonant column device. It caused a sudden jump in the resonant frequency that led to values of $G_{\text{max}}$ around 2-3 times larger the ones measured for the block sample. Because of this unjustified response, only values of $G_{\text{max}}$ below $\gamma=0.02\%$ are reported in Figure 3-6 for this specimen. Values of $G_{(vh)}$, estimated from BE tests using the bulk density of CRS specimens, are included in Figure 3-6 (a) for comparison. The comparison between BE and RC results shows an an important increase in $G_{(vh)}$ is observed due to sample recompression to $p_{\text{in-situ}}$ in the RC apparatus. Similar values to the in situ small-strain stiffness (SDMT-8) are reported for the block specimen whereas lower values are observed for specimens P100 and U75. This behaviour confirms the effect of tube sampling on soil structure. For the same strain level, values of $G_{(vh)}$ in specimens P100 and U75 are approximately 1.5 MPa and 2.50 MPa lower than the block sample, respectively.

The degradation of the small-strain stiffness with shear strain is clearly observed in Figure 3-6(b) despite shear strains smaller than 0.5 % were applied in RC tests. The normalized stiffness ($G_{(vh)}/G_{(vh)\text{max}}$) was fitted using the hyperbolic relationship (Hardin and Drnevich 1972):

$$\frac{G_{(vh)}}{G_{(vh)\text{max}}} = \frac{1}{\left(1 + \frac{\gamma}{\gamma_{\text{ref}}}\right)} \quad (3-3)$$

where, $\gamma_{\text{ref}}$ is a reference shear strain. Two main trends may be observed in this figure which are represented by reference shear strains of 0.375 % (block) and 0.10 % (U75). Although the experimental data for specimen P100 seems to show a similar trend to the block sample up to $\gamma=0.02\%$, the higher level of destructuration experienced by the P100 specimen (see following section) makes it plausible to expect quicker stiffness degradation at larger strain levels as it was subjected to a higher level of sampling disturbance. The shear strain required to reduce the initial shear modulus to 0.70 $G_{(vh)}$ in
the U75 specimen is about one order of magnitude lower (i.e., 0.033 %) than the block sample (0.21%). This shows the larger destructuration induced by the U75 Shelby tube.

3.4.3 Variation of soil damping with shear strain

The variation of the soil damping $D$ with the shear strain is shown in Figure 3-6(c). For $\gamma<0.005 \, \%$, $D$ seems to increase linearly up to 3 %, irrespective of the sample provenance. It is apparent that at small strains, soil damping is not affected by the type of sampler but different trends are observed at large shear strains. The U75 specimen reaches values up to 13.35 % whereas the value for the block sample remains lower than 4 %. It can be seen that $D$ reduces after reaching a maximum value in the U75 specimen. Although no clear explanation has been found it is speculated here that this behaviour may be due to modifications in the soil fabric caused by shear distortion on a partially disturbed clay sample. The specimen P100 shows values closer to the block specimen for $\gamma<0.013 \, \%$ and then a deviation from this trend is observed. Ashmawy et al. (1995) observed minor influence of sampling disturbance on damping ratio at small strain but significant effects at large strain on reconstituted kaolinite slurry. The disturbed specimen appeared to have lower damping ratio curves, which is in disagreement with the results presented in this study. A possible reason for such discrepancy may be attributed to the fact that Ashmawy et al. induced sample disturbance by applying freezing-thawing cycles rather than tube sampling. On the other hand, laboratory results obtained by Safaqah & Riemer (2006), who employed two different tube samplers for inducing soil disturbance, showed good consistency with the behaviour presented in this paper. These results highlight the negative effect of some types of tube samplers on soil damping, an aspect that has been overlooked in previous studies despite its relevance for seismic design of geotechnical infrastructure. Further research is still needed in this field in order to obtain conclusive results.
3.4.4 Compressibility

The results of the CRS compression tests are presented in Figure 3-7. Vertical dashed lines are used in this figure to indicate the in situ vertical effective stress, $\sigma'_{\text{v,in situ}}$, representative of the specimens. Figure 3-7 (a) shows the variation of the volumetric
strain with the vertical effective stress. Block and O89 specimens show the lowest volumetric deformation at $\sigma'_\text{v-in situ}$ (less than 1.7 %) followed by the P100 and U75 specimens with 3.9 % and 4.2 %, respectively. The small deformation that has been required to reach $\sigma'_\text{v-in situ}$ implies less degree of soil destructuration. This is consistent with the larger yield stress, $\sigma'_\text{yield}$ as well as the steeper slope of the compressibility curve post-yielding observed in block and O89 specimens, as shown in Figure 3-7(b). Grey stars are used in this figure to indicate the $\sigma'_\text{yield}$, estimated using the Casagrande method. $\sigma'_\text{yield}$ varies from 65 kPa to 20 kPa. The value obtained from the U75 specimen, which is even lower than the $\sigma'_\text{v-in situ}$ (i.e. OCR<1), is incompatible with the stress history of the marine deposits at the Ballina site. Specimen P100 shows a $\sigma'_\text{yield}$ slightly higher than $\sigma'_\text{v-in situ}$, thus, much lower than the level of overconsolidation ratio (OCR) reported by Pineda et al. (2016a). Sampling disturbance is also noted in terms of the oedometric modulus, $M=\frac{d\sigma'_\text{v}}{d\varepsilon}$, mainly in the overconsolidated range (see Figure 3-7(c)). The initial oedometric modulus reduces from 2.72 MPa (block sample) to 0.25 MPa (U75 sample). Block and the O89 specimens show a similar trend which is followed by the P100 specimen only at larger stress levels.

Figure 3-8 shows the variation of the compression index, $C_c=-\frac{\partial \varepsilon}{\partial \log(\sigma'_\text{v})}$, with the vertical effective stress normalized by the appropriate $\sigma'_\text{yield}$ for each specimen in the normal compression range only. The strong non-linearity of $C_c$ with $\sigma'_\text{v}$ is characterized by a peak value, reached between 1.20 – 2.0 $\sigma'_\text{yield}$. A constant value is reached at large stresses once the natural soil structure has been erased. As expected, strong non-linearity is observed in the block and O89 specimens, with peak values of 2.95 and 2.25, respectively. Lower peak values of $C_c$ are reported for P100 and U75 specimens (1.83 and 1.38, respectively) for which a more gentle post peak reduction in $C_c$ is observed. The post peak value seems to be similar for all specimens tested ($C_c \approx 1.30$), except for the U75 sample ($C_c \approx 1.0$).
Chapter 3: Effects of sampling disturbance in geotechnical designs

Figure 3-7: CRS compression test results
Chapter 3: Effects of sampling disturbance in geotechnical designs

Figure 3-8: Variation of $C_c$ with normalised stress level

Table 3-2 shows the estimated coefficient of consolidation, $c_v$, obtained from CRS tests according to ASTM D4186 (2006). Values reported in Table 3-2 correspond to the $\sigma'_v$-in situ, $\sigma'_\text{yield}$ as well as the value measured at $\sigma'_v$ of 100 kPa. The $c_v$ decreases dramatically during one-dimensional compression and leads to a value around 0.20 m$^2$/year for an effective vertical stress around 100 kPa.

<table>
<thead>
<tr>
<th>Vertical stress parameter (kPa)</th>
<th>$c_v$ (m$^2$/year)</th>
<th>Block</th>
<th>P100</th>
<th>O89</th>
<th>U75</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\sigma'_v$-in situ</td>
<td>2.7</td>
<td>3.0</td>
<td>6.0</td>
<td>0.28</td>
<td></td>
</tr>
<tr>
<td>$\sigma'_\text{yield}$</td>
<td>1.7</td>
<td>1.5</td>
<td>3.0</td>
<td>0.7</td>
<td></td>
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<tr>
<td>$\sigma'_v$, 100 kPa</td>
<td>0.17</td>
<td>0.2</td>
<td>0.7</td>
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</tr>
</tbody>
</table>

3.4.5 Undrained shear strength

Figure 3-9 shows the temporal evolution of volumetric strain during the anisotropic consolidation stage at targeted K stress ratio in SS tests. Block and O89 specimens show a maximum volumetric strain around 2.5 %. This value is 2 and 3.3 times smaller compared with the deformation measured in P100 and U75 specimens, respectively. The
results of the undrained shearing stage are shown in Figure 3-10. Peak shear stresses, $\tau_{xy}$-peak, of 16.9 kPa, 16.7 kPa, 16.2 kPa and 14.6 kPa were measured in block, O89, U75 and P100 specimens, respectively. Despite the small variation in $\tau_{xy}$-peak between specimens, important differences in the stress-strain response are observed in Figure 3-10. A relatively brittle post-peak response, commonly displayed by structured soils, is shown by block and O89 specimens. Ductile behaviour is observed in P100 and U75 specimens. Values of shear strain at peak demonstrate the differences in stress-strain responses between block (3.87 %) and U75 (15.5 %) specimens. On the other hand, similar excess pore water pressure response is observed in Figure 3-10, at least up to about 5% shear strain. The increase in excess pore water pressure observed in the block specimen just after reaching $\tau_{xy}$-peak, appeared to be due to the propagation of a shear band.

The variation of the secant friction angle, $\phi_{sec}$, with the shear strain is presented in Figure 3-11. Two main trends are observed. Peak friction angles varying from 44° to 41° are observed in O89 and block specimens, respectively. At shear strains larger than 15 %, the friction angle reaches an average value of 36° for all specimens as also reported by Pineda et al. (2016a). This value is similar to the critical state friction angle (constant volume conditions) reported by Mayne (2016) for a variety of natural soft soils such as Bothkennar clay (UK), Onsoy clay (Norway) and Burswood clay (Australia).

Figure 3-12 shows the variation of the secant shear modulus, $G_{u,sec}=\Delta \tau_{xy}/\Delta \gamma$, estimated from the stress-strain curves presented in Figure 3-10. Values around 3 MPa were measured for block and O89 specimens at shear strains around 0.02 %. At the same strain level, $G_{u,sec}$ reduces to 2 MPa for the P100 specimen. The U75 specimen shows lower values of shear secant modulus compared with block and O89 specimens. This does not reflect the strong difference in volumetric compression measured during consolidation, which suggest larger soil disturbance. In fact, it appears that soil destructuration caused by tube sampling using the Shelby tube is to some extent, at least in terms of soil stiffness, compensated by the large volumetric compression (densification) occurred during consolidation. More discernible differences in soil stiffness are observed between block and O89 specimens with respect to the P100. It might indicate that the stiffening effect produced by the reduction in void ratio during consolidation was smaller in this case than the destructuration caused by tube sampling.
Figure 3-9: Volumetric strain during $K_0$ consolidation

Figure 3-10: Results from the shearing stage in simple shear tests
Chapter 3: Effects of sampling disturbance in geotechnical designs

Figure 3-11: Variation of the secant friction angle with the shear strain

Figure 3-12: Stiffness degradation curves
3.5 SAMPLING DISTURBANCE ASSESSMENT

The proposal by Lunne et al. (1997) was adopted in this study to assess sample quality. This method is based on the change in void ratio required to recompress the soil to the *in situ* vertical effective stress in triaxial and CRS tests, normalized by the initial void ratio ($\Delta e/e_0$). For soils with OCR<2, Lunne et al. divide the sample quality into four levels as follows: (1) excellent ($\Delta e/e_0 < 0.04$), (2) good to fair ($0.04 < \Delta e/e_0 < 0.07$), (3) poor ($0.07 < \Delta e/e_0 < 0.14$) and (4) very poor ($\Delta e/e_0 > 0.14$).

Figure 3-13 compares the sample quality descriptor ($\Delta e/e_0$) estimated for all tested specimens against the sampler diameter. It is clear that the block (Sherbrooke) and the fixed piston (O89) sampler consistently provide samples of excellent quality for all element testing. Extreme variation in sample quality, from very poor to excellent, is obtained from Shelby (U75) samplers, while the free piston sampler (P100) provides samples of excellent and good to fair.

Figure 3-13: Variation of the sample quality index with the sampler diameter

Figure 3-14 shows the variation of the shear wave velocities $V_{S(vh)}$ and $V_{S(hh)}$, measured in BE tests, against $\Delta e/e_0$. Shear wave velocities have been normalized against the value obtained from SDMT-8 test. As shown in Figure 3-5, $V_{S(hh)}$ is less affected by sampling than $V_{S(vh)}$, at least for unstressed conditions. Both $V_{S(vh)}$ and $V_{S(hh)}$ show values higher than 0.70 $V_{S(in situ)}$. Once again, sample derived from block and O89 sampling show superior quality compared to other sampling techniques. A trend between the decrease
in $V_s$ with increasing $\Delta e/e_0$ may only be depicted for $V_s(hh)$. However, further experimental data is required to get conclusive results.

**Figure 3-14: Normalized shear wave velocities vs $\Delta e/e_0$**

Ambiguous results are shown in Figure 3-15, which shows the variation of modulus, $G$, and $D$, as a function of $\Delta e/e_0$ measured in RC tests. Values of $D$ plotted in this figure correspond to $\gamma = 0.05\%$. All tested specimens have *excellent* quality based on Lunne et al.’s. criterion, despite the important variation in $G$ reported in this figure. $G$ reduces in U75 and P100 specimens by about 45 % and 28 %, respectively, compared with the block ($G_{(block)} \approx G_{(SDMT)}$). Values of $D$ up to four times larger are clearly indicative of specimens with different quality. This potentially misleading relationship between the shear modulus and damping ratio measured in the RC device and $\Delta e/e_0$ may be attributed here to the stress path applied by the RC apparatus during consolidation. The isotropic recompression path followed in RC tests does not represent the true anisotropic stress state of the soil, due to the absence of the deviatoric component which contributes, to a large extent, to the deformation behaviour of the clay.

Figure 3-16 compares the estimated $\sigma'_{yield}$, $M$ and $C_{c\text{-peak}}$ against $\Delta e/e_0$ obtained from CRS tests. The trend shown in Figure 3-16 is qualitatively consistent with published data, i.e. the estimated $\sigma'_{yield}$, initial $M$ and $C_{c\text{-peak}}$ reduces with decreasing sample quality. $\sigma'_{yield}$ reduces 30 % and 67 % in P100 and U75 specimens respectively. The reduction of $M$ and $C_{c\text{-peak}}$ is even more dramatic. It may be noted that the P100 specimen has a *good*
to fair quality according to the adopted sample quality criterion. This would imply that results from this specimen could be used with confidence in design, clearly a misleading outcome judging by the comparison against mechanical properties measured in either block or O89 specimens.

![Graph showing variation of $G_0$ and $D$ with $\Delta e/e_0$](image)

**Figure 3-15: Variation of $G_0$ and $D$ with $\Delta e/e_0$**

The sample quality assessment for the specimens tested under simple shear conditions is shown in Figure 3-17. Peak undrained shear strength, $\tau_{xy\text{-peak}}$, shear strain at peak, $\gamma_{\text{peak}}$, as well as the undrained secant modulus at 50% of $\tau_{xy\text{-peak}}$ are plotted against $\Delta e/e_0$. Despite the small variation of $\tau_{xy\text{-peak}}$ with sample quality, the effects of sampling disturbance are clearer in terms of the stress-strain responses, which vary from brittle to ductile behaviour (see Figure 3-10). This is reflected in the value of shear strain measured at $\tau_{xy\text{-peak}}$ which increases up to 4 times in the U75 specimen. The undrained shear strength for U75 being not much lower than the other samplers especially the block and O89. Even though the sample was clearly quite significant disturbed, it nevertheless underwent a significant decrease in void ratio during consolidation. This eventually resulted in an increase of undrained shear strength, counteracting the loss in undrained shear strength.
due to sampling induced destructuring. The P100 specimen classifies here as good to fair quality. This is a contradictory result based on the comparison of its mechanical response against block and O89 specimens.

![Graph showing variation of σ', M, and Cc-peak with Δe/e₀](image)

**Figure 3-16**: Variation of σ', yield, M and Cc-peak with Δe/e₀
3.6 IMPLICATIONS IN GEOTECHNICAL DESIGNS

Although very good agreement between mechanical properties is shown for block and O89 specimens both of which had a sample quality index $\Delta e/\varepsilon_0$ of ‘excellent’, misleading correlations are observed for specimens retrieved with free-piston (P100) and the Shelby (U75) samplers. This fact suggests some concerns about the reliability of soil parameters used in geotechnical design, where these samplers are used. As noted in the
Introduction, fixed-piston samplers are not widely used in practice whereas the U75 Shelby sampler is the most common choice, especially in Australia.

Figure 3-18 compares values of yield stress (σ’yield), compression index (C_{c-peak}), coefficient of consolidation (c_v at σ’yield) and undrained shear strength (S_u) obtained for each sampler type against values obtained in the laboratory (Pineda et al., 2016) and in situ (Kelly et al., 2017) characterization studies carried out at Ballina site. Laboratory tests reported by Pineda et al. (2016a), obtained from tube samples retrieved using the O89 sampler, provided specimens of ‘excellent’ and ‘good’ quality. Figure 3-18 reports values of coefficient of consolidation estimated from CRS tests, incremental loading (creep) tests as well as values of c_v obtained from CPTu tests. Undrained shear strength estimated from field vane (VT) tests, which provide similar values to those obtained from simple shear tests (e.g., Terzaghi et al., 1996; Kouretzis et al., 2017), are also reported in Figure 3-18 for comparison. Very good agreement is observed between laboratory values and in situ characterization studies and parameters estimated here for block and O89 samplers. This indicates that reliable predictions would be obtained by using soil parameters estimated from these samplers, particularly in the case of the block sampler. On the other hand, soil parameters estimated using P100 and U75 samplers are markedly lower compared to laboratory and in situ characterization studies (except for the undrained shear strength) which (a-priori) would lead to poor predictions.

Figure 3-18: Comparison between soil properties obtained in this study with parameters estimated from laboratory (Pineda et al. 2016a) and in situ (Kelly et al. 2017) characterisation studies.
To assess the practical implications of sampling disturbance, two classical problems in soil mechanics are analysed in this paper, using the sets of soil parameters estimated in this study from each sampler type. To do so, the geometry of the trial embankment and the trial footing constructed at Ballina site by the ARC Centre of Excellence for Geotechnical Science and Engineering (CGSE) were adopted here (Figure 3-19). The embankment, constructed after installing vertical drains (PVDs) for accelerating soil consolidation, and the shallow footing were used in an International Prediction Symposium (CGSE, 2016) whose outcomes have been recently published by Kelly et al. (2018) and Doherty et al. (2018), respectively. With the aim of emphasizing the effects of sampling disturbance on predicted soil behaviour a few simplifications were made in the predictions presented below. Vertical drains were neglected whereas a single layer soil profile was adopted. Moreover, the original square footing was modelled as strip footing to use parameters from SS for plane strain condition.

Although these assumptions made a difficult direct comparison between predicted and in situ measurements, the excellent quality and representativeness of the soil parameters obtained using the block sample indicates that those predictions would represent the in situ behaviour, for the particular scenario under study.

![Figure 3-19: (a) Embankment problem and (b) footing problem](image)
3.6.1 Settlement and excess pore pressure prediction underneath an embankment

Figure 3-19(a) shows the adopted geometry of the embankment, the same as the one recently constructed at the Ballina field testing facility (Kelly et al., 2017b). The nominal dimensions of the embankment are: 27 m long at the base, 15 m long at the crest, 3 m high and nominal inclination of 2H:1V. The total vertical surcharge applied by the embankment at the ground surface is 63 kPa. A simplified soil stratigraphy using one single layer (10.5 m in thickness) is adopted here for simplicity. No prefabricated vertical drains are considered in this exercise.

Predictions of settlement as well as excess pore pressure underneath the centreline of the embankment were made using the four sets of parameters obtained from CRS tests (1 per sampler type) summarized in Table 3-3. The following predictions were made (see also Appendix 1):

- End of primary consolidation settlement due to the weight of the embankment using 1-D consolidation theory (Equations 3-A1 and 3-A2 in Appendix 1).
- Time variation of excess pore water pressure, degree of consolidation and consolidation settlement due to the construction of the embankment via the Finite Difference Method (FDM) (Equations 3-A3 to 3-A5 in Appendix 1).
- Secondary compression settlement using the approach by Mesri and Choi (1985) which is based on 1-D consolidation theory (Equation 3-A6 in Appendix 1). Because the effects of sampling disturbance on the secondary compression coefficient, $C_\alpha$, were not evaluated within the experimental program described above, it was decided to adopt a constant value for the prediction exercise irrespective of the sampler type, which is equal to 0.05 (Pineda et al., 2016a).

Based on the value of $\sigma'_\text{yield}$ obtained from the block specimen, which seems to represent more closely the in situ conditions, the entire soil profile will move from overconsolidated (OC) to normally consolidated (NC) conditions at the end of construction of the embankment ($\sigma'_{\text{final}} = 105$ kPa). However, it will take some time for each soil layer to reach equilibrium in effective stresses due to the consolidation experienced by the clay. This is particularly important for the selection of $c_v$ to be used in the prediction due to the fact it reduces dramatically once yielding occurs. Values of $c_v$ reported in Table 3-3.
correspond to the average between $\sigma'_{\text{v-in situ}}$ and $\sigma'_{\text{yield}}$ (OC states) and the average between $\sigma'_{\text{yield}}$ and $\sigma'_{100\ kPa}$ (NC states).

Table 3-3: Parameters obtained from CRS tests used in settlement and excess pore pressure predictions underneath an embankment

<table>
<thead>
<tr>
<th>Sampler</th>
<th>$\sigma'_{\text{yield}}$ (kPa)</th>
<th>Compression index, $C_c$</th>
<th>Swelling index, $C_s$</th>
<th>$c_v$ before yield (m$^2$/yr)</th>
<th>$c_v$ after yield (m$^2$/yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block</td>
<td>65</td>
<td>2.94</td>
<td>0.23</td>
<td>2.20</td>
<td>0.95</td>
</tr>
<tr>
<td>P100</td>
<td>45</td>
<td>1.83</td>
<td>0.24</td>
<td>2.25</td>
<td>0.85</td>
</tr>
<tr>
<td>O90</td>
<td>62</td>
<td>2.21</td>
<td>0.20</td>
<td>4.5</td>
<td>1.85</td>
</tr>
<tr>
<td>U75</td>
<td>42</td>
<td>1.29</td>
<td>0.17</td>
<td>0.50</td>
<td>0.2</td>
</tr>
</tbody>
</table>

Figure 3-20 compares the predicted surface settlements using the set of compressibility parameters determined from CRS tests for each sampler type. Larger total settlement is predicted when using parameters obtained from block or O89 specimens which had sample quality indexes of ‘excellent’. The settlement curve predicted for the P100 specimen is similar to those obtained for the block and O89. Although this result would be considered in agreement with its sample quality index of ‘good to fair’, it may be noted the important differences in soil properties between this sample and block and O89 specimens shown in Figure 3-7 and Figure 3-8. Total settlement is mainly controlled by $\sigma'_{\text{yield}}$ and $C_c$. The ‘good’ settlement prediction obtained for the P100 specimen is attributed here to the counterbalance effects of a lower compression index (that reduced the predicted total settlement) and a lower yield stress (that increased the predicted total settlement). Therefore, Figure 3-20 shows the underestimation of total settlement when soil parameters are obtained using disturbed specimens such as those obtained with the P100 and U75 samplers. The predicted total settlement after 1000 days ranges around 0.78 - 0.82 m when using block and O89 soil parameters, respectively. The predicted total settlement reduces to 0.35 m and 0.55 m if parameters from P100 or U75 specimens are adopted, respectively. A reduction between 30 and 50 % in total settlement with respect to the predicted value using block parameters is thus observed. Total settlement is mainly controlled by $\sigma'_{\text{yield}}$ and $C_c$. The underestimation of the total settlement is attributed here
to the low values of $C_c$ obtained from P100 and U75 specimens. Although lower values of $\sigma'_{\text{yield}}$ caused by sampling disturbance lead to an overestimation of the total settlement, this effect does not seem to counterbalance (at least for the parameters reported in this paper) the influence of $C_c$. Underestimation of total settlement caused by embankment construction on soft clay has been commonly observed in Australian practice. This result is likely, to a large extent, due to the use of inappropriate sampling techniques, such as the U75 Shelby tube.

Predicted excess pore water pressures are shown in Figure 3-21. Two trends are clearly observed. Similar excess pore pressure dissipation is predicted using parameters from block and O89 specimens. The slower excess pore pressure dissipation predicted using parameters derived from P100 and U75 specimens may be explained by the important reduction in $c_v$ post-yielding which is more dramatic in specimens previously subjected to large sample disturbance (P100 and U75). It can be noted that, after 2000 days, the dissipation of excess pore water pressure was lower than 90%. Therefore, no secondary compression settlement has been included into settlement curves presented in Figure 3-20.

![Figure 3-20: Predicted embankment settlement and dissipation of excess pore pressure](image-url)
Figure 3-21: Predicted pore pressure dissipation curves at depth of 6 m

3.6.2 Settlement and bearing capacity of a strip footing

Figure 3-19(b) shows the geometry of the shallow footing, which is modified from the square footing built at the Ballina field testing facility (Gaone et al., 2017) to a strip footing. The strip footing has a width of 2.0 m and 0.6 m in height. The footing base is located at 1.4 m below the ground surface. Predicted settlement and undrained bearing capacity were calculated using the results from simple shear (SS) tests summarized in Table 3-4. It includes values of undrained shear strength and secant modulus determined at 50 % of the peak shear strength.

Table 3-4: Parameters obtained from SS tests used in undrained bearing capacity and final settlement before failure in undrained condition

<table>
<thead>
<tr>
<th>Sample</th>
<th>Secant Modulus at 50 % strength, G&lt;sub&gt;50&lt;/sub&gt; (kPa)</th>
<th>Undrained shear strength, s&lt;sub&gt;u&lt;/sub&gt; (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Block</td>
<td>1260</td>
<td>16.89</td>
</tr>
<tr>
<td>P100</td>
<td>710</td>
<td>14.64</td>
</tr>
<tr>
<td>O90</td>
<td>990.5</td>
<td>16.64</td>
</tr>
</tbody>
</table>
The footing elastic settlement was calculated using Equation 3-A7 (see Appendix 2) which considered the foundation embedment depth and the location of rigid layers at limited depth. On the other hand, the undrained bearing capacity was computed according to Prandtl’s solution (Equation 3-A8 in Appendix 2) including shape, depth and inclination correction factors.

Figure 3-22 compares the predicted undrained bearing capacity, $q_f$, and the elastic settlement, $s_e$ at $q_f$. Similar $q_f$ values (135 - 140 kPa) were obtained using parameters from block, O89 and U75 specimens. Despite the quite different stress-strain response observed in SS tests (Figure 3-10), the reduction in $q_f$ is less than 11%, a value that seems within the acceptable range of uncertainty in most geotechnical problems. $s_e$ varies from 77 mm to 123 mm, with largest values predicted with the parameters from the P100 specimen. The results presented above indicate that, despite the different stress-strain responses observed in SS tests, prediction of undrained bearing capacity of shallow footing in Ballina clay is not strongly affected by sampling disturbance but certainly affect the determination of the elastic settlement. Differences of more than 60% are observed when comparing settlement due to block and P100 samples, for example.
3.7 CONCLUSIONS

Safe and cost-effective geotechnical design is critically dependent on the representativeness of soil parameters, as these affect the accuracy of the associated predictions. An evaluation of the effects of sampling disturbance caused by different sampling tools on soil parameters estimated from laboratory tests and their consequences in geotechnical design has been presented in this paper. Sample quality assessment has confirmed that, as previously reported for other natural soft soils worldwide, block samples as well as fixed-piston samples provide specimens of excellent quality. On the other hand, misleading sample properties are obtained for specimens retrieved with free-piston samplers and Shelby tubes, when their mechanical properties are compared with block specimens.

The implications of sampling disturbance in geotechnical design were highlighted using predictions made for an embankment as well as a shallow footing problem. Total settlement prediction underneath an embankment is strongly affected by sampling disturbance as a consequence of the important reduction in $\sigma'_{\text{yield}}$ and compression index. Practitioners have to be aware of the fact that there are cases where correct predictions can be achieved, but for the wrong reasons, which by no means reflects good practice. It may occur, for instance, when the reduction in $C_c$ is largely compensated for by the reduction in $\sigma'_{\text{yield}}$.

On the other hand, despite the differences in stress-strain behaviour observed in undrained simple shear tests, the predicted undrained bearing capacity seems less influenced by sampling disturbance, a situation that may lead to misleading interpretation due to the important discrepancies in stress-strain behaviour observed in undrained tests. From a practical perspective, it is important to remark that reliable and cost-effective predictions of geotechnical infrastructure is possible with minimum improvements to the current practice for sampling and testing of soft clays, in particular, by incorporating fixed-piston samplers into current practice.
3.8 APPENDIX 1: CALCULATION OF SETTLEMENT AND EXCESS PORE WATER PRESSURE DUE TO THE EMBANKMENT CONSTRUCTION

The primary consolidation settlement was determined from 1-D consolidation theory using either Equations 3-A1 or 3-A2, depending on the current stress state of the soil (OC or NC). Equation A1 was used in cases where the final effective stress was less than the \( \sigma'_{\text{yield}} \) \((s_{\text{final-OC}})\). If the current stress state reached values larger than the \( \sigma'_{\text{yield}} \), Equation 3-A2 was employed for stress increments beyond this value \((s_{\text{final-NC}})\).

\[
\begin{align*}
s_{\text{final-OC}} &= \frac{C_r}{1+e_0} h_z \log \left( \frac{\sigma'_{\text{initial}} + \Delta \sigma_z}{\sigma'_{\text{initial}}} \right) \quad (3-A1) \\
s_{\text{final-NC}} &= \frac{C_r}{1+e_0} h_z \log \left( \frac{\sigma'_{\text{yield}}}{\sigma'_{\text{initial}}} \right) + \frac{C_c}{1+e_0} h_z \log \left( \frac{\sigma'_{\text{initial}} + \Delta \sigma_z}{\sigma'_{\text{yield}}} \right) \quad (3-A2)
\end{align*}
\]

where \( s_{\text{final}} \) is the settlement of the layer, \( e_0 \) is the initial void ratio, \( C_c \) is the compression index, \( C_r \) is the recompression index, \( h_z \) is the height of the layer, \( \sigma'_{\text{initial}} \) is the initial vertical effective stress and \( \sigma'_{\text{yield}} \) is the yield stress.

FDM was used here to compute the dissipation of excess pore pressure due to consolidation (Equation 3-A3) as it allows the variation in soil parameters (e.g., the consolidation coefficient \( c_v \)) with nodes due to its fundamental algorithm. The degrees of consolidation as well as the settlement \( s_t \), at any time \( t \), were calculated using Equations 3-A3 to 3-A5:

\[
\begin{align*}
u(i, j+1) &= u(i, j) + \beta [u(i-1, j) + u(i+1, j) - 2u(i, j)] \quad \text{where } \beta = \frac{c_v t}{h_z^2} \quad (3-A3) \\
U &= \begin{bmatrix} u_0 & u_{i,0} \\ u_0 & u_{i,0} \end{bmatrix} \quad (3-A4) \\
s_t &= U \times s_{\text{final}} \quad (3-A5)
\end{align*}
\]

where \( s_{\text{final}} \) is the final settlement, \( U \) is the average degree of consolidation, \( u_0 \) is the initial excess pore pressure, \( u_{(i,j)} \) is the excess pore pressure at node \( i \) at time \( j \) and \( c_v \) is the coefficient of consolidation.

The secondary compression settlement was calculated following Mesri and Choi (1985). Secondary compression was assumed to occur after completion of 90% of primary
consolidation (i.e., t > t₀). The secondary compression settlement was computed according to Equation A6

\[ s_{\text{secondary}} = \frac{h_z}{1+e_0} C_a \left( \frac{t}{t_p} \right) \] 3-(A6)

where \( s_{\text{secondary}} \) is the settlement due to creep whereas \( t_p \) is the time at which yielding occurs (days).

### 3.9 Appendix 2: Calculation of Settlement and Bearing Capacity of a Shallow Footing

The average settlement of a flexible footing on saturated clay soil with given Poisson’s ratio of soil, \( \mu_s \) as 0.5 was computed as follows:

\[ s_e = \mu_1 \mu_2 \frac{q_0 B}{E_0} \] 3-(A7)

where \( \mu_1 \) is the correction factor for finite thickness of an elastic soil layer, \( \mu_2 \) is the correction factor for depth of embedment of foundation, \( q_0 \) is the net applied pressure on the foundation, \( B \) is the width of the strip footing, and \( E_0 \) is elastic soil modulus. The details of the correction factors can be found in Christian & Carrier (1978) who modified the original correction factors by Janbu et al. 1956.

The undrained bearing capacity was calculated as follows:

\[ q_f = 5.14 s_c d_c i_c s_u \] 3-(A8)

where \( s_c \) is shape correction factor, \( d_c \) is depth correction factor, \( i_c \) is simplified Meyerhof inclination factor and \( s_u \) is the undrained shear strength. Correction factors were estimated as follows:

\[ s_c = 1 + 0.2 \left( \frac{B}{L} \right) \text{; } B = \text{width and } L = \text{length of footing} \]

\[ d_c = 1 + 0.4 \left( \frac{D}{B} \right) \text{ if } (D/B) \leq 1 \]

\[ d_c = 1 + 0.4 \tan^{-1} \left( \frac{D}{B} \right) \text{ if } (D/B) > 1 \text{ where } D = \text{depth of footing} \]
3.10 REFERENCES


CHAPTER 4. PREDICTED AND MEASURED BEHAVIOUR OF AN EMBANKMENT ON PVD-IMPROVED BALLINA CLAY

Abstract: This chapter presents Class-A and Class-C predictions of the behaviour of an embankment built on soft Ballina clay improved with prefabricated vertical drains. Predictions were carried out using hand calculations and the finite-difference method. The latter approach allowed the variation of soil parameters and stress levels with depth to be considered in the analyses. An alternative systematic procedure for estimating soil parameters based on high-quality laboratory data is described. Class-A predictions highlighted some disagreements with the measured total settlements and pore pressure dissipation rates. For Class-C predictions, the choice of geotechnical parameters used in the analyses was guided by a systematic assessment of the stress states undergone by soil elements underneath the embankment centreline. This led to a better agreement between predicted and measured data, which demonstrates the potential of the proposed procedure for future analyses of embankment behaviour on soft Ballina clay.
4.1 INTRODUCTION

The prediction of settlements for embankments built on soft ground is a classical problem in soil mechanics. Yet, this remains a challenging task for geotechnical engineers despite significant advances in laboratory and in situ testing, as well as numerical and constitutive modelling. Natural soft soil deposits typically display low undrained shear strength and stiffness, high compressibility, low permeability and weak structure as a result of complex physico-chemical interactions that take place during soil deposition. Poor understanding of the behaviour of natural soft soil deposits (mainly due to poor site characterization) may lead to erroneous selection of soil parameters. A key aspect for the selection of representative soil parameters is to consider the particular stress path imposed by the embankment (Zdravković & Jardine 2001). The selected method of analysis, and constitutive model, also play significant roles in the ability to predict settlements, lateral displacements and excess pore water pressures (Poulos 1999). The use of ground improvement techniques, such as prefabricated vertical drains (PVDs), requires additional factors to be accounted for in such analyses.

The Embankment Prediction Symposium (EPS) organized by the ARC Centre of Excellence for Geotechnical Science and Engineering (CGSE) has offered a great opportunity to assess current practice in predicting settlements caused by the construction of an embankment improved with PVDs at the National Soft Soil Testing Facility in Ballina, Australia (Kelly et al. 2016). Class-A predictions were presented and then compared to the field data during the symposium. Class-C predictions (back analyses) were also requested by CGSE to match the measured behaviour. Class-C predictions aim at improving the accuracy and reliability of the methods used to predict the behaviour of embankments constructed on soft estuarine clays, which are commonly found along the east coast of Australia.

This paper describes Class-A and Class-C predictions of embankment behaviour. Predictions were carried out using one-dimensional calculations along with a semi-empirical approach and the finite-difference method (FDM). One-dimensional methods were used due to their simplicity and common use in current practice. Emphasis was given in this paper on the method used to interpret in situ and laboratory data for the selection of geotechnical parameters. The selected method was based on considerations of soil stress conditions under the embankment. Predictions of settlements and excess pore water
pressures were then compared with field data. Finally, a sensitivity analysis was carried out to assess the uncertainty associated with the selection of soil parameters on the predictions made. Details of the methods and analyses used are described in the next sections.

4.2 SUBSOIL CONDITIONS AND EMBANKMENT CHARACTERISTICS

Figure 4-1 summarises the main index and mechanical properties of the soil profile at the Ballina site obtained from in situ and laboratory tests (Kelly et al. 2016; Pineda et al. 2016). The laboratory tests were performed from O89 specimens. The soil profile may be divided into five main layers as follows: (a) a shallow upper crust mainly composed of silty sands ($z < 2$ m), (b) a transition zone composed of silty clays with high shell content ($2 < z < 4$ m), (c) homogenous soft Ballina clay layer ($4 < z < 11$ m), (d) a transition sandy layer ($11 < z < 14$ m), and (e) a Pleistocene stiff clay layer ($z < 14$ m).

Ballina clay displays high plasticity with liquid limit values slightly higher than its natural water content (Figure 4-1a). Dry density reduces with depth (void ratio increases) and remains almost constant below a depth of 4 m. This is consistent with the increase in the clay fraction which is, mainly composed of illite, kaolinite and interstratified illite/smectite. In the soft Ballina clay layer, the soil is slightly overconsolidated ($1 < \text{yield stress ratio (YSR)} < 2$, Figure 4-1b). The coefficient of consolidation of the clay layer ($c_v$) at yield stress ($\sigma'_{\text{yield}}$) varies between approximately 1 and 4 $\text{m}^2$/year (CRS and IL-Creep tests). The horizontal coefficient of consolidation ($c_h$) estimated from CPTu and piezoball tests (Kelly et al. 2016) shows a similar range of variation, which suggests small permeability anisotropy in Ballina clay. The undrained shear strength ($s_u$) increases with depth. The $s_u$ is greatest in triaxial compression, intermediate in vane shear and least in triaxial extension. The sensitivity estimated from field vane tests varies between 1.5 and 5.5 (Kelly et al. 2016).

The embankment is nominally 80 m long by 15 m wide at the crest, with a total height of 3 m and nominal inclination of 1.5H: 1V. The working platform is approximately 95 m long by 25 m wide, with a height of 0.6 m. The embankment is divided into 3 sections, two 30 m long with PVDs and one 20 m long with Jute PVDs. Vertical drains were installed in the soil underneath the working platform after its completion. The spacing between drains is 1.2 m with a square grid. Vertical drains are
100 mm wide and 3 mm thick. The mandrel has a width of 120 mm by 60 mm in a rectangular shape. Instrumentation of the embankment includes inclinometers (INCLO), settlement plates (SP), magnetic extensometers (MEX), vibrating wire piezometers (VWP) and hydraulic profile gauges (HPG). Figure 4-2 shows the embankment geometry and the location of the instrumentation. SP3 is located at section 1, MEX1, VWP5, INCLO 1 and 2 are located at section 2, whereas SP2 is located at section 3. SP2 and SP3 are located at the top surface and measure the total settlement of the embankment. MEX1 measures the localised vertical settlements at depths of 2, 5, and 8 m, VWP5 monitors the pore pressure dissipation at depths of 2, 6 and 9.5 m and INCLO 1 and 2 measure the lateral displacement at the toe on each side of the embankment. The unit weight of the fill used to construct the embankment, estimated via nuclear density tests, is 20.9 kN/m³. The total vertical surcharge applied by the embankment at the ground surface is 63 kPa. As explained in Appendix A, elastic solutions were used to determine the stress increments with depth. From the surveying data, the topography of the embankment during construction and the position of the instruments were obtained using AutoCad Civil3D® software. Figure 4-3 shows the schematic vertical cross section of the embankment. Table 4-1 summarises the geometry of the vertical cross sections used in the predictions for the two stages of embankment construction. The embankment dimensions vary slightly at each location, especially the embankment slope on both sides.

As a minimum, symposium participants were asked to provide at least, the following estimations:

- Time-settlement curves for SP2 and SP3;
- Time-settlement curves at three distinct depths for MEX1;
- Variation of total pore pressures over time for VWP5; and
- Lateral deformation for INCLO 1 and 2 at the end of construction and after three years from the start of construction.
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![Graph showing predicted and measured behaviour of an embankment on PVD-improved Ballina clay](image-url)
Figure 4-1: (a) Main index and (b) mechanical properties of the soil profile at the Ballina site.
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Figure 4-2: Nominal dimensions of the embankment.

Figure 4-3: Schematic embankment cross section used for vertical stress increment calculations.

Table 4-1: Dimensions of the sections based on the location of instruments.

<table>
<thead>
<tr>
<th>Section</th>
<th>Instruments</th>
<th>Embankment Dimensions (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Stage 1</td>
</tr>
<tr>
<td></td>
<td></td>
<td>a  b  c  d  h_e</td>
</tr>
<tr>
<td>1</td>
<td>SP3</td>
<td>34.1  2.5  4.0  40.6  0.9</td>
</tr>
<tr>
<td>2</td>
<td>MEX1</td>
<td>34.3  2.1  3.6  40.1  0.9</td>
</tr>
<tr>
<td>2</td>
<td>INCLO1/2</td>
<td>34.3  2.1  3.6  40.1  0.9</td>
</tr>
<tr>
<td>3</td>
<td>SP2</td>
<td>32.6  2.9  3.7  39.1  0.9</td>
</tr>
</tbody>
</table>

SP = settlement plate; MEX = magnetic extensometer; VWP5: vibrating wire piezometer; INCLO: inclinometer

Note: Stage 1 corresponds to completing the working platform including the sand blanket. Stage 2 refers to completing the embankment with PVD after completion of Stage 1.
4.3 PREDICTION METHODS USED IN THE ANALYSIS

The upper 10.5 m of soil was considered in the predictions. This includes the shallow sandy layer, the transition silty clay and the soft Ballina clay above 10.5 m. The sandy layer and the stiff Pleistocene clay undergo negligible deformations. For each cross section of the embankment, the 10.5 m deep soil profile was divided into 21 sublayers. The geotechnical parameters for each 0.5 m thick sublayer were estimated from laboratory test results (CGSE 2016; Pineda et al. 2016). Two methods of analysis were used to predict vertical settlements, excess pore water pressures and lateral displacements in the presence of PVDs. The first approach combines 1D consolidation theory (Terzaghi & Frolich 1936) with the semi-empirical method proposed by Barron and Hansbo (Barron 1948; Hansbo 1976) (BH method). In the second approach, 1D consolidation theory is used in conjunction with FDM. Although the adopted methods do not explicitly account for particular features of soft clay behaviour such as rate effects, soil anisotropy and soil destructuration, they may provide reliable estimates of the global embankment behaviour. Their simplicity is advantageous as they may be easily implemented in practice without requiring an excessive number of parameters. They can also be used to validate complex numerical simulations. The methods were implemented in a spreadsheet using Microsoft Excel®. In both Class-A and Class-C predictions, the following estimations were made:

1. End of primary consolidation settlement due to the weight of the working platform and the embankment using hand calculations;
2. Time variation of excess pore water pressure, degree of consolidation and consolidation settlement due to the construction of the working platform (without PVD) using the FDM;
3. Time variation of excess pore water pressure, degree of consolidation and consolidation settlement due to the construction of the embankment after installation of PVDs using the BH method;
4. Same as in step 3 but using the FDM;
5. Secondary compression settlement from 1D consolidation theory; and
6. Lateral displacements using empirical correlations from Tavenas et al. (Tavenas et al. 1979).

A brief description of each calculation step used in the analysis is given below.
4.3.1 End of primary consolidation settlement due to working platform and embankment

The end of primary consolidation settlement was determined from 1-D consolidation theory. Ballina clay is slightly over-consolidated with a yield stress ratio typically lower than 2. In cases where the final vertical effective stress was less than the \( \sigma'_{\text{yield}} \), Equation 4-1 was used. For these cases the final state of the soil is overconsolidated (\( s_{\text{final-OC}} \)). For sublayers loaded to a final vertical effective stress larger than the \( \sigma'_{\text{yield}} \), Equation 4-2 was employed instead. In this case, the final state of the soil is normally consolidated (\( s_{\text{final-NC}} \)). The total final settlement at a given depth was estimated as the sum of the settlements in the underlying sublayers.

\[
s_{\text{final-OC}} = \frac{C_r}{1+e_0} h_z \log \left[ \frac{\sigma'_{v0} + \Delta \sigma_v}{\sigma'_{v0}} \right]
\]

\[
s_{\text{final-NC}} = \frac{C_r}{1+e_0} h_z \log \left[ \frac{\sigma'_{v0} + \Delta \sigma_v}{\sigma'_{\text{yield}}} \right] + \frac{C_c}{1+e_0} h_z \log \left[ \frac{\sigma'_{v0} + \Delta \sigma_v}{\sigma'_{\text{yield}}} \right]
\]

\( s_{\text{final}} \) = settlement of sub-layer (either OC or NC) (m)
\( e_0 \) = initial void ratio
\( C_c \) = compression index
\( C_r \) = recompression index
\( h_z \) = height of sublayer (m)
\( \sigma'_{v0} \) = initial vertical effective stress (kPa)
\( \sigma'_{\text{yield}} \) = yield stress (kPa)
\( \Delta \sigma_v \) = stress increment due to embankment weight (kPa)

4.3.2 Consolidation settlement due to the construction of the working platform only (without PVDs) using FDM

Classical 1-D consolidation theory does not account for variations in soil parameters due to the non-homogeneity of the layer (among other factors), especially \( c_v \). The \( c_v \) varies with depth and stress level. Assuming a single value for the entire soil layer may lead to non-reliable predictions of the settlement. Therefore, the FDM was used here...
to account for variations in soil parameters across the sublayers (Duncan 1993; Farnsworth et al. 2013) and estimate the dissipation of excess pore pressure generated by the embankment construction (Equation 4-3). Due to the division of the soil profile into sublayers with different $c_v$ values, Equation 4-3 was modified to account for the difference in soil properties at the interface between two separate layers (see Equation 4-4(Das 2013)). Equation 4-4 was employed to obtain the excess pore pressure due to the construction of the working platform (without PVDs) where the drainage path is preferentially vertical. The initial excess pore pressure ($u_0$) was assumed to be equal to the stress increment in the layer imposed by the embankment (see Appendix A). The boundary conditions used in Equation 4-4 were full drainage ($u = 0$) at $z = 0$ and $z = 10.5$ m.

From the excess pore pressure, the degree of consolidation in the vertical direction was calculated using Equation 4-5. The settlement of the embankment, $s_t$, at any time, $t$, after completing the working platform was calculated via Equation 4-6 (Balasubramaniam et al. 2010; Almeida & Marques 2013).

\[
u(i,j+1) = \nu(i,j) + \beta [\nu(i-1,j) + \nu(i+1,j) - 2\nu(i,j)] \quad \text{where } \beta = \frac{c_v \Delta t}{h_z^2} \quad (4-3)
\]

\[
u(i,j+1) = \frac{c_{v,1} \Delta t}{h_z^2} \left[ \frac{1 + \left( \frac{h_z}{k_1} \right)}{1 + \left( \frac{h_z}{k_1 c_{v,1}} \right)} \right] \times \left[ \frac{2k_1}{k_1 + k_2} \nu(i-1,j) + \frac{2k_2}{k_1 + k_2} \nu(i+1,j) - 2 \nu(i,j) \right] + \nu(i,j) \quad (4-4)
\]

\[
U = \left[ \frac{u_0 - \nu(i,j)}{u_0} \right] \quad (4-5)
\]

\[
s_t = U \times s_{\text{final}} \quad (4-6)
\]

$s_{\text{final}}$ = final settlement (Eqs. 1 and 2) (m)
$s_t$ = settlement at any time, $t$ (m)
$U$ = average degree of consolidation in the particular sublayer
$u_0$ = initial excess pore pressure (kPa)
$\nu(i,j)$ = excess pore pressure at node $i$ at time $j$ (kPa)
$h_z$ = height of sublayer (m)
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\[ c_v = \text{coefficient of consolidation (m}^2\text{/year)} \]

\[ c_{v,1} = \text{coefficient of consolidation of top layer at the interface between two layers (m}^2\text{/year)} \]

\[ c_{v,2} = \text{coefficient of consolidation of bottom layer at the interface between two layers (m}^2\text{/year)} \]

\[ k_1 = \text{hydraulic conductivity of top layer at the interface between two layers (m/s)} \]

\[ k_2 = \text{hydraulic conductivity of bottom layer at interface between two layers (m/s)} \]

### 4.3.3 Consolidation settlement due to the working platform and embankment (with PVDs) using the BH method

In the presence of PVD, water may flow along vertical and horizontal directions. However, for a relatively thick clay layer (> 5 m), it is reasonable to assume that drainage occurs mainly in the horizontal direction (Almeida & Marques 2013; Farnsworth et al. 2013). Therefore, the average degree of consolidation is assumed to be similar to the average degree of consolidation in the horizontal direction \( (U_r) \) (Equations 4-7 to 4-17).

The determination of \( U_r \) accounts for the influence of smear (Equation 4-11) and the influence of the diameter and equivalent diameter of the PVDs (Equations 4-12 to 4-15). Variations in the efficiency of the PVDs with time (e.g. due to clogging) are not considered in this paper. The settlement, \( s_t \), at any time \( t \), is calculated from Equation 4-6. This set of equations is based on the works of Barron (Barron 1948) and Hansbo (Hansbo 1976). The approach has been successfully used in the design of embankments on very soft soils (Almeida & Marques 2013).

\[ U = U_r \quad (4-7) \]

\[ U_r = 1 - e^{-\frac{ST_r}{F}} \quad (4-8) \]

\[ T_r = \frac{c_h t}{d_e^2} \quad (4-9) \]

\[ F = F(n) + F(s) \quad (4-10) \]
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\[ F(s) = \left( \frac{k_h}{k_v} \right) \ln \left( \frac{d_e}{d_w} \right) \]  \hspace{1cm} (4-11)

\[ F(n) = \frac{n^2}{n^2 - 1} \ln(n) - \frac{3n^2 - 1}{4n^2} \approx \ln(n) - 0.75 \]  \hspace{1cm} (4-12)

\[ n = \frac{d_e}{d_w} \]  \hspace{1cm} (4-13)

\[ d_e = 1.05 \times \text{spacing of wick drains} \]  \hspace{1cm} (4-14)

\[ d_w = \text{perimeter of wick drains} / 2\pi \]  \hspace{1cm} (4-15)

\[ d_s = \kappa d_m \]  \hspace{1cm} (4-16)

\[ d_m = \sqrt{4w_m l_m / \pi} \]  \hspace{1cm} (4-17)

\[ U_r = \text{average degree of consolidation in horizontal direction} \]

\[ T_r = \text{time factor for horizontal drainage} \]

\[ c_h = \text{coefficient of consolidation in the horizontal direction} (\text{m}^2/\text{year}) \]

\[ k_h = \text{hydraulic conductivity in the horizontal direction} (\text{m/s}) \]

\[ k_v = \text{hydraulic conductivity in the vertical direction} (\text{m/s}) \]

\[ d_e = \text{diameter of influence with square pattern} (\text{m}) \]

\[ d_w = \text{equivalent diameter of the PVD} (\text{m}) \]

\[ d_s = \text{diameter of disturbed zone} (\text{m}) \]

\[ d_m = \text{equivalent diameter of the mandrel} (\text{m}) \]

\[ w_m = \text{width of the mandrel} (\text{m}) \]

\[ l_m = \text{length of the mandrel} (\text{m}) \]

\[ \kappa = \text{constant for disturbance (smear) zone} \]

\[ F(s) = \text{influence of smear effects} \]

\[ F(n) = \text{influence of diameter and equivalent diameter of the PVD} \]
4.3.4 Consolidation settlement due to the working platform and embankment (with PVDs) using FDM

As an alternative to the BH method, FDM was used to estimate the dissipation of excess pore pressure due to the embankment construction, in the presence of PVDs. Only radial drainage was considered, an assumption also adopted in previous studies (Almeida & Marques 2013; Farnsworth et al. 2013). In this method, four nodes were allocated from the vertical axis of a reference PVD to the centre line between two adjacent PVDs (Figure 4-4). The boundary condition at the vertical axis of the PVD corresponded to zero excess pore pressure. At the centre line between two PVDs, the excess pore pressure was maximum and dissipated over time. The pore pressure was assumed to vary linearly with radial distance. Equation 4-18 was used to estimate the radial dissipation of excess pore pressure at the centreline between two PVDs. The degree of consolidation and settlement were then obtained from Equations 4-5 and 4-6, respectively. Unlike the BH method, no correction for smear effects was made in the FDM. Therefore, a slight overestimation of the settlement may be expected.

\[ u(i, j+1) = u(i, j) + \beta_r \left[ u(i-1, j) + u(i+1, j) + \frac{u(i+1, j) - u(i-1, j)}{2 \left( \frac{r_i}{\Delta r} \right)^2} - 2u(i, j) \right] \]

where \( \beta_r = \frac{c_h t}{\Delta r^2} \)

**Figure 4-4:** Variation of excess pore pressure across different nodes for radial drainage.
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\[ u(i,j) = \text{excess pore pressure at node } i \text{ at time } j \text{ (kPa)} \]

\[ c_h = \text{coefficient of consolidation in the horizontal direction (m}^2/\text{year}) \]

\[ r_i = \text{radial distance from the centreline between two adjacent PVDs to the node of interest (m)} \]

\[ \Delta r = \text{distance between two nodes (m)} \]

### 4.3.5 Secondary compression settlement

The secondary compression settlement was calculated according to the method presented by Mesri and Choi (Mesri & Choi 1985). For Class-A predictions, secondary compression was assumed to occur after 95% of the primary consolidation was achieved in each sublayer. It was also assumed that the secondary compression coefficient \( C_\alpha \) remained constant under constant stress and was not affected by drainage conditions. In reality, secondary compression settlement occurs concurrently with primary consolidation due to the viscosity of the soil structure (Almeida & Marques 2013). This aspect is discussed in the back-analysis carried out in the Class-C predictions. The secondary compression settlement was computed according to Equation 4-19:

\[ s_{sec} = \frac{h_z}{(1+e_0)} C_\alpha \left( \frac{t}{t_p} \right) \]  

\( h_z = \text{sub-layer height (m)} \)

\( s_{sec} = \text{secondary compression settlement (m)} \)

\( t_p = \text{time corresponding to 95% degree of consolidation in the sublayer (day)} \)

\( t = \text{time (day)} \)

\( C_\alpha = \text{secondary compression index from incremental loading (Creep) test} \)

### 4.3.6 Lateral displacement

The lateral displacement at the toe of the embankment was predicted using the empirical expression proposed by Tavenas et al. (1979), which was derived from field observations of 21 different embankments. This empirical expression relates the maximum lateral displacement to the settlement via a parameter, \( \alpha \) (Equation 4-20). The parameter \( \alpha \) takes different values depending on the soil state (OC or NC) (Tavenas et al.
1979). The $\alpha$ values adopted in the Class-A and Class-C predictions are discussed in Section 4.4 and 4.5, respectively.

$$y_m = \alpha s$$

\(y_m\) = maximum lateral displacement (m)

\(\alpha\) = ratio of maximum lateral displacement to vertical displacement

\(s\) = total settlement (m)

### 4.3.7 Selection of soil parameters

The profiles of mechanical properties shown in Figure 4-1b include estimates for both initial and yield stress conditions in situ. This information is typically insufficient to properly reproduce the behaviour of geotechnical structures under different loading scenarios. The selection of soil parameters requires proper understanding of the stress paths imposed by the embankment at different locations in the soil profile. The 1D methods used in this paper provide approximate tools to predict the behaviour of the soil underneath the embankment. Strictly speaking, they apply solely to soil states under the embankment centre line, where no lateral deformation occurs.

In general, the settlement caused by the construction of embankments on soft soils is controlled by: (i) the overconsolidation ratio (OCR or YSR), (ii) the coefficients of consolidation ($c_v$ and $c_h$), (iii) the compressibility index ($C_c$), (iv) creep effects (e.g., $C_\alpha$) and embankment geometry. While this may often be overlooked, the rigorous selection of soil parameters requires a deep understanding of soil behaviour and proper knowledge of in situ and laboratory testing techniques. With naturally structured soft soils, like Ballina clay, the highly non-linear response observed during 1D compression (due to soil destructuration) poses additional challenges to the selection of a (unique) set of soil parameters to be used in design.

To illustrate this aspect, Figure 4-5 shows the results of two 1D compression tests carried out on Ballina clay specimens obtained from 5.22 m (IL test) and 5.49 m (CRS test) depths. Although both CRS and IL tests may be used to study soil compressibility, the CRS test has the advantage of providing a continuous compressibility curve ($e$-log $\sigma'_v$). This simplifies the estimation of the $\sigma'_{\text{yield}}$ but also provides continuous variation of the $C_c$ with stress level. Creep effects can be directly assessed from IL tests which also
provide good estimates of other keys including $c_v$ and hydraulic conductivity ($k_w$). In Figure 4-5, the vertical stress increment ($\Delta \sigma_v$) imposed by the embankment at a depth of 5.5 m is indicated by the shaded area. As observed in Figure 5a the stress state moves from an overconsolidated (OC) to a normally consolidated (NC) state as a consequence of the stress increment applied by the embankment. Based on $\sigma'_{\text{yield}}$ values reported in [3], similar situations occur throughout the soil profile. The strong non-linearity of the compressibility curve highlights the challenges associated with any attempt of estimating a unique value for the $C_c$. Alternatively, one may represent the NC part of the compressibility curve by defining two compressibility indices: (i) $C_{c-1}$ for stresses in the range $\sigma'_{\text{yield}} < \sigma' < \sigma'_{\text{v}} = 100\text{kPa}$ and, (ii) $C_{c-2}$ for effective vertical stresses larger than 100 kPa. Inspection of Figure 4-5b indicates that, although $C_{c-2}$ may represent the $C_c$ at large stresses with high confidence, the adoption of an intermediate value $C_{c-1}$ clearly does not capture the strong soil non-linearity. The $c_v$ as well as the $C_\alpha$ are strongly affected by the stress increment applied by the embankment (see Figure 4-5c and 4-5d). $c_v$ reduces dramatically after yielding, whereas $C_\alpha$ reaches a maximum value around $\sigma'_{\text{yield}}$ and then decreases with increasing stress level.

It is important to note that it will take some time for each soil layer to reach equilibrium in effective stresses, due to the consolidation experienced by the clay. This aspect introduces some degree of judgement in the definition of the stress level used for the selection of soil parameters. In this paper, a simple methodology was used to estimate the behaviour of the soil under the embankment. The criterion used here is based on the definition of a reference vertical effective stress at which soil parameters are assessed thus requiring less judgement, which may be highly subjective. As discussed next, the definition of the reference vertical effective stress is refined in the Class-C predictions (back-analysis) to consider the strong influence of soil destructuration on the soil response, particularly regarding $c_v$. 
Figure 4-5: Compressibility and consolidation properties of Ballina clay: (a) Compressibility curve; (b) Variation of \( C_c \) with stress level; (c) Variation of \( c_v \) with stress level and testing method; (d) Variation of \( C_a \) with stress level.

4.4 CLASS-A PREDICTIONS

4.4.1 Determination of soil parameters for the Class-A predictions

Figure 4-6 shows a typical compressibility curve (e-log \( \sigma' \)) obtained from CRS testing on Ballina clay. Initial in situ (\( \sigma'_{i} \)), yield and final (i.e., post construction) vertical effective stresses are identified in this figure using arrows. For the Class-A predictions, the reference vertical effective stress used for the estimation of soil parameters is defined as: \( \sigma'_{r,ref-A} = (\sigma'_{i} + \sigma'_{f,final})/2 \) (Figure 4-6). The values of \( \sigma'_{r,ref-A} \) for each sublayer are listed in Table 4-2.
Figure 4-6: Example of the estimation of $\sigma'_{\text{ref},A}$ for the selection of soil parameters (data from INCLO2-CRS-5.49 m [3]).

Table 4-2 summarises the soil parameters estimated from the laboratory characterization study reported in (CGSE 2016; Pineda et al. 2016). Table 4-2 includes:

- $e_0$ and $\rho_{\text{bulk}}$: estimated from Tables 1-4 (Pineda et al. 2016).
- $OCR$: These values were computed as the ratio of $\sigma'_{\text{yield}}$ to the $\sigma'_{\psi}$ reported in Table 2 (Pineda et al. 2016). Yield stresses were previously corrected by rate effects (Pineda et al. 2016) using a correction factor of 0.84.
- $C_r$: for simplicity, the $C_r$ was assumed equal to the swelling index ($C_r = C_s$). Values of $C_s$ were estimated for each sublayer from IL test results summarised in Table 3 (Pineda et al. 2016).
- $C_c$: values of $C_c$ were estimated from CRS tests. The variation of $C_c$ with the effective vertical stress is highly nonlinear due to progressive soil destructuration (see Figure 4-9 in (Pineda et al. 2016)). Therefore, $C_c$ was estimated from Figure 9 (Pineda et al. 2016) for the particular $\sigma'_{\text{ref},A}$ of each sublayer.
- $c_v$: although estimations from CRS and IL tests seem to follow the same trend, only IL (Creep) test results had been used in the estimation of $c_v$. CRS tests were not considered here due to the scatter observed, mainly between 2 m and 4.5 m depths, which is attributed to the small excess pore water pressure measured during the tests for stresses lower than $\sigma'_{\text{yield}}$. Therefore, the IL results reported in
Figure 13 and Table 3 (Pineda et al. 2016) were fitted using an exponential function from which values of $c_v$ were estimated at $\sigma'_{v,\text{ref-A}}$ for each sublayer.

- $C_0$: These values were determined from Figure 12 (Pineda et al. 2016) for the particular $\sigma'_{v,\text{ref-A}}$ of each sublayer.
- $\alpha$ values of 0.18 (OC state) and 0.91 (NC state) were adopted in these predictions, in agreement with the data set reported by Tavenas et al. (Tavenas et al. 1979).
### Table 4-2: Soil parameters used in Class-A and Class-C predictions

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>Bulk unit weight of soil, γ (kN/m³)</th>
<th>OCR/YSR</th>
<th>G&lt;sub&gt;υ&lt;/sub&gt;-sec</th>
<th>Class-A predictions</th>
<th>Class-C predictions</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td>C&lt;sub&gt;e&lt;/sub&gt;, C&lt;sub&gt;s&lt;/sub&gt;, C&lt;sub&gt;α&lt;/sub&gt;</td>
<td>Initial void ratio, e&lt;sub&gt;υ&lt;/sub&gt;</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>(m&lt;sup&gt;3&lt;/sup&gt;/yr)</td>
</tr>
<tr>
<td>0.5</td>
<td>21.1</td>
<td>5.48</td>
<td>2950</td>
<td>1.09, 0.10, 0.009, 0.002</td>
<td>147.00, 42.1</td>
</tr>
<tr>
<td>1.0</td>
<td>21.1</td>
<td>3.92</td>
<td>2950</td>
<td>1.37, 0.75, 0.009, 0.002</td>
<td>147.00, 47.8</td>
</tr>
<tr>
<td>1.5</td>
<td>21.1</td>
<td>3.40</td>
<td>2950</td>
<td>1.64, 0.83, 0.096, 0.002</td>
<td>7.12, 50.2</td>
</tr>
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4.4.2 Comparison of Class-A predictions with measured data

The embankment was completed in two stages. First the working platform with sand blanket was built in one day. Then, after installation of the PVDs, the embankment was also constructed in one day, 20 days after the completion of stage 1. In the figures below, the measured data is labelled as MD. The predicted behaviour for the Class-A predictions are labelled as A-BH (Barron and Hansbo method) and A-FDM (Finite-difference method), depending on the method used.

Figure 4-7 compares the measured and predicted settlements at the location of settlement plates SP2 and SP3. Some discrepancies are observed between measured and predicted settlements. A faster settlement rate was predicted by the FDM than for the BH method. This faster settlement rate is due to the fact that smear effects due to PVD installation were not considered in the FDM. The FDM overestimated the initial part of the settlement curve ($t < 400$ days) but underestimated the total settlement after 1090 days by 10%. Two different responses were predicted by the BH method at the SP2 and SP3 locations, reflecting the embankment asymmetry at the end of construction. However, the measured settlements at SP2 and SP3 show almost identical responses. Closer inspection of Figure 4-7 shows high settlement rate was actually measured between 50 and 100 days, indicating that a 30-day delay took place between the end of the embankment construction (at 20 days) and the settlement mobilization. This delay is not captured in the Class-A predictions. For $t > 800$ days, the predicted settlement rates were lower than the measured one in all cases.

Comparison between measured and predicted pore water pressures at depths of 2, 6 and 9.5 m is presented in Figure 4-8. In the Class-A predictions, it was assumed that negligible excess pore pressure was generated during the construction of the first 1 m of fill (sand layer and working platform) due to the slow rate of construction. While this assumption seems to be valid only for shallow (2 m) and deeper (9.5 m) layers due to their proximity to the drainage boundaries, the same assumption was adopted for the entire soil profile. Additionally, the influence of the settlement of VWP5 on the pore pressure response was not considered in the Class-A predictions. The predicted dissipation rates at depths of 2 and 6 m was faster than the field measurements. At a depth of 9.5 m, the predicted dissipation rate matched reasonably well with the measured data. The FDM method predicted faster dissipation of excess pore water pressure than the BH method, due to the fact that smear effects were not considered in the FDM method. The
measured response observed at a depth of 6 m indicates much lower dissipation rates in the field and highlights the differences between the *in situ* and the adopted coefficients of consolidation.

Figure 4-9 shows the relative settlement curves, measured and predicted, at depths of 2, 5 and 8 m. FDM predictions overestimate the settlement magnitude and rate as previously observed in Figure 4-7. The BH method shows closer agreement with the field data including the settlement rate at a deeper depth of 8 m.

The predicted and measured lateral displacements at the locations of inclinometers INCLO 1 and INCLO 2 are shown in Figure 4-10. The empirical method outlined by Tavenas et al. (Tavenas et al. 1979) overestimates both short and long-term responses, with the maximum lateral displacements being overpredicted by more than 100%. This behaviour suggests that the values of the parameter $\alpha$ recommended by Tavenas et al. (Tavenas et al. 1979) are not appropriate to predict the response of the embankment at the Ballina site. Reasons for such a discrepancy may be attributed to the variations in the soil profile, embankment geometry as well as the presence of the PVDs. This aspect is analysed in more detail in the following sections.

There are significant differences in the predictions obtained using the BH and FDM approaches, which may be explained, in part, by the fact that the FDM does not account for smear effects associated with PDV installation. Comparison between predicted and measured behaviour indicates that the adopted coefficients of consolidation were inadequate to properly predict the dissipation of excess pore pressures.
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Figure 4-7: Comparison between predicted (Class-A) and measured surface settlements with time.

Figure 4-8: Comparison between predicted (Class-A) and measured total pore pressures over time.
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Figure 4-9: Comparison between predicted (Class-A) and measured relative settlements.

Figure 4-10: Comparison between predicted (Class-A) and measured lateral displacements at two different times, T1 after completion of embankment and T2 after three years of construction.

4.5 CLASS-C PREDICTIONS (BACK-ANALYSES)

The comparison between Class-A predictions and measured data highlighted the uncertainties associated with the estimation of settlements and excess pore water pressures in Ballina clay. This is in consistent with previous studies that have highlighted the difficulties associated with predicting embankment behaviour accurately (Davis & Poulos 1975; Balasubramaniam et al. 2007; Chai et al. 2013). A significant (and
surprising) aspect to remark about the measured data is that the excess pore water pressures have not totally dissipated after 1000 days. Obvious differences during the settlement and pore pressure dissipation rates were observed between the predicted and measured data for the last 700 days of the monitoring period. This behaviour suggests that $c_v$ and $k_w$ have been strongly reduced due to the stress increment applied by the embankment. Therefore, values of $c_v$ (and $c_h$) at *in situ* or even $\sigma'_\text{yield}$ levels may not be appropriate to capture the real phenomena happening after completion of the embankment.

The Class-C predictions (back-analyses) described in this paper were aimed at assessing the predicting methods described above to identify key aspects of the soil behaviour that should be carefully considered to generate reliable predictions. An attempt is made here to develop a practical methodology to select soil parameters aimed at reducing subjective inputs. The method of analysis and the adopted soil parameters both contribute to the level of accuracy and reliability associated with settlement and pore-water pressure predictions produced by a given analysis. These two aspects are discussed in detail in the following sections. In the Class-C predictions, the completion of the embankment was changed from day 20\(^{th}\) to day 50\(^{th}\), which corresponds to the actual completion day reported by the CGSE. Additionally, the excess pore pressures generated during the construction of the sand blanket and the working platform were included in this analysis.

### 4.5.1 Revision of soil parameters for Class-C predictions

In the Class-A predictions, $\sigma'_{v,\text{ref-A}}$ was defined as the average between the initial *in situ* and final effective stresses was used for the selection of soil parameters. The use of the initial *in situ* vertical effective stress in the estimation of $\sigma'_{v,\text{ref-A}}$ tends to provide values close to the $\sigma'_\text{yield}$, at which soil destructuration starts. Therefore, $C_c$ tends to be underestimated, whereas $c_v$ tends to be overestimated (see Figure 4-5). A new definition of the reference vertical stress was adopted in the Class-C predictions, based on the following observation. Measured pore water pressures show that around 50 % of the maximum excess pore water pressure dissipated within a few months after embankment completion. Correspondingly, during this timeframe, the effective stresses increased from initial *in situ* to yielding conditions. After that, the stress state of the soil slowly approaching its final state. Therefore, the reference vertical effective stress used in the
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Class-C predictions is redefined as the average between the yield and final effective vertical stresses: \( \sigma'_{v,\text{ref-C}} = (\sigma'_{v,yield} + \sigma'_{v,final})/2 \) (see Figure 4-6).

Parameters such as OCR, \( c_v \), \( C_c \) and \( C_\alpha \) play a key role on settlement and pore water pressure predictions. OCR and \( C_c \) control the maximum settlement, whereas \( c_v \) and \( C_\alpha \) control the dissipation of excess pore water pressure and the settlement rate. After a critical review of the soil parameters used in the Class-A predictions, some modifications were made for the Class-C predictions, as described below and summarised in Table 4-2.

- **OCR**: no modifications were made to the OCR values adopted in the Class-A predictions.
- **\( c_v \)**: the higher pore pressure dissipation rates predicted in the Class-A analysis compared to the measured data suggests that the adopted values of \( c_v \) were larger than the *in situ* ones. A key feature of Ballina clay is the strong reduction in \( c_v \) (and \( k_w \)) after yielding. This reduction is due to soil destructuration and the significant decrease in \( e_0 \) induced by small increments in vertical effective stress. \( c_v \) reduces by about one order of magnitude as the vertical effective stress increases from the \( \sigma'_{yield} \) to the final stress (Figure 4-5a and 4-5c). This suggests that adopting an approach that distinguishes between OC and NC soil states may be useful. Therefore, a two-part \( c_v \) profile was selected that combines a section for OC states and another one for NC states (Figure 4-11). This figure shows the variation of \( c_v \) with effective vertical stress for specimens from depths equal to 2.81, 6.43 and 9.81 m, tested in IL tests (grey region), as well as values obtained from CRS tests at \( \sigma'_{\text{vo}} \). The \( c_v \) profile for OC states is obtained by averaging the values retrieved from IL and CRS tests and extends up to a vertical stress of 60 kPa, the maximum *in situ* vertical stress in the profile (continuous line in Figure 4-11). The \( c_v \) profile for NC states is given by the results from IL tests at a depth of 6.43 m and applies to stresses larger than 60 kPa. Based on these profiles, the function “VLOOKUP” in the Excel® spreadsheet was used to select the value of \( c_v \) at \( \sigma'_{v,\text{ref-C}} \) in each sublayer. Open symbols are used in Figure 4-11a to represent the values of \( c_v \) selected in the Class-C predictions. Figure 4-11b compares the values of \( c_v \) used in both predictions. The \( c_v \) profile for NC states reduces by about around one order of magnitude, in agreement with the behaviour shown in Figure...
4.5c. The incorporation of these two $c_v$ profiles into the BH and FDM approaches is described in Section 5.2.

Figure 4-11: (a) $c_v$ curves from experimental data and (b) comparison of coefficients of consolidation from Class-A and Class-C predictions.

- $C_c$: in the Class-A predictions, values of $C_c$ were estimated from plots of $C_c$ vs. $\sigma' / \sigma'_{yield}$ at a depth of interest (Figure 9 in (Pineda et al. 2016)). Such a procedure was impractical considering the number of sublayers used in the analysis. Moreover, some scatter was also observed due to the natural variability of the samples. An improved selection procedure was attempted for the Class-C predictions. Figure 4-12a compares plots of $C_c$ vs. $\sigma' / \sigma'_{yield}$ obtained from CRS tests (borehole Inclo 2) (Pineda et al. 2016). The three main trends observed in this figure are consistent with the three layers composing the soil profile. Curves from depths equal to 1.86, 3.14 and 7.75 m were selected to represent the shallow sandy layer, the transition layer and the soft clay layer, respectively. Values of $C_c$ were obtained at the effective reference stress, $\sigma'_{v,ref}$ of each sublayer by using the function “VLOOKUP” in the Excel® spreadsheet. Estimated $C_c$ values are indicated in Figure 4-12a using solid symbols. Figure 4-12b shows the comparison between $C_c$ values adopted in Class-A and Class-C predictions. The new $C_c$ profile shows a consistent trend and less scatter compared to the original one.
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Figure 4-12: Compression index values: (a) selected from experimental data and (b) comparison of values from Class-A and Class-C predictions.

- \( C_\alpha \): the method used to determine \( C_\alpha \) was simplified in the Class-C predictions by selecting only four curves from Figure 12 (Pineda et al. 2016) to represent the behaviour of the three main layers that compose the entire profile. These correspond to specimens from depths of \( 0.91 \) m \((z < 2 \) m\), \( 2.81 \) m \((2 \) m \(< z \) < \( 4.5 \) m) as well as \( 5.22 \) m and \( 6.43 \) m \((4.5 \) m \(< z \) < \( 10.5 \) m). Similar estimates of \( C_\alpha \) were obtained with the new selection method (Figure 4-13a) compared to those used in the Class-A prediction.

- \( C_r \): in the Class-A predictions \( C_r \) was assumed equal to \( C_s \). Such an assumption may be useful in cases where the estimation of \( C_r \) is doubtful due to the strong influence of sample disturbance (which is not the case of the specimens tested (Pineda et al. 2016)). However, this assumption may not be realistic in weakly structured soils like Ballina clay, as important soil destructuration takes place if yielding occurs. In the recompression zone, an open fabric dominates the soil response, whereas at large stress levels, the original fabric is destroyed due to soil destructuration and reduction in void ratio. Therefore, differences between \( C_r \) and \( C_s \) may be significant. Inspection of CRS and IL data (Pineda et al. 2016) shows that \( C_r \) is slightly higher than \( C_s \) (~0.17 compared to ~0.10). Values of \( C_r \) used in the Class-C predictions are shown in Table 4-2 and Figure 4-13b. The lowest value in Figure 4-13b corresponds to the upper sandy layer.
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- $e_0$: the void ratio estimated at unstressed conditions ($\sigma = 0$) rather than the initial *in situ* void ratio (i.e., void ratio at $\sigma'_v$) was used in the Class-A predictions. Despite its minor influence on the predicted behaviour due to the good quality of the tested specimens, $e_0$ was re-assessed for each sublayer using the ratio $\Delta e/e_0$ (sample quality assessment) summarised in Table 2 (Pineda et al. 2016). Although small differences are observed in Figure 4-13c, void ratios at initial *in situ* stresses were used in the Class-C predictions.

**Figure 4-13:** Changes made to parameters that have minor effects on the predictions: (a) $C_a$; (b) $C_r$ and (c) $e_0$.

- $\alpha$: in the absence of *in situ* data for Ballina clay, the after construction response was fitted by increasing the value of $\alpha$ from 0.18 to 0.26. On the other hand, good representation of the measured data (within 3 years) was obtained by reducing $\alpha$ from 0.91 to 0.36. Although the back-analysis of lateral displacements was merely based on a fitting process, rather than a rational approach, the adjusted values lie within the range reported by Tavenas et al. (Tavenas et al. 1979).

### 4.5.2 Review of the methods of analyses used for the Class-C predictions

Two important features revealed by the measured data were the very high settlement rates immediately after the completion of the embankment and the slow dissipation of excess pore water pressures for elements in the centre of the clay layer. Some adjustments to the method of analysis were made in the Class-C predictions aiming at better
representing these two characteristics of the soil response. These adjustments can be summarised as:

1. As described above, two $c_v$ profiles were selected to simulate OC and NC soil states. The selection of $c_v$ values for each sublayer was based on: (i) the initial stress in the case of OC states, or (ii) the reference stress in the case of NC states.

2. The calculation of the degree of consolidation (NC states) was modified in the BH method due to the adoption of two $c_v$ profiles. The adjustment was implemented as:

$$U_{S2} = U_{S1} + (1 - U_{S1}) \times U_{c_v,NC} ; U_{S1} = U_{c_v,OC}$$

where:

$U_{S1}$ = average degree of consolidation up to yield stress;

$U_{S2}$ = average degree of consolidation from yield stress to final effective stress;

$U_{c_v,OC}$ = average degree of consolidation based on $c_v$ in OC state; and.

$U_{c_v,NC}$ = average degree of consolidation based on $c_v$ in NC state.

3. Creep settlements were assumed to occur just after yielding. This implies that creep may also occur during primary consolidation, which is consistent with the so-called Hypothesis B described in (Ladd 1977).

4. The ratios $d_s/d_m$ was decreased from 2 to 1.75, while $k_h/k_h'$ was increased from 1 to 1.75. These modifications are justified by the data provided by Almeida et al. (Almeida & Marques 2013), who reported ranges of variation between 1.5 and 5 and from 1 to 5 for $d_s/d_m$ and $k_h/k_h'$, respectively.

5. Smear effects were considered in the FDM. Thus, $c_h$ was factored by 0.57 to be consistent with the ratio $k_h/k_h'$. It was assumed that the permeability of the smear zone controls the dissipation of excess pore pressure in the horizontal direction.

6. The effect of settlement, which results in an increased distance between VWP5 and the static water table, was included in the predictions of the pore pressures.
4.5.3 Revised procedure for Class-C predictions

The modifications made in the Class-C predictions were implemented following a step-by-step approach that used Class-A predictions as a baseline. The approach is summarised in Table 4-3. This sequential approach was useful to evaluate the key factors affecting the settlement and pore pressure responses in Ballina clay. Figure 4-14 summarises the evolution of the settlement and pore pressure predictions (P1 to P9) according to the procedure described above. For simplicity, only predictions from the BH method are presented in Figure 4-14. Measured data and Class-A predictions are included in Figure 4-14 for comparison. The shift in the starting date from the 20th day to the 50th day is shown in predictions P1 (Figure 4-14a). An important improvement in the predictions of settlement and pore pressure is obtained in P2 by adopting the two $c_v$ profiles described above (Figure 4-14b). This demonstrates the key role played by $c_v$ on the rate of settlement. The total pore pressure response at a depth of 6 m is remarkably well predicted in P2. Refinements made to the selection of $C_c$ result in a better predictions of the settlement response in P3 (Figure 4-14c). The pore pressure response remains unchanged when using the new $C_c$ profile. Assuming that creep starts right after the $\sigma'_{yield}$ is exceeded does not affect the pore pressure response, but leads to a slight overestimation of the total settlement (prediction P4, Figure 4-14d). As the pore pressure response is also controlled by the smear zone created by the mandrel, two minor modifications were also made to the method of analysis as described above. The adoption of some degree of permeability anisotropy (P5) shown in Figure 4-14e led to even better predictions overall, in particular of the total pore pressure response. In prediction P6, the modification in $d/dm$ results in a slightly reduced total settlement (with respect to P4) and better predictions of the total pore pressure (Figure 4-14f). The last three modifications made in P7 ($C_\alpha$, Figure 4-14g), P8 ($C_r$, Figure 4-14h) and P9 ($e_0$, Figure 4-14i) result in minor refinements to the predictions of total settlements and total pore pressures.

Table 4-3: Approach used and changes made for Class-C predictions

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<th>Figure 4-14</th>
<th>Label</th>
<th>Changes</th>
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<td>Embankment completion was changed from 20th day to 50th day, and excess pore pressure from stage 1 was included in the analysis</td>
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<td>Description</td>
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<td>----------</td>
<td>-----------</td>
<td>-------------</td>
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<tr>
<td>4-14(b)</td>
<td>$P_2-c_v$</td>
<td>Coefficient of consolidation defined for NC and OC stages</td>
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<td>4-14(c)</td>
<td>$P_3-C_z$</td>
<td>Compression index values refined</td>
</tr>
<tr>
<td>4-14(d)</td>
<td>$P_4-C_{u-time}$</td>
<td>Creep starts immediately after yield stress is exceeded</td>
</tr>
<tr>
<td>4-14(e)</td>
<td>$P_5-k_h/k'_h$</td>
<td>Ratio of permeability in horizontal direction to permeability of smear zone in horizontal direction was changed from 1 to 1.75</td>
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<tr>
<td>4-14(f)</td>
<td>$P_6-d_L/d_m$</td>
<td>Ratio of smear zone diameter to equivalent mandrel diameter was changed from 2 to 1.75</td>
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<td>4-14(g)</td>
<td>$P_7-C_{u}$</td>
<td>Secondary compression index values refined</td>
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<tr>
<td>4-14(h)</td>
<td>$P_8-C_r$</td>
<td>Recompression index values refined</td>
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<tr>
<td>4-14(i)</td>
<td>$P_9-e_0$</td>
<td>Initial void ratio values refined</td>
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</table>
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Figure 4-14: Sequential procedure followed in the Class-C prediction to incorporate the new set of parameters.

4.5.4 Comparison of Class-C predictions with measured data

Figure 4-15 compares the measured and predicted surface settlements at two different sections. It can be seen that the use of revised parameters as well as the additional modifications implemented into the calculation procedure led to an excellent agreement between predictions and the field data. Similar results were obtained regardless of the method of analysis employed (BH and FDM). Figure 4-15 and Figure 4-16 suggest that
the predicted settlements, either on the surface or at depth, tend to be underestimated between 280 to 500 days. The maximum difference, however, is less than 10 % and, at later times, the Class-C predictions and measured data tend to converge.

Figure 4-17 shows that the predicted initial pore water pressure overestimates the values measured at depths of 2 and 6 m. This is due to the fact that the construction of the 1-m thick working platform and 2-m thick embankment was assumed to have taken place in one single day, which does not represent the real construction history. This assumption implies undrained loading. Therefore, the maximum initial excess pore water pressure in the predictions is equal to the stress increment imposed by the embankment at a depth of interest. On the other hand, the small excess pore water pressure measured after embankment completion (around 50 and 60 % of the theoretical vertical stress increment) agrees with the behaviour discussed in Leroueil et al. [16]. They reported minor excess pore pressure development before yielding. This pore pressure response is captured to some degree in the predictions by the fast pore pressure dissipation occurring after peak, which is controlled by the $c_v$ in OC state. After yielding, the value of $c_v$ is reduced, leading to a reduced rate of dissipation. This response is controlled by the $c_v$ profile in NC states. Overall, the use of two profiles for representing the pore water pressure response seems to capture the measured behaviour in a consistent way.

![Figure 4-15: Comparison between predicted (Class-C) and measured surface settlements for SP2 and SP3.](image)
Figure 4-18 shows that the predicted lateral displacements agree reasonably well with the measured data. However, the predictions cannot account for different lateral displacements on each side of the embankment.

![Graph showing predicted vs measured lateral displacements](image)

**Figure 4-16:** Comparison between predicted (Class-C) and measured relative settlements at three different depths.

![Graph showing predicted vs measured pore pressure responses](image)

**Figure 4-17:** Comparison between predicted (Class-C) and measured pore pressure responses over time at three different depths.
4.6 SENSITIVITY ANALYSIS

A sensitivity analysis was performed to evaluate the influence of different soil parameters on the predicted behaviour. To some extent, this provides insight on the uncertainty associated with the selection of soil parameters either due to poor interpretation of test results or inappropriate assessment of the stress path experienced by the soil profile at the centerline due to the embankment construction. Several combinations of parameters may lead to similar settlement predictions. However, parameter selection should have a rational justification based on available data. Eight different scenarios were evaluated in this analysis. These are summarised in Table 4-4. To keep the analysis as simple as possible, the remaining parameters/methods were kept identical to those used in the Class-C predictions (Table 4-2). Only the BH method was used for the sensitivity analysis described here.

Surface settlements from the analyses were compared with the measured data at four different timeframes: 122nd day, 248th day, 501st day and 1090th day. Predicted and measured pore water pressures (6 and 9.5 m depths) were compared at 248th day, 501st day and 1090th day. Figure 4-19a compares the predicted surface settlements for the eight scenarios with the measured data. Class-C predictions were also included for comparison. The inspection of Figure 4-19b shows that total settlements can be over-predicted by more than 30 % and under-predicted by about 40 %. An upper bound is given by T4-cv where cv was adopted from a CRS test at a depth of 3.14 m. The higher cv used in this case
resulted in faster dissipation of excess pore pressure. This suggests that creep settlements started earlier as soil consolidated at a faster rate. When the upper bound parameters for $C_a$ (T8-$C_a$) are used, total settlements calculated were increased by 10% as to the measured data. There is a tendency for underestimating the total settlements as it was also observed in Class-A predictions presented in the EPS.

The lowest predictions for the total settlements arose from T2-$C_c$ where the lower bound profile of $C_c$ (CRS at depth 3.14 m) was used. Values of $C_c$ were approximately 60% lower than the values adopted in the Class-C predictions. The upper bound values for $C_c$ (T3-$C_c$), which tend to be similar to the adopted $C_c$ profile, resulted in 10% differences compared to the measured data. The use of uncorrected $\sigma_{yield}$ values reduced the predicted total settlements by 30%. It is important to note that $\sigma_{yield}$ is also used in the selection of $C_c$ and $c_v$. Therefore, the error involved may be much higher.

**Table 4-4: Selected scenarios for the sensitivity analysis**

<table>
<thead>
<tr>
<th>Cases</th>
<th>Changes</th>
<th>Justification</th>
</tr>
</thead>
<tbody>
<tr>
<td>T1-OCR</td>
<td>$OCR$ uncorrected by rate effects</td>
<td>$\sigma_{yield}$ reported in [2] is uncorrected by strain rate effects</td>
</tr>
<tr>
<td>T2-$C_c$</td>
<td>$C_c$ from CRS data at depth 3.14 m</td>
<td>$C_c$ from this test data represents lower bound solutions</td>
</tr>
<tr>
<td>T3-$C_c$</td>
<td>$C_c$ from CRS data at depth 5.89 m</td>
<td>$C_c$ from this test data represents upper bound solutions due to the large variation of $C_c$ with increasing stress level</td>
</tr>
<tr>
<td>T4-$c_v$</td>
<td>$c_v$ from CRS data at depth 3.14 m for stage 2 only</td>
<td>$c_v$ from this test data represents upper bound solutions</td>
</tr>
<tr>
<td>T5-$c_v$</td>
<td>Average $c_v$ values used</td>
<td>Three average values of $c_v$ were used for three different layers in each stage, representing typical approach taken by practitioners</td>
</tr>
<tr>
<td>T6-$t_{90}$</td>
<td>Secondary compression starting after 90% consolidation</td>
<td>A common approach adopted in practice</td>
</tr>
<tr>
<td>T7-$C_a$</td>
<td>$C_a = 0.002$ assumed for all layers</td>
<td>This average value of $C_a$ represents lower bound solutions</td>
</tr>
<tr>
<td>T8-$C_a$</td>
<td>$C_a = 0.06$ assumed for all layers</td>
<td>This average value of $C_a$ represents upper bound solutions</td>
</tr>
</tbody>
</table>
Chapter 4: Predicted and measured behaviour of an embankment on PVD-improved Ballina clay

Figure 4-19: Comparison of total settlements predicted for different cases with measured data and Class-C predictions: (a) Evolution of settlements with time (b) Percentage variations of the settlements for each case from the measured data.

Figure 4-20 shows the outcomes of pore pressures evaluated at 6 and 9.5 m depths. There is a tendency for underestimating the total pore pressure response. As expected, only \( c_v \) has a major influence on the predicted pore water pressure. Case T4-c\( v \) resulted in lower predictions as a consequence of the higher rate of excess pore pressure dissipation. Overall, these analyses suggest that the uncertainties for predicting total settlements can vary within \( \pm 35\% \) while for total pore pressure response, it can vary within \( \pm 10\% \).

Figure 4-20: Comparison of total pore pressures predicted for different cases for Class-C predictions with measured data.

4.7 CONCLUDING REMARKS

Simple hand calculations coupled with the finite-difference method were used to predict the behaviour of an embankment built on PVD-improved Ballina clay. Four soil parameters were identified to control the settlements and pore pressure responses: OCR,
$c_v$, $C_c$ and $C_\alpha$. Comparisons of Class-A predictions with the measured data showed important discrepancies, mainly in terms of settlement rate and pore pressure dissipation rate. These two aspects were strongly affected by the values of $c_v$ selected from laboratory tests, which seemed to be too high compared with the _in situ_ value of $c_v$ after construction. The selection of $c_v$ was also affected by the choice of OCR, as OCR influences the value of $\sigma'_{yield}$ used in the analyses. $C_c$ and $C_\alpha$ altered the settlement magnitude but did not affect the pore pressure response. The time assumed for creep to start, which affected the analyses involving the $C_\alpha$ parameter, influenced the settlement rate.

Class-C predictions were made with prior knowledge of the performance of the embankment. Soil parameters were revised, in particular $C_r$, $C_\alpha$, $C_c$ and $c_v$. Class-C predictions showed good agreement with the measured data, indicating the importance of considering realistic parameters for the calculations. It was demonstrated that good predictions were obtained by using two separate $c_v$ profiles (OC and NC states). The sensitivity analysis, which was carried out by varying key soil parameters, yielded a range of responses within ± 40% for the total surface settlement and ± 10% for the total pore pressures, with respect to the measured values.

This chapter demonstrates that simple techniques such as hand calculation and finite-difference methods are still relevant in current practice to predict settlements of embankments. The chapter described a methodology that can be used to carefully select soil parameters from laboratory and _in situ_ data. The use of this methodology led to predictions of settlements and total pore pressure response that were in good agreement with measured data.
Chapter 4: Predicted and measured behaviour of an embankment on PVD-improved Ballina clay

4.8 APPENDIX

Traditionally, vertical stress increments have been estimated using elastic solutions which vary depending on particular loading types and boundary conditions. The soil is assumed to be a semi-infinite elastic, homogeneous, isotropic and weightless material. For the stress calculation, in this case, simplified solutions (Aysen 2002) based on Boussinesq’s theory have been used. The load from the toe of the embankment to the crest level of the embankment for both sides is assumed to vary linearly, while the load at the crest level is assumed to be uniform. Thus, the stresses from the embankment are assumed to be the combination of uniformly and linearly loaded infinite strips.

The geometry of the embankment used for the estimation of the vertical stresses with depth is given in Figure 4-21. The increment of vertical stress at depth \( z \) was determined by multiplying the calculated influence factors by the loading from the embankment as shown in Equation 4-A1 (Aysen 2002). Two sides from left and right were determined separately due to the asymmetrical geometry of the embankment for the two different construction stages.

\[
\begin{align*}
\Delta \sigma_z &= q_{q,1} + q_{q,2} \\
I_{q,1} &= \frac{1}{\pi} \left[ p + r \frac{\tan^{-1} \frac{p}{1 + r^2 + pr}}{p} \right] \\
I_{q,2} &= \frac{1}{\pi} \left[ x + y \frac{\tan^{-1} \frac{x}{1 + y^2 + xy}}{x} \right]
\end{align*}
\]

\( \Delta \sigma_z \) = vertical stress increment at depth \( z \)

\( q = \gamma h_e \) (kPa)

\( \gamma \) = bulk unit weight of the fill (kN/m\(^3\))

\( h_e \) = height of the embankment (m)

\( I_{q,1} \) = influence factor based on the depth and geometry of the embankment from left side

\( I_{q,2} \) = influence factor based on the depth and geometry of the embankment from right side

---

Figure 4-21: Vertical stress increment under an embankment.
\[ p = \frac{a_1}{z} \; ; \; r = \frac{b_1}{z} \; ; \; x = \frac{a_2}{z} \; \text{and} \; y = \frac{b_2}{z}; \]

Figure 4-22 shows the distribution of total vertical stresses at three depths underneath the embankment: 0.5 m, 5 m and 10 m. According to this approach, the total vertical stress increment at the centreline of the embankment reduces from 63 kPa at the ground surface to only 53 kPa at 10 m depth \((\Delta \sigma_{\text{ave}} \approx 58 \text{ kPa})\). This suggests that all soil layers considered in the problem move from a slightly over-consolidated to a normally consolidated state due to the loads imposed by the embankment.

**Figure 4-22: Vertical stress distribution underneath the embankment at three different depths**
4.9 REFERENCES


Barron, RA 1948, 'Consolidation of fine-grained soils by drain wells', *Journal of the Soil Mechanics and Foundation Division (ASCE)*, vol. 73, no. 6, pp. 811-835. Available from: ASCE.


CHAPTER 5. MECHANICAL BEHAVIOUR OF INTACT AND RECONSTITUTED CALCAREOUS SILTS

Abstract: Calcareous silts are encountered in many offshore areas where oil and gas exploitations are taking place (e.g., Arabian gulf, south of Brazil, south east and north west of Australia). As these explorations are going into deeper water, the probability of encountering these types of soils gets higher. Understanding behaviour of calcareous silts still remain challenging as undisturbed silt samples are difficult to obtain, and most studies rely on reconstituted silt samples. Furthermore, these materials are highly variable in composition and behaviour, ranging from cemented calcarenites, through uncemented sands and silty sands, calcareous muds and high-plasticity calcareous ‘clays’. The purpose of this chapter is to characterise the mechanical behaviour of intact and reconstituted calcareous silts from two different water depths. The comparisons are done based on microstructure characterisation using scanning electron microscopy (SEM) images supported by index tests, one-dimensional compression tests and undrained monotonic triaxial tests. The results have shown that the characteristics of calcareous silts such as large intra-voids within particles with random particles shapes and sizes affect silt compressibility and undrained shearing behaviours for both intact and reconstituted specimens.
5.1 INTRODUCTION

Calcareous sediments are encountered in many offshore areas where oil and gas explorations are taking place (e.g., Arabian gulf, southern Brazil, south east and north-west regions of Australia). Previous research has shown that calcareous sediments are highly variable in composition and behaviour, ranging from cemented calcarenites, through uncemented sands and silty sands, calcareous muds and high-plasticity calcareous ‘clays’ (McClelland 1988; Nyland 1988; Price 1988; Mao & Fahey 1999). With offshore activities extending into deep water areas, the sediments that engineers have to build on are essentially fine-grained calcareous soils, with various amounts of silt and clay-sized particles.

While a significant amount of work has been published on the behaviour of cemented and uncemented calcareous sands (Carter et al. 1988; Coop 1990; Coop & Atkinson 1993; Sharma & Ismail 2006), knowledge of the response of fine-grained calcareous silts is still limited. Mao and Fahey (2003) had described the behaviour of two calcareous silts with various amounts of clay-sized particles. Their study focused on the response in undrained cyclic simple shear tests of reconstituted specimens. One silty material studied with 60% by weight of clay-sized particles (‘muddy’ silt), was reconstituted using a synthetic flocculant and heat treatment to reproduce the microstructure and behaviour of the natural soil (Mao & Fahey 1999). The results of monotonic undrained simple shear tests on normally consolidated specimens showed a contractive response, similar to the undisturbed soil. However, unflocculated specimens exhibited a dilative response, which may be attributed to a lower initial void ratio and a different soil structure. The second silt material studied was mixed with some commercially produced carbonate powder and silicon oil (resulting in 73% silt and 27% sand). For this silt, the undrained shear behaviour was dilative with strong strain hardening characteristics.

Previous studies have shown that various factors affect the shear behaviour of calcareous silty soils, such as the clay-sized fraction, initial void ratio, stress history and soil structure (Cola & Simonini 2002; Thevanayagam et al. 2002; Murthy et al. 2007; Anantanasakul et al. 2012). Recent work by Lehane et al. (2014) has compared the mechanical response of two carbonate sediments from Australia’s North West Shelf and has shown that fines content affects the undrained shear strength of soils, by controlling
the void ratio/density. However, published data on intact calcareous silty soils are still limited and there is a lack of guidance on how test results obtained from reconstituted specimens may be used to infer the behaviour of intact soil. If this type of guidance becomes available, it will allow extension of the conclusions obtained from physical modelling, which is typically performed on reconstituted samples, to the in situ (intact) soil conditions. The aim of this paper is to make a contribution in this direction, by conducting an experimental program on intact and reconstituted calcareous silty soils, which includes microscopic and mechanical soil characterisation.

5.2 SOILS TESTED AND EXPERIMENTAL PROCEDURE

Two natural calcareous silts from offshore Western Australia were tested; Silt A from 575 m water depth and Silt B from 1341 m water depth. The samples were obtained using a 80 mm diameter piston tube sampler. The samples available for testing were from relatively shallow depth below the mudline, between 3 and 4 m. Both silts contained a significant amount of clay-size particles, or mud, as indicated by the grain size distribution curves shown in Figure 5-1 (23 % for Silt A and 45 % for Silt B). In-situ water content, index properties and calcium carbonate (CaCO$_3$) content are listed in Table 5-1. For both silts, the natural water content was close to the liquid limit and the CaCO$_3$ content was greater than 75%. According to the Unified Soil Classification System (USCS), Silt A is classified as silt with traces of sand while Silt B is clayey silt with high plasticity for both soils based on their Atterberg limits. Additionally, using the proposed classification by Fookes (1985), the two silts can also be classified as calcareous muddy silts.
Figure 5-1: Particle size distribution for Soil A and Soil B

Table 5-1: Index properties of Soil A and Soil B

<table>
<thead>
<tr>
<th>Silts</th>
<th>Initial water content, $w_0$</th>
<th>Liquid limit, $w_L$</th>
<th>Plasticity index, $I_p$</th>
<th>Specific gravity, $G_s$</th>
<th>Calcium carbonate content (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>(%)</td>
<td>(%)</td>
<td>(%)</td>
<td>(+)</td>
<td>(%)</td>
</tr>
<tr>
<td>Silt A</td>
<td>50.2</td>
<td>52.8</td>
<td>22.2</td>
<td>2.69</td>
<td>76.4</td>
</tr>
<tr>
<td>Silt B</td>
<td>81.2</td>
<td>82.5</td>
<td>34.6</td>
<td>2.70</td>
<td>91.4</td>
</tr>
</tbody>
</table>

5.2.1 Reconstituted Specimens

To prepare reconstituted specimens of each soil, a slurry with a targeted water content of around 1.2 times the liquid limit was prepared by mixing the original soil sample with deionised water. Next, the slurry was left for at least 12 hours and was mixed again before it was poured into a greased cylindrical consolidation tube with internal diameter of 72 mm. The silt slurry was then loaded to axial stress ($\sigma_a$) levels of approximately 9, 16, 30 and 60 kPa in stages. The slurry was allowed to consolidate under each stress increment until the axial displacement of the top cap was less than 1 mm/day. Finally, the sample was extruded out from the tube and trimmed according to the required dimensions for each element test.
5.2.2 Microstructure characterisation using scanning electron microscope (SEM) and X-ray diffractometer (XRD)

Small cubes approximately 5 mm in size were cut from both intact and reconstituted specimens for microscopic characterisation. As the SEM used was not suitable for wet specimens, specimens were freeze-dried using liquid nitrogen and a freeze drying machine. The dried specimens were cracked open in the middle section and the cracked surface was used as the surface for the scanning images. The specimens were glued onto the stubs and coated with carbon paint. The carbon paint improves the specimen imaging by preventing electron charges build-up on the specimens. The SEM specimens were then scanned using a Zeiss 1555 VP-FESEM using various settings to produce the SEM images. In addition, X-ray crystallography (Panalytical Empyrean XRD) was performed on the specimens to identify the mineralogy of each soil.

5.2.3 Fall-cone sensitivity tests

A fall-cone apparatus was used to measure the sensitivity (ratio of intact to remoulded shear strength) of the two soils. A 30° (apex angle) cone of 80 g was used. The test was carried out by letting the cone fall freely and penetrates under its own weight for 5 s. A 45 mm long section of the sampling tube was cut and tested at five points on each side (top and bottom) and the measured penetration depths were averaged to give the measurement for the intact specimen. The specimen was then extruded from the tube and remoulded for at least 5 minutes. The soil was filled into a cup of 56 mm in diameter and 42 mm in height and was subjected to fall cone penetration. The test was repeated three times and the measured penetration depths were averaged to give the measurement for the remoulded specimen.

5.2.4 One-dimensional compression tests using constant rate of strain (CRS) apparatus

CRS tests were carried out in general accordance with ASTM Standards (ASTM Standard D4186 2006) with a specimen size of 24 mm in height and 43 mm in diameter. The specimens were saturated using a back pressure of 200 kPa for at least 24 hours, under a low vertical effective stress of 3 kPa to prevent swelling during saturation. In one-dimensional compression, back pressure was applied at the top of the specimen while pore pressure was measured at the bottom. The specimens were loaded up to a maximum vertical stress of 1600 kPa using a strain rate of 1.85 %/hr and then unloaded at a rate of
2.4 %/hr (no pause was applied prior to unloading, in contrast to the ASTM standard). The maximum pore-water pressure ratio in the normally consolidated range during the loading phase of the test for both silts was less than 15 %, as required in the standard.

### 5.2.5 Anisotropically consolidated undrained (CAU) triaxial tests

The triaxial tests were performed using a GDS Enterprise Level Dynamic Triaxial Testing System (ELDYN). Triaxial specimens were prepared by extruding a sample from a 140 mm long section of sampling tube, which was then trimmed to a specimen of 105 mm in height and 50 mm in diameter. Before putting the specimen on the base pedestal, the dimensions of the specimens were measured and all the lines in the triaxial system were saturated with de-aired water. In the saturation phase, 5 kPa effective stress was first applied to the specimen by increasing the cell pressure and back-pressure to 30 kPa and 25 kPa, respectively. This effective stress was maintained for an hour before the next increment of stress was applied. In the next increment, the cell pressure and back-pressure were increased up to 510 kPa and 505 kPa, respectively at a rate of 30 kPa/hr. The specimen was then left for at least 48 hours before the B-check was performed. A B-value of at least 0.96 was required before proceeding to the consolidation stage.

Various consolidation stresses were considered, corresponding to either normally consolidated or over-consolidated states. The specimen was consolidated anisotropically (K₀ = 0.5 was imposed in this stage) in two stages up to the selected stress state. In the first stage, the axial stress was increased until the state of stress reached the required K₀ value. In the second stage, a linear stress path was prescribed until the selected stresses were reached. A deviatoric stress rate of 6 kPa per day was used. Consolidation was considered complete when the excess pore pressure had fully dissipated and the axial strain rate was around 0.0035 %/day or less. At the end of consolidation, the specimen was sheared at a nominal constant rate of 7 %/day up to 20 % axial strain.

### 5.3 RESULTS

#### 5.3.1 Fall cone sensitivity tests

The undrained shear strength was calculated using the equation:

\[ s_u = K Q / h^2 \]  (5-1)
where $K$ is a cone factor that depends on the conical angle, $Q$ is the weight of the cone and $h$ is the average measured penetration depth. $K$ is determined to be 0.85 based on the angle of $30^\circ$, according to Kumar and Wood (1999). The sensitivities of each specimen are shown in Table 5-2. Based on the reviews done by Abuhajar et al. (2010) on established soil sensitivity classifications such as CFEM (2006) or Canadian Engineering manual and Rankka et al. (2004) or Swedish classification, Silt A and B can be characterised medium to high sensitive soils for most of the existing classifications in their paper with higher sensitivity of Silt B. Both silts have similar remoulded undrained shear strength, but Silt B has higher undisturbed undrained shear strength compared to Silt A despite having higher water content. This may indicate that Silt B is more sensitive to remoulding due to its open soil structure i.e. high void ratio.

**Table 5-2: Results of fall cone sensitivity tests.**

<table>
<thead>
<tr>
<th>Silts</th>
<th>Initial water content, $w_o$ (%)</th>
<th>Remoulded water content, $w_r$ (%)</th>
<th>Undisturbed undrained shear strength, $s_u$ (kPa)</th>
<th>Remoulded undrained shear strength, $s_{ur}$ (kPa)</th>
<th>Sensitivity, $s_u/s_{ur}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>60.4</td>
<td>56.3</td>
<td>18.5</td>
<td>2.5</td>
<td>7.35</td>
</tr>
<tr>
<td>A2</td>
<td>57.2</td>
<td>59.2</td>
<td>12.2</td>
<td>1.8</td>
<td>6.70</td>
</tr>
<tr>
<td>B1</td>
<td>82.9</td>
<td>85.0</td>
<td>28.9</td>
<td>3.0</td>
<td>9.60</td>
</tr>
<tr>
<td>B2</td>
<td>85.8</td>
<td>86.8</td>
<td>19.0</td>
<td>2.4</td>
<td>7.98</td>
</tr>
</tbody>
</table>

**5.3.2 Microstructure characterisation**

A few SEM images were selected to highlight the soil microstructure in intact and reconstituted specimens of both silts. Figure 5-2 shows the comparison of soil microstructure between intact and reconstituted Silt A specimens at 1 500 and 10 000 times magnification. From the lower magnification images, it can be seen that the microstructure of Silt A is composed of a clay matrix made of flocculated clay particles, with silt inclusions (large angular particles). For the intact specimen, a large size of microfossil can be seen at the centre of the image, with size approximately 30 μm to 50 μm (Figure 5-2 (a)). The microfossil has a honeycomb structure with intra voids of radius ranging between 3 μm to 5 μm. This type of structure is not visible in the case of the reconstituted specimen, suggesting that the microfossil particles may have been crushed during the reconstitution process. At higher magnification, the microstructure of the intact specimen appears quite open with relatively large voids between particles, while for the
reconstituted specimen, a more compacted structure can be observed, with smaller voids (Figure 5-2 (c) and (d)).

![SEM images of Silt A](image1)

(a) Intact Silt A (x 1 500)  
(b) Intact Silt A (x 10 000)

![SEM images of Silt A](image2)

(c) Reconstituted Silt A (x 1 500)  
(d) Reconstituted Silt A (x 10 000)

**Figure 5-2: SEM images of Silt A**

Figure 5-3 compares the microstructure between intact and reconstituted Silt B specimens at 1 500 and 5 000 times magnification. In contrast with Silt A, Silt B does not exhibit a typical clay microstructure, but instead the microstructure is composed of a conglomerate of broken microfossils and elongated and angular particles. However, from the PSD, Silt B has a higher percentage of clay-size particles compared to Silt A. A large presence of coccoliths can be seen in the intact specimen of Silt B as shown on Figure 5-3 (a) and (b). Coccoliths are microscopic circular plates resulting from the breakage of coccospHERES (formed by single-celled algae such as *Emiliania huxleyi*). They are individual chalk plates that contribute to a significant presence of calcium carbonates in this soil. The presence of coccoliths in Silt B is consistent with the water depth of this soil (i.e. 1341 m) as coccoliths are normally found in deeper water. In the case of the reconstituted specimen of Silt B, full size coccoliths cannot be seen, which suggests that
the coccoliths may have been damaged during the process of mixing and reconstituting the specimen. The arrangement of particles in reconstituted specimen of Silt B appears to be well distributed with abundant elongated and angular particles (Figure 5-3 (c) and (d)). Similarly, as for Silt A, the structure of the intact specimen appears looser with larger voids compared to the reconstituted specimen.

![SEM images of Silt B](image)

(a) Intact Silt B (x 1,500)  
(b) Intact Silt B (x 5,000)

(c) Reconstituted Silt B (x 1,500)  
(d) Reconstituted Silt B (x 5,000)

**Figure 5-3: SEM images of Silt B**

With reference to XRD results, both silts were composed of mainly calcite, aragonite, halite and alpha quartz (low quartz) with variable percentages, in agreement with previous studies of calcareous studies sands (Carter et al. 1988; Coop 1990; Coop & Atkinson 1993; Sharma & Ismail 2006).

### 5.3.3 Compressibility

Figure 5-4 and Figure 5-5 show the compression curves obtained from CRS tests carried out on intact and reconstituted specimens of both silts. The compression parameters determined from the CRS tests results are listed in Table 5-4. Both intact silts exhibit an initially stiff response at low stress followed by a gradual increase in compressibility until the response becomes almost linear in the $e$-$\log\sigma'_v$ plot. However,
no sharp bend in the curve is observed at yield stress, indicating that the ‘intact’ specimens may have been subjected to disturbance during tube sampling or specimen preparation. Silt A has a lower void ratio compared to Silt B but it appears that Silt A is more compressible than Silt B. Compared to intact specimens, both reconstituted silts show a stiffer response, as observed in their normalised compression curves.

**Figure 5-4:** Compression curves of soil A: (a) actual data and (b) normalised data

**Figure 5-5:** Compression curves of Soil B: (a) actual data and (b) normalised data

Due to the largely non-linear response of soil during compression, it is not meaningful to report a unique value for the compression index, $C_c$. Therefore, a continuous variation of $C_c$ was derived, by differentiating a sixth-degree polynomial equation fitted to the compression curve in $e$-$\log \sigma'$ space. Figure 5-6 illustrates the change of compression indices for both soils when compressed to stress levels up to ten
times the vertical yield stress, $\sigma_{\text{yield}}$, determined using Casagrande method. The $C_c$ of both silts falls closely within the range observed for calcarenites (cemented calcareous sands), for which $C_c \sim 0.4 - 0.55$ (Carter et al. 1988). For intact Silt A, once $C_c$ has reached its maximum value of approximately 0.4, it remains stable with increasing effective vertical stress, $\sigma'$, whereas for reconstituted Silt A, $C_c$ increases continuously with increasing $\sigma'$. However, the range of $C_c$ for both intact and reconstituted specimens of Silt A is almost similar. In contrast to Silt B, $C_c$ is increasing with increasing $\sigma'$ for both intact and reconstituted specimens while the range of $C_c$ values is significantly lower for the reconstituted specimen, half the values of intact specimens. The swelling index, $C_s$, is similar for both soils and is not affected by the reconstitution process.

**Figure 5-6: Change of compression indices with normalised effective vertical stress: (a) Soil A and (b) Soil B**

The values of the estimated in-situ vertical effective stress, $\sigma'_v0$, and $\sigma_{\text{yield}}$ determined using the Casagrande method, are listed in Table 5-3. For both silts, the yield stress ratio (YSR = $\sigma_{\text{yield}}/\sigma'_v0$) is greater than 1, even though there is no evidence of previous stress loading history for these sediments. ‘Apparent’ pre-consolidation is a common feature of many offshore clay deposits, which has been attributed by several authors to aging effects (Schmertmann 1991; Puech et al. 2005). Compared to Silt A at the same depth, Silt B which is more fine-grained, exhibits a higher natural void ratio, higher yield stress, higher yield stress ratio and higher compression index.
Table 5-3: Compressibility parameters from CRS tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>e₀</th>
<th>σ'₀ (kPa)</th>
<th>σ'yield (kPa)</th>
<th>σ'/σ'₀</th>
<th>Cₛ</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>1.62</td>
<td>21</td>
<td>35</td>
<td>1.7</td>
<td>0.03</td>
</tr>
<tr>
<td>A2</td>
<td>1.60</td>
<td>24</td>
<td>36</td>
<td>1.5</td>
<td>0.04</td>
</tr>
<tr>
<td>B1</td>
<td>2.43</td>
<td>13</td>
<td>60</td>
<td>4.6</td>
<td>0.05</td>
</tr>
<tr>
<td>B2</td>
<td>2.36</td>
<td>16</td>
<td>80</td>
<td>5.0</td>
<td>0.05</td>
</tr>
<tr>
<td>AR</td>
<td>1.33</td>
<td>N/A</td>
<td>60</td>
<td>1.0</td>
<td>0.03</td>
</tr>
<tr>
<td>BR</td>
<td>1.97</td>
<td>N/A</td>
<td>62</td>
<td>1.0</td>
<td>0.05</td>
</tr>
</tbody>
</table>

5.3.1 Behaviour in undrained triaxial compression

The list of triaxial tests carried out is summarised in Table 5-4. Three tests were conducted on each soil, with varying values of vertical consolidation stress, σ'vc. For Silt A, values of σ'vc were selected as follows: (i) close to the in situ vertical stress, (ii) close to the yield stress, (iii) significantly larger than the yield stress. A similar logic was followed for Silt B, except for the lowest consolidation stress value, which was selected to be lower than the yield stress, but high enough such that it could be controlled accurately by the loading system (the in situ vertical stress was quite low for this case – approximately close to 10 kPa).

Table 5-4: Summary of triaxial compression tests

<table>
<thead>
<tr>
<th>Specimen</th>
<th>e₀</th>
<th>σ'vc (kPa)</th>
<th>σ'rec (kPa)</th>
<th>εvol (%)</th>
<th>εc</th>
<th>Peak deviator stress, qpeak (kPa): Axial strain at qpeak, εaxial,peak (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>A1</td>
<td>1.67</td>
<td>20</td>
<td>10</td>
<td>2.83</td>
<td>1.59</td>
<td>21.5</td>
</tr>
<tr>
<td>A2</td>
<td>1.61</td>
<td>35</td>
<td>17.5</td>
<td>4.51</td>
<td>1.50</td>
<td>28.0</td>
</tr>
<tr>
<td>A3</td>
<td>1.64</td>
<td>70</td>
<td>35</td>
<td>9.42</td>
<td>1.39</td>
<td>58.5</td>
</tr>
<tr>
<td>B1</td>
<td>2.21</td>
<td>35</td>
<td>17.5</td>
<td>5.14</td>
<td>2.04</td>
<td>45.3</td>
</tr>
<tr>
<td>B2</td>
<td>2.41</td>
<td>70</td>
<td>35</td>
<td>6.30</td>
<td>2.19</td>
<td>82.7</td>
</tr>
<tr>
<td>B3</td>
<td>2.5</td>
<td>200</td>
<td>100</td>
<td>13.27</td>
<td>2.04</td>
<td>219.5</td>
</tr>
<tr>
<td>AR</td>
<td>1.33</td>
<td>70</td>
<td>35</td>
<td>5.35</td>
<td>1.20</td>
<td>63.4</td>
</tr>
<tr>
<td>BR</td>
<td>2.01</td>
<td>70</td>
<td>35</td>
<td>3.44</td>
<td>1.91</td>
<td>63.3</td>
</tr>
</tbody>
</table>

Figure 5-7 shows the response for intact and reconstituted specimens of Silt A during undrained triaxial compression. Specimen A1, which can be considered as “over-consolidated”, shows a contractive-dilative behaviour, with the deviator stress increasing monotonically up to its maximum value, whereas the other “normally consolidated”
specimens exhibit a contractive response, with the deviator stress reaching a peak before decreasing. The reconstituted specimen, AR reaches higher peak shear strength (8 % higher) compared to the intact specimen consolidated to the same stress level (A3). This is consistent with the higher density (lower void ratio) of the reconstituted specimen. Except for A3, all tests reached the same critical state line in the effective stress space.

Figure 5-7: Undrained shearing response of intact and reconstituted specimens of Silt A

Figure 5-8 presents the undrained shearing response of Soil B. Both specimens B1 and B2 exhibit a contractive-dilative response with monotonically increasing deviator stress, whereas the “normally consolidated” specimens B3 and BR show a contractive response. Only specimen BR displays a peak deviator stress followed by a gradual drop. In comparison with the intact specimen consolidated to the same stress level (B2), the reconstituted specimen reaches a lower peak shear strength (31% lower). The vertical consolidation stress for specimen B2 was selected to be close to the yield stress (around 60 to 80 kPa, Table 5-4), however, the shearing response obtained seems to indicate that the specimen may have been “over-consolidated” (similar response as B1). In comparison to soil A, the peak shear strength for all specimens of soil B occurs at larger shear strains.
Figure 5-8: Undrained shearing response of intact and reconstituted specimen of Silt B

The excess pore pressure response during undrained shearing for intact and reconstituted specimens for both silts shown in Figure 5-9 and Figure 5-10 respectively. Pore pressure built up rapidly at the start of shearing for both soils with substantial fluctuations of pore pressure response for both silts and no obvious trend can be determined. Comparing intact and reconstituted specimens of Silt A and Silt B consolidated to the same effective stress ($\sigma'_{vc}=70$ kPa), intact specimen of Silt A showed contradictory behaviour with its reconstituted specimen; increasing excess pore pressure with increasing strain for intact specimen while decreasing excess pore pressure with increasing strain for reconstituted specimen. It is possible that the reduction of excess pore pressure of reconstituted specimen, AR is unlikely affecting the stress paths. Likewise, excess pore pressure of intact specimen of Silt B, B2 decreased with increasing strain while reconstituted specimen, BR showed increasing excess pore pressure with increasing strain which may explain their contractive and dilative behaviour.
Chapter 5: Mechanical behaviour of intact and reconstituted calcareous silt

Considering the point at which the deviatoric stress stabilises during undrained triaxial compression as the critical state, one can define the critical state friction angle, $\phi'_{cv}$ ~ 36.8° (M~1.5) for Silt A and $\phi'_{cv}$ ~ 43.8° (M~1.8) for Silt B as shown in Figure 5-7 and Figure 5-8. Silt A has a lower $\phi'_{cv}$ compared with Silt B while reconstituted specimens of Silt A and Silt B reach the same critical state line as the corresponding intact specimens. Silt B, which has a higher calcium carbonate content, which is typical of carbonate soils, compares well with an average value $\phi'_{cv}$ = 44.8° reported by Airey & Fahey (1991) for calcareous sands and silts and a value $\phi'_{cv}$ = 40° reported by Coop (1990).
for calcareous sand. Interlocking of shell particles may contribute to higher $\phi'_{cv}$ of Silt B compared with Silt A.

## 5.4 DISCUSSION

Microscopically, two main distinctive characteristics of calcareous soils contribute to their specific behaviour: (i) the presence of intra-particle voids, and (ii) irregular shape of particles from microfossils such as coccoliths. These observations were reported in previous research (Hyodo et al. 1996; Hyodo et al. 1998; Sharma & Ismail 2006). These characteristics result from various chemical, physical, mechanical, and biological deposition processes of skeletal remains of marine organisms in deep water. Also related to these features is the susceptibility to particle breakages, which affects the compression and shear behaviour of these soils. Similar characteristics can be seen in Silt A and Silt B based on their SEM images. These silts have particles with irregular shapes and sizes, large voids between particles and internal voids within particle. However, the specimen reconstitution process changes the fabrics and arrangements of particles, as shown in the SEM images. These changes result in a more compacted arrangement and reduced void ratio. Presence of intra-void within particles, such as void in coccoliths, contributes to the void ratio calculation based on water content (Demars 1982). In Silt A and B, this calculation may not be a good indicator to analyse soil behaviour, especially undrained strengths of silts, as suggested by Lehane et al. (2014), as it does not reflect particle packing only, but also intra-particle voids.

Except for the swelling index, $C_s$, the compressibility parameters ($\sigma'_{yield}$ and $C_c$) are affected by the reconstitution process. Yield stress of the reconstituted specimens depends on the maximum stress applied to reconstitute the specimen, whereas in the case of the intact specimens, $\sigma'_{yield}$ is related to the soil’s fabric and potential of light cementation. Higher $\sigma'_{yield}$ is observed in Silt B compared to Silt A, with Silt B also having a more fine-grained and open structure, and a higher sensitivity. For Silt A, the compression index, $C_c$, of the reconstituted specimen is similar to that of the intact specimens, while for SIlt B, $C_c$ of the reconstituted specimen is only half of the values determined for the intact specimens.

From the series of undrained triaxial compression tests conducted on intact specimens of Silt A and Silt B the following pattern seems to emerge: over-consolidated
specimens ($\sigma^{\prime}\text{vc} < \sigma^{\prime}\text{yield}$) exhibit a contractive/dilative response with strain-hardening (A1, B1 and B2), whereas normally consolidated specimens show a contractive response (A2, A3, B3). The reconstituted specimens, which are normally consolidated, also exhibit a contractive response (AR, BR).

It is difficult to replicate the stress-strain response during undrained shearing of intact specimen using reconstituted specimens for both silts. The response depends on the state of the specimens before shearing, i.e., final void ratio and effective stresses, but also on the structure, which differs between intact and reconstituted specimens. For comparison between reconstituted and intact specimens, AR and BR were consolidated to the same effective stress as A3 and B2, respectively. For Silt A, AR and A3 exhibit a similar contractive behaviour. However, AR has a lower void ratio and different structure than A3, which results in a higher undrained shear strength. For Soil B, B2 and BR display a different behaviour, as B2 appears to be over-consolidated, showing contractive/dilative response and BR shows a contractive behaviour (normally consolidated), which results in a lower undrained shear strength compared to B2. However, reconstituted specimens reached the same critical state lines as the intact specimens for both silts, which may suggest some state parameters from reconstituted specimens can be used to represent the state parameters of intact specimens. It also suggests that results from physical modelling, which is generally based on reconstituted samples, may be directly applicable to intact soil in the case of processes that are governed by the critical state parameter M (or friction angle).

5.5 CONCLUSION

A series of laboratory tests and microscopic studies using SEM have been carried out to compare the mechanical properties of intact and reconstituted calcareous silty soils from two different water depths. Due to the characteristics of calcareous silts, such as susceptibility to particle breakage, large intra-void within the particles and irregular shape of particles, it remains challenging to fully understand the behaviour of these silts. Based on the preliminary results of this study, the following enhancement of our understanding on fine-grained calcareous soils can be concluded:

a. The percentage of microfossils (which relates to the percentage of calcium carbonate) in soil structure and fabrics increases with water depth, as shown in
SEM images. The increase in microfossils (highly breakable) and percentage of calcium carbonate correlates with higher soil compressibility. However, additional data is required to confirm this initial outcome.

b. Important characteristics of calcareous soils, such as large intra-voids within particles and irregular particles shapes and sizes contribute to soil compressibility and behaviour during undrained shearing (such as high friction angle).

c. Depending on the soil type and soil structure, the compression index can either be well replicated by the reconstituted specimen (soil A) or underestimated by the reconstituted specimen (soil B).

d. The type of undrained shearing response, i.e., contractive/dilative or contractive seems to be correlated with the initial state of the soil at the end of consolidation, i.e., over-consolidated or normally consolidated.

e. It is difficult to represent the behaviour in undrained shearing of intact specimens using reconstituted specimens, as soil states such as void ratio and effective consolidation stresses, as well as soil structure, affect the stress paths of the soil during shearing. However, the critical state parameters such as the slope of the critical state line (or friction angle) are well reproduced using reconstituted specimens.

Further investigations are required to establish a proper framework for fine-grained calcareous soils and refined guidance on the applicability of using reconstituted specimens to represent the behaviour of intact specimens.

5.6 REFERENCES


Sharma, SS & Ismail, MA 2006, 'Monotonic and cyclic behavior of two calcareous soils of different origins', *Journal of Geotechnical and Geoenvironmental Engineering*, vol. 132, no. 12, p. 1581.

CHAPTER 6. EXPERIMENTAL ASSESSMENT OF SAMPLING DISTURBANCE IN CALCAREOUS SILT

Abstract: One of the challenges in the study of the mechanical behaviour of natural fine-grained soils in deep water lies in the selection of representative soil specimens for laboratory testing. These soil specimens are commonly obtained using thin-walled samplers with various diameters, outer cutting angles, wall thicknesses and extra elements such as pistons either fixed or free. Soil disturbance from thin-walled sampling is unavoidable even when the best sampling techniques are used. A physical modelling program using particle image velocimetry and digital image correlation (PIV/DIC) techniques was designed to obtained a refined understanding of the disturbance experienced by the soil during sampling. This program allowed the determination of displacement and strain fields around tube samplers penetrating in calcareous soil. The natural calcareous soil used in this study was obtained from the North-West Shelf of Australia. The movements of soil particles were captured using a digital camera during penetration of a half tube sampler against a Perspex window. The images obtained were then analysed using the free image analysis program GeoPIV-RG to determine the displacement and strain fields in the soil. This study focuses on the effects of different sampler wall thicknesses and sampling penetration rates on the soil disturbance. The results indicate that sampling with a thin tube (diameter to thickness ratio, B/t = 40) leads to a relatively undisturbed zone at the center of the tube, with diameter approximately half the tube diameter. Sampling under undrained conditions with the thin tube results in lower strains in the soil near the soil-tube interface, compared with the partially drained case. When using a thicker tube (B/t = 20), significant shear strain is observed in the majority of the sampled soil.
6.1 INTRODUCTION

The demand for offshore structures has been increasing due to the positive outcomes of exploration into deeper water for oil and gas and the expanding growth of offshore renewable energy sectors, especially in the North Sea regions. The costs to build these structures are very high due to the greater risks and challenges involved. Hence, it is important to have accurate geotechnical parameters of the site to be used in geotechnical design systems such as anchoring systems, pipeline systems and even subsea shallow foundations. Site investigation in deep water is normally carried out using seabed based equipment or remotely operating vehicles (ROVs) such as PROD (Carter et al. 1999) or ROVDRILL (Edmunds et al. 2012) to retrieve soil samples. ROVs can perform in situ site investigation by pushing full flow penetrometers such as the T-bar (cylindrical) (Randolph et al. 1998) and ball (Kelleher & Randolph 2005) penetrometers or pushing tube samplers to collect soil samples. Disturbance due to tube sampling is inevitable, and most of the retrieved soil samples may not truly represent the characteristic of in-situ soils.

In onshore environments, the Sherbrooke sampler or block sampling is well known to retrieve excellent quality samples for element tests in the laboratory. However, it is not possible to use the Sherbrooke sampler to retrieve soil samples in deep water especially if soil sampling is done from a vessel. Most soil samplings are performed using a tube sampling method such as jumbo piston corer (Young et al. 2000), STACOR® (Borel et al. 2002) and even the ROV employs tube sampling techniques (Shelby tubes) to obtain soil samples for element tests in the laboratory. Sampling disturbance is an important consideration when interpreting geotechnical parameters from laboratory data. Soil models for numerical and constitutive modelling are normally calibrated using these parameters and interpretations of laboratory data obtained from disturbed specimens will lead to inaccurate design.

The typical sediments that engineers encounter offshore and need to be designed for are very soft, fine-grained materials like clay, mud and carbonate silts. Sampling disturbance of soft soils such as clay has been studied by many researchers in the past, addressing various effects such as tube geometry (i.e. diameter, cutting toe angle and tube thickness), piston presence and types of soil samplers such as Sherbrooke sampler and Osterberg sampler (Baligh et al. 1987; Lunne et al. 1997; Clayton et al. 1998b; Clayton
Most of the studies on sampling disturbance have been carried out based on element tests, or numerical modelling, with very few studies using a physical modelling approach. There is little published data on soil movements caused by sampling, other than the early work by Hvorslev (1949) on soil deformation due to various sampling techniques based on photographic observations, and the work by Baligh et al. (1987) on the ideal sampling approach (ISA), based on the prediction of centreline strain path in tube samples of saturated clay using the strain path method (SPM). Recently, Yan et al. (2010) reported results of physical modelling tests using particle image velocimetry (PIV) simulating sampling in lightly overconsolidated kaolin clay, while similar tests were conducted in transparent soil by Hover et al. (2013).

To date, no study on soil movements caused by sampling in fine-grained offshore calcareous soils has been reported and the effect of sampling rate on the deformation in this type of soil has not been investigated.

The purpose of this paper is to (i) determine the deformation field inside and outside a sampling tube penetrating in calcareous soil and identify soil zones along the tube with minimal disturbance, (ii) investigate the effects of tube thickness and penetration rate on the displacement and strain field inside and outside the tube and (iii) compare the vertical strain path determined along the tube centerline with the approximate theoretical solution derived by Baligh et al. (1987) using the strain path method. The paper presents a physical modelling program that simulates the sampling process in a calcareous soil, by pushing a half tube sampler against a transparent Perspex window, while images of soil deformations were captured by a high-speed camera. The PIV/DIC technique (White et al. 2003; Stanier et al. 2015) was then used to obtain the displacement and strain fields in the soil by analysing images with the MATLAB written program called GeoPIV-RG (Stanier et al. 2015). The testing program consisted of four tests conducted with two different tube thicknesses and two sampling rates, simulating undrained and partially drained condition. Soil characterisation was also conducted using a piezoball test and one-dimensional consolidation test.
6.2 TEST APPARATUS

A PIV box of 0.22 m x 0.39 m x 0.33 m with transparent Perspex windows as the front and back panels was used as a consolidation chamber to contain the soil. The Perspex window had dimensions 0.39 m x 0.33 m and contained equally spaced markers (black dot with white background circle), which were used to correlate measurements in pixels with field measurements in mm. The PIV box was fitted into a steel strongbox commonly used in centrifuge tests, which supported the actuator and brackets to hold the PIV camera, as shown in Figure 6-1. The tube sampling was performed by using an electrically driven actuator with two degrees of freedom, designed and manufactured at The University of Western Australia (UWA). The actuator was controlled using the in-house software (PACS) to push the half tube sampler and piezoball at required penetration rates. The half-tube sampler was fabricated by cutting a standard 50 mm diameter aluminium hollow tube into two halves and attaching a connecting bolt at the top of the half-tube. The tip of the half tube was edged to produce a cutting toe angle of 15°. The vertical edges of the half tube sampler were lined with a rubber membrane to provide a good sealant between the tube and the window with minimum friction, while the tip was lightly coated with epoxy to produce a stiff tip for penetration into the soil mass.

The camera used to capture the images was a charge-coupled device (CCD) sensor camera (Allied Vision Technologies Prosilica GC2450C) with Gigabit Ethernet connectivity for fast data transfer. A fast data transfer from the camera to the computer was necessary due to the high frame rate per second used. An in-house lighting system consisting of a pair of LED panels was set-up to ensure the light was uniform, bright and of similar colour as natural light. The camera and lighting system were controlled using two separate in-house software packages, DigiCAM and DigiLED, respectively.
6.3 SOIL TESTED AND EXPERIMENTAL PROCEDURE

6.3.1 Soil sample preparation and soil characterisation

The offshore calcareous soil used in this study was obtained from the North West Shelf of Australia with its index properties shown in Table 6-1. It contains 24 % clay, 66 % silt and 10 % fine sand. The soil was reconstituted by mixing the soil with synthetic sea water (35 g of NaCl per litre of deionised water) to produce a slurry with a water content of approximately 110 %. Synthetic sea water was used to replicate the origin of the soil. The soil was mixed for at least 48 hours to ensure uniformity and homogeneity of the mixture. Before pouring the slurry into the PIV box, the interior surface of the PIV box wall was coated with silicon grease, and the bottom surface was layered with a geotextile. The slurry was poured slowly into the box containing a layer of water to ensure no air entrapment during the process. A geotextile was layered on top of the slurry before placing the top plate. Water was allowed to drain from the top and bottom of the box. The box was placed into a consolidation frame to be consolidated in multiple stages up to the maximum consolidation pressure of 50 kPa. Due to the height limitation of the PIV box, the box was refilled three times with the same slurry to achieve the required height of consolidated soil (i.e., 280 mm). For the first two refills, the soil was consolidated up to 35 kPa and for the last refill, the soil was consolidated up to 50 kPa. A linear variable displacement transducer (LVDT) was used to measure the displacement during soil
consolidation. When the displacement was less than 1 mm per day, primary consolidation was considered completed. The soil was then unloaded to a vertical stress of 5 kPa prior to testing. The final water content of the consolidated soil mass was approximately 51%.

<table>
<thead>
<tr>
<th>Table 6-1: Index properties of the calcareous silt tested in this study</th>
</tr>
</thead>
<tbody>
<tr>
<td>Liquid limit, $w_L$ (%), 52</td>
</tr>
<tr>
<td>Plasticity index, $I_p$ (-), 22</td>
</tr>
<tr>
<td>Specific gravity, $G_s$ (-), 2.69</td>
</tr>
<tr>
<td>Initial water content of slurry, $w_{slurry}$ (%), 110</td>
</tr>
<tr>
<td>Particle size distribution (clay / silt / sand) (%), 24 / 66 / 10</td>
</tr>
<tr>
<td>Coefficient of uniformity, $C_u$ $[D_{60}/D_{10}]$, 46</td>
</tr>
<tr>
<td>Coefficient of curvature, $C_c$ $[(D_{30})^2/(D_{10}xD_{60})]$, 1.2</td>
</tr>
</tbody>
</table>

Due to the fine-grained nature of the soil used, black seeding particles were used to create the necessary surface texture for PIV test. The spreading pattern on the soil was compared and matched upon the given image texture with artificial seeding ratio of 0.5 recommended by Stanier and White (2013). The main purpose to use this ratio is to optimise the image texture for more accurate image analysis. Once the Perspex panels were placed back in position, the PIV box was clamped to the strongbox into a fixed position. The camera, actuator and LED lighting system were then set up as shown in Figure 6-1.

Figure 6-2 shows the testing configuration in plan view, with two sampling tests conducted in each PIV box. Two consolidated soil samples were prepared successively in the PIV box to perform the four samplings included in the testing program. Before pushing the half tube samplers, a mini piezoball test (piezoball diameter, $D_b$: 20 mm) was carried out at a rate of 2 mm/s to determine the undrained shear strength, $s_u$, profile of the soil samples. After the sampling simulation, the walls of the PIV box were dismantled and a cubic block of soil of approximately 100 mm was cut from the mid-height of the sample and used for a one-dimensional consolidation test using constrain rate of strain oedometer tests (CRS).
Chapter 6: Experimental assessment of sampling disturbance in calcareous silt

Figure 6-2: Plan view of mini piezoball and tube samplers’ penetration locations

Figure 6-3: (a) Undrained shear strength with depth from mini-piezoball test, (b) normalised compressibility curves

Figure 6-3 (a) shows the profiles of $s_u$, determined using piezoball resistance equations with $N_{ball}=11$ (Colreavy et al. 2015). Both soil samples have similar $s_u$ (~20 kPa) over the first 100 mm, below which the sample in Box 2 showed a slight decrease in $s_u$ (~17 kPa). Thus, it appears that the sample in Box 2 is slightly less uniform compared to the one in Box 1 and that the soil in the lower part of Box 2 may not have the same profile of density with depth as Box 1. The one-dimensional compression curves obtained from CRS tests and plotted in Figure 3 (b) indicate that the soil sample in Box 2 is slightly more compressible than the one in Box 1. Pre-consolidation stress obtained for both boxes
were similar, approximately 60 kPa. This is consistent with the lower $s_u$ observed for the soil in Box 2 and the noted differences in soil density in the lower part of Box 2.

6.3.2 Sampling simulation

The edges and interior of the tube sampler were greased with silicon oil before it was attached to the actuator. Using the actuator, the tube sampler was moved horizontally until it applied a slight pressure against the Perspex window (maximum force measured on the load cell of 0.005 kN), to seal the contact between the tube and the window. To simulate sampling, the tube sampler was moved vertically at a prescribed rate over a length of 140 mm, while the camera captured sampling images. At the end of the penetration, a resting period of five minutes was imposed to allow for pore pressure dissipation, before retrieving the tube at a similar rate. Subsequently, a second sampling test was performed on the opposite side of the PIV box (Figure 6-2). Four tube sampling simulations were carried out, and the details of tube geometry and penetration rate used are outlined in Figure 6-4 and Table 6-2.

![Figure 6-4: Definition of sampling tube geometry](image)

<table>
<thead>
<tr>
<th>Box</th>
<th>Sampling tests</th>
<th>Outside diameter, B (mm)</th>
<th>Thickness, t (mm)</th>
<th>Cutting toe angle, OCA (°)</th>
<th>Internal diameter of cutting shoe, $D_i$ (mm)</th>
<th>Area ratio, AR $(B^2-D_i^2)/D_i^2$</th>
<th>B/t</th>
<th>Penetration rate (mm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>TN_UD</td>
<td>50</td>
<td>1.25</td>
<td>15</td>
<td>47.5</td>
<td>0.11</td>
<td>40</td>
<td>2.0</td>
</tr>
<tr>
<td>1</td>
<td>TK_UD*</td>
<td>50</td>
<td>2.5</td>
<td>15</td>
<td>45</td>
<td>0.23</td>
<td>20</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>TN_PD</td>
<td>50</td>
<td>1.25</td>
<td>15</td>
<td>47.5</td>
<td>0.11</td>
<td>40</td>
<td>0.4</td>
</tr>
<tr>
<td>2</td>
<td>TK_PD</td>
<td>50</td>
<td>2.5</td>
<td>15</td>
<td>45</td>
<td>0.23</td>
<td>20</td>
<td>0.4</td>
</tr>
</tbody>
</table>

Note: TN-thin tube; TK-thick tube; PD-partially-drained and UD-undrained
The tube sampling penetration rates were selected to simulate two different drainage conditions, fully undrained and partially drained. According to previous studies, these two drainage conditions can be characterised in terms of the normalised velocity, \( V \), defined in Equation 6-1 (Finnie 1993; Finnie & Randolph 1994):

\[
V = \frac{vD_e}{c_v}
\]  

(6-1)

where \( v \) is the velocity of the penetrating object, \( D_e \) is the equivalent diameter of the penetrating object and \( c_v \) is the coefficient of consolidation of the soil. \( D_e \) is the diameter of a hypothetical cylinder, which has the same planar area as the penetrating object (Cassidy 2012). In the case of the half tube sampler of outer diameter \( B \) and thickness \( t \), \( D_e \) is defined as:

\[
D_e = \sqrt{2Bt}
\]  

(6-2)

Based on the study by Finnie & Randolph (1994), investigating T-bar penetrometer tests in silty soil, undrained conditions occur for \( V > 30 \), whereas fully drained conditions occur for \( V < 0.01 \). Additionally, the work of Schnaid et al. (2010) on piezocone penetration in tailings materials shows that a transition from drained to partially drained behaviour takes place for \( 0.1 < V < 1 \), whereas a transition from partially drained to undrained takes place for \( 10 < V < 100 \). The tailings materials in that study had consolidation properties almost similar to those of the silty soil used in the present paper. Hence, in this study, a penetration rate \( v = 2.0 \) mm/s was selected to simulate undrained condition \( (V = 40) \), whereas \( v = 0.4 \) mm/s was used for partially drained conditions \( (V = 7) \), as shown on Figure 6-5. Coefficient of consolidation used in the calculation was derived from the CRS test results.
Chapter 6: Experimental assessment of sampling disturbance in calcareous silt

6.4 RESULTS

The program GeoPIV-RG was used to analyse the images captured during the tests and determine the displacement and strain field in the soil. Only the results of three tests are presented, as test TK_UD with the thick tube did not yield acceptable results. This is due to some inappropriate sealing between the tube and the window affecting the analysed results. The effect of tube thickness can be assessed by comparing tests TN_PD and TK_PD, whereas the effect of penetration rate can be evaluated from tests TN_UD and TN_PD.

6.4.1 Evolution of soil displacements during tube sampling

Figure 6-6 shows the displacement vector field due to the sampling of the thin tube at 2 mm/s (test TN_UD) for two different penetration stages: midway penetration and final penetration. Displacement fields have been enlarged 10 times for clarity. At midway penetration, a wedge of soil moving upwards (soil heave) is visible in the top part of the tube sample. Below this wedge, the displacements appear to be negligible inside the tube, except close to the tube wall, where the soil is moving down and inward. The soil displacements are larger outside the tube compared to inside the tube, due to the shape of the cutting toe (see Figure 6-4), which pushes the soil down and radially outward. Close to the surface, the soil outside the tube is moving up and radially outward. The soil ahead of the tube sampler has negligible disturbance. However, at final penetration, small downward displacements are observed ahead of the tube sampler. At this stage, the wedge
at the top of the tube sample has fully developed to a size of approximately one tube diameter. Similarly as for midway penetration, displacements inside the tube are small except near the tube side walls. This analysis suggests that no significant disturbance occurs in the lower part of the tube sample during sampling.

a. Midway penetration

b. Final penetration

Figure 6-6: Displacement vector field during sampling with the thin tube at 2 mm/s (TN_UD): (a) midway penetration and (b) final penetration

6.4.2 Effects of tube thickness and penetration rate on induced displacement and strain fields

The displacement vector field obtained at the end of tube penetration at a rate of 0.4 mm/s (partially drained) is plotted in Figure 6-7 (a) for both thin and thick tubes. When sampling with the thick tube, the soil displacements inside the tube are larger and the zone of disturbance along the soil-tube interface is wider compared with the case of the thin tube. For the thick tube, downward displacement is observed at the centreline, whereas these are negligible for the thin tube. However, the zone of disturbance at the top of the sample has a lesser extent in the case of the thick tube. Similar displacement patterns are observed outside the tube regardless of the tube thickness, with a tendency for more radial displacement in the case of the thick tube.
Figure 6-7 (b) shows the displacement vector field at the end of penetration for the thin tube in the case of UD and PD penetration. In both cases, soil displacements inside the tube are relatively small, except at the tube side wall and in the upper part of the sample near the soil surface. Displacements along the tube inside wall are slightly smaller in the undrained case. This seems to agree with the expectation that the soil reaches a lower peak shear strength at lower strain in undrained conditions, compared to partially drained condition. The soil displacements outside the tube are larger and directed more downward in the case of partially drained penetration compared to undrained penetration.
Figure 6-7: Comparison of soil displacement field at full penetration: (a) Thin tube and thick tube at 0.4 mm/s; and (b) thin tube at 0.4 mm/s and 2.0 mm/s.

Figure 6-8 shows the effects of tube thickness and penetration rate on the shear strain field in the soil inside and outside the tube sampler at the end of penetration. For the sake of clarity, only the contours up to a shear strain of 20% are plotted. Sampling with the thick tube leads to a wider zone of shear straining. The shear zone inside the tube extends over more than half the soil sample radius from the tube sidewall in the case of the thick tube sampler, whereas it is slightly less than the half of sample radius for the case of the thin tube. In both cases, the shear zone inside the tube is wider than the one outside the tube. Sampling at higher penetration rate (2 mm/s, undrained condition) using the thin tube produced less deformation inside the tube compared to sampling at partially drained condition, as shown in Figure 6-8 (b). The maximum shear strain at the soil-tube interface is ~10% for test TN_UD, whereas it reaches over 20% for test TN_PD. Defining the ‘undisturbed’ zone as the region of soil with shear strain below 2%, only the thin tube pushed at undrained conditions (TN_UD) produces an ‘undisturbed’ sample with size approximately half the tube diameter, excluding the top part of the sample. A similar size sample of slightly lower quality is obtained when sampling with the thin tube in partially drained conditions.
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Figure 6-8: Shear strain field: (a) thin and thick tube at 0.4 mm/s (partially drained) and (b) thin tube at 0.4 mm/s (partially drained) and 2.0 mm/s (undrained)
6.4.3 Volumetric strain history during tube sampling

Figure 6-9 shows the locations of two elements, one located close to the tube sidewall (element S) and the other close to the tube centreline (element C). These elements were selected such that they would be in the middle of the tube by the end of tube penetration. The origin of the normalised depth (z = 0) coincides with the location of the sampler tip at those elements and positive volumetric strain indicates extension.

![Figure 6-9: Location of two soil elements along the tube for strains analysis](image)

For an element located at the tube centreline, the volumetric strain induced by tube penetration is relatively small, ranging between ± 4 %. For the thin tube pushed in undrained condition, no volumetric strain is recorded ahead of the tube sampler. When the soil moves inside the sampler, an irregular sinusoidal pattern is observed, showing a small peak in extension (+1%) followed by a significant peak in contraction (-4.5%) and a final extension phase (~3%). The peak in contraction at the centreline correlates with the peak in extension occurring at the tube sidewall as soil passes the tip of the sampler.
The two partially drained tests show irregular volumetric strain paths during tube penetration with a dilative response varying between 0 % and 2.5 %.

Non-negligible volumetric strain is measured in the case of undrained sampling with the thin tube, both at the tube sidewall and at the tube centerline at the level of the tube tip. This seems to indicate that some partial drainage occurs locally, although the drainage condition may be considered undrained globally, at the scale of the tube.

![Figure 6-10: Volumetric strain paths for all cases: (a) element located very close to tube sidewall and (b) element located at tube centreline](image)

**6.5 DISCUSSION**

Baligh et al. (1987) used the strain path method (Baligh (1985) to predict strains during undrained penetration of an idealised sampler in saturated incompressible clay. They showed that in the inner half of the tube sample, variations in soil strains are small and strains can be estimated by the values calculated at the sample centreline. These strains consist mainly of vertical strain, with the other components of strain being negligible.

Figure 6-11 shows the vertical strain path of an element located at the tube centreline for the three sampling tests carried out in this study and comparison with the approximate theoretical solution (B-TN) by Baligh et al. (1987). For penetration with the thin tube either undrained or partially drained, the overall shape of the vertical strain paths
resembles that of the theoretical prediction. However, it appears that partially drained penetration of thin tube is the closest to the theoretical solutions. The strain paths show an initial compression phase ahead of the tube sampler, an extension phase when the cutting edge is close to the element and a second compression phase when the cutting edge has moved passed the element. However, unlike the theoretical solution, the observed strain paths are not symmetrical, with the initial peak in compression reaching 1% and the peak in extension reaching -2% for both penetrations. Similar observations have been reported in previous studies (Clayton & Siddique 2001; Hover et al. 2013). For thin tube sampling in partially drained condition, the peak compressive strain occurred earlier at \( z/B = -0.38 \) while the peak extensive strain occurred later at \( z/B = 0.62 \) as to sampling in undrained condition. The peak compressive strain for undrained condition happened at \( z/B = -0.02 \) while the peak extensive strain at \( z/B = 0.07 \). Comparison with the theoretical solution, the normalised depths for peak compressive and extensive strains is approximately at \( z/B = \pm 0.4 \) respectively, which almost similar as partially drained penetration.

Figure 6-11 also shows that strain is more localised at the level of the sampler tip than predicted by the theoretical solution. In addition, some residual compressive strain is measured, whereas in the theoretical solution, the strain tends back to zero once the sampler’s tip has moved away from the soil element. In the case of partially drained penetration with the thick tubes, the vertical strain path is irregular with non-zero strain measured ahead of the sampler, mostly compressive strain is observed (up to 1.7%).
Another aspect to discuss here is the sampler retrieval process. Sampler retrieval plays an important role on sample disturbance. Because of the very large soil movements occurred during this stage, unrealistic results were obtained from GeoPIV-RG and therefore neither displacement fields or strain contours are presented here. Nevertheless, the digital images shown in Figure 6-12 give a qualitative insight on the mechanisms that occur during sampler retrieval using open samplers. There, three mechanisms may be recognized: (i) the progressive development of a shear plane at the sampler tip, (ii) the soil heave at the bottom of the borehole (due to the interaction between the uplift force and soil suction), and (iii) the creation of tension cracks, due to the loss of lateral support for the soil located at the bottom of the sampler, which leads to low soil recovery.

**Figure 6-11: Vertical strain path at tube centreline for all sampling tests and comparison of observed strain path for sampling with the thin tube in undrained condition with the prediction by Baligh et al. (1987)**
Figure 6-12: Effects of sample retrieval

The test results analysed above are for open tube samplers (U50). Observations of displacements and strains indicate significant soil disturbance in the upper part of the sample, in a region of size approximately equal to the tube diameter. These results
confirm, once again, the important soil disturbance caused by Shelby tubes which, in turn, leads to unreliable estimation of soil parameters from laboratory tests. Therefore, laboratory results obtained from Shelby tubes, which is by far the most common sampler used in practice due to its simple operational principle, should be taken with caution.

6.1 CONCLUSION

A physical modelling program using the particle image velocimetry (PIV) technique was designed to determine the displacement and strain fields around tube samplers penetrating offshore calcareous soil, with the aim of identifying zones of least disturbance inside the tube. Small scale sampling simulations were conducted and the results were analysed to determine the effects of tube thickness and sampling rate on soil deformation inside the tube. The analysis focused on tube penetration only. The following conclusions can be drawn.

1. The sampled soil is deformed the most in two zones, namely the top of the tube and the zone near the soil-tube interface. When sampling with the thin tube (B/t = 40), a sample with relatively small disturbance (shear strain < 2%) is recovered at the centre of the tube, with a size of approximately half the tube diameter.

2. When the tube thickness is increased (B/t = 20) significant shear strains (> 2%) are produced in the majority of the sampled soil during penetration. In addition, the volumetric strain (dilation) at the soil-tube interface is greater compared to the case of the thin tube at the same penetration rate.

3. Increasing the sampling rate of the thin tube such that sampling occurs in an undrained condition results in significantly lower strains at the soil-tube interface compared to the partially drained case. However, some dilative strain is observed near the tube sidewall during undrained sampling, indicating that some drainage probably occurs locally in the shear band. A further increase in the rate of sampling (> 2.0 mm/s) may lead to negligible volumetric strains near the tube side wall and a better quality sample for this type of soil. This could be investigated in future work.

4. The vertical strain path at the tube centreline determined for the sampling test with the thin tube in both drainage conditions shows a pattern similar to the sinusoidal response predicted by Baligh et al. (1987) with a few differences: i) the observed
strain path is asymmetrical with a higher peak in extension compared to the initial compression peak, ii) the observed strain path is more localised around the sampler tip, iii) non-zero residual compression strain is observed. Interestingly, penetration of thin tube in partially drained condition appears to be closest to the theoretical solution by Baligh et al. (1987). For the sampling test performed using thick tube under partially drained conditions, the vertical strain paths exhibit a somewhat irregular pattern, which does not resemble the theoretical solution derived for undrained conditions.

5. During sample retrieval, three mechanisms may be recognized: (i) the progressive development of a shear plane at the sampler tip, (ii) the soil heave at the bottom of the borehole (due to the interaction between the uplift force and soil suction), and (iii) the creation of tension cracks, due to the loss of lateral support for the soil located at the bottom of the sampler, which leads to low soil recovery. Hence, presence of piston is important to minimise soil disturbance during sample retrieval.

The results of this analysis are aligned with previous studies, which recommend using thin tube samplers and trimming the sample prior to laboratory testing, including disregarding the soil at the top and bottom of a sampling tube. It has been shown that varying the sampling rate has some effect on the sample quality when sampling in carbonate silty soil. Further study is required to investigate sampling with larger diameter tubes, higher sampling rates, sampler with piston, as well as sampler extraction methods.

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CHAPTER 7. PHYSICAL MODELLING OF TUBE SAMPLING IN CALCAREOUS SILTY SOILS

Abstract: Effects of tube sampling on offshore calcareous soils are investigated using a physical model to simulate tube sampling under a controlled environment. A physical model was designed to simulate field sampling operations in three different configurations (i) two tube samplings using half-tubes against a glass window to assess soil deformation using particle image velocimetry (PIV), (ii) two tube samplings using half-tubes against an aluminium wall, with embedded pore pressure transducers (PPT) aligned with the centreline of the tube, to measure excess pore pressure generation and (iii) two tube samplings using full tubes to retrieve soil specimens for element testing. Different perspectives were used to explore the effects of tube sampling on geotechnical parameters measured in the laboratory, such as effective stress assessment using pore pressure readings along the tube centreline, measurement of deformation zones from PIV analysis and measured geotechnical parameters from soil element tests. Different drainage conditions were imposed during sampling by varying the tube penetration rate. The outcomes of the physical model have shown that the soil inside the tube may experience localised failure due to high excess pore pressure development during sampling, while different penetration rates affect the shear strain developed along the interface between the inner surface of the tube and the soil. Additionally, laboratory results have shown that tube sampling in silty soil may result in densification, leading to higher values of undrained shear strength, small strain stiffness and yield stress.
7.1 INTRODUCTION

Design of foundation systems relies on soil strength parameters determined through either in situ testing or laboratory testing. It is well-known that sampling, transportation, and preparation of soil elements for laboratory testing may change stress, density, water content and soil fabric. The effects of these changes on the mechanical properties measured in the laboratory have been widely studied for clays (Lacasse et al. 1986; Santagata & Germaine 2002; Ladd & DeGroot 2003; Lunne et al. 2006; Santagata et al. 2006; Lim et al., 2018). Soil disturbance due to tube sampling can be divided in three main components as follows: (i) mechanical distortions from sampling method depending on types of tube samplers used, (ii) stress relief due to the change of total stress from removal of overburden in the field to zero total stress state in the laboratory and (iii) physical changes in the specimens during sampling activities such as transportation, storage, sample extrusion and specimen preparation (Lefebvre & Poulin 1979; Rochelle et al. 1981; Lunne et al. 1997; DeGroot et al. 2005; Santagata et al. 2006).

One common approach used to quantify the effect of sampling disturbance on measured soil properties consists of comparing experimental results obtained from tube specimens with those estimated using block specimens, which display minimum disturbance and therefore are representative of the in-situ conditions. This approach was followed by Lunne et al. (2006) who used the Sherbrooke sampler to obtain high quality block samples of soft Norwegian marine clays. For most of the clays tested, the peak undrained shear strength measured in anisotropically consolidated undrained compression (CAUC) triaxial tests was significantly higher for the undisturbed block samples than for the tube samples, with the block samples exhibiting a higher degree of softening (strength reduction after peak). The effect of sampling disturbance on the small-strain stiffness (where peak shear stress normally occurs for clays) was deemed to be due to clay structure breakdown. At large strain, the dominating effect is the reduction in water content during reconsolidation to in situ stresses.

Another approach consists of evaluating the changes in boundary conditions, resulting deformation and loading history occurring during sampling. Skempton and Sowa (1963) investigated the effects of reducing the in situ anisotropic stresses acting on the soil sample to zero, which would represent specimen states prior to laboratory testing. This “perfect sampling approach” (PSA) disregarded mechanical disturbance effects.
Clay samples subjected to PSA exhibited the same undrained shear strength as undisturbed samples, despite some differences in effective stress paths. By extending the PSA, Baligh et al. (1987) investigated mechanical disturbance effects due to tube penetration. They proposed the “ideal sampling approach” (ISA), which is based on the strain history calculated using the strain path method (Baligh 1985) at the centreline of the sampler during undrained penetration. Laboratory test results on Boston Blue clay reported by Baligh et al. (1987) and Santagata and Germaine (2002) show that tube penetration as well as shear stress relief affects the shear stress-strain response of the soil significantly, particularly by decreasing its undrained strength and eliminating strain softening characteristics.

Limited studies on the effects of sampling on the properties of offshore calcareous soils have been reported, despite their importance in practice, as most undisturbed offshore soils are sampled using tube sampling methods. The approach proposed by Lunne et al. (2006) has not been applied to offshore calcareous soils, as undisturbed block sampling cannot be carried out by offshore drillers. Additionally, most of the soils encountered in offshore environments are composed of calcareous silty materials, with a complex behaviour, which is still not well understood. So far, little attention has been paid to the effects of sampling disturbance on silty materials. The recent work by Carroll and Long (2017) on silty materials obtained using piston tube samplers and block samplers from three different sites in Ireland and Norway has shown contrasting responses between samples from each site. Soil samples from Letterkenny in Ireland and Refnevien in Norway show similar quality between specimens obtained using piston tube samplers and block samplers, whereas soil samples from Skibbereen in Ireland show densification in piston sampling compared to block sampling. Similarly, Pineda et al. (2013), Sau et al. (2014) and Arroyo et al. (2015) performed an extensive experimental program including CAT scans to evaluate sample disturbance on block samples of silty deposits obtained in the deltaic zone of the Llobregat River in Barcelona. Using CRS oedometer tests, they had shown that block sampling technique produced a very high-quality sample for element tests. Krage et al. (2005) also compared sampling disturbance criteria based on Lunne et al. (1996)’s approach on clay and found that the approach is not suitable for silty materials with low plasticity.
In order to fill this gap, this chapter aimed at investigating the effects of tube sampling on offshore calcareous soils sampled in a controlled environment. A physical model was designed to simulate field sampling operations in three different configurations (i) two tube samplings using half-tubes against a glass window to assess soil deformation using particle image velocimetry (PIV), (ii) two tube samplings using half-tubes against an aluminium wall, with embedded pore pressure transducers (PPT) aligned with the centreline of the tube, to measure excess pore pressure generation and (iii) two tube samplings using full tubes to retrieve soil specimens for element testing. The drainage conditions included: (i) undrained penetration and (ii) partially drained penetration, achieved by varying tube penetration rates.

7.2 DESIGN OF PHYSICAL MODEL

The physical model shown in Figure 7-1 consists of two major parts: the sampling system and the consolidation chamber. The sampling system includes an upper frame to provide rigidity to the actuator, a high speed custom-made linear actuator with high precision motion control, a lower frame with bottom base to support the actuator on top of the consolidation chamber and a tube sampler for sampling using U50 Shelby tubes (either full or braced half-tubes). The consolidation chamber (0.67 m x 0.52 m x 0.8 m) includes a hardened glass window (19 mm in thickness) for digital image correlation/particle image velocimetry (DIC/PIV) analysis, a top consolidation plate with multiple removable covers, which are removed to let the tube samplers penetrate into the soil, as well as six pore pressure transducers (PPT) embedded into the back panel and aligned with the centreline of the sampling tube. Drainage valves were installed at the top of the consolidation plate and the bottom of the consolidation chamber to allow water drainage at top and bottom during the consolidation stage.

One of the common problems of pushing a rigid model against a transparent window for PIV/DIC is the deflection of the model away from the wall during penetration. Particularly, this phenomenon will be more pronounced for a deep penetration such as a tube sampling of 500 mm in length. Extra bracings using three steel rods were welded at the back of the tube sampler to prevent deflection and provide additional rigidity (see Figure 1-2). The gaps between the rods were filled with epoxy and the surface was smoothed using sand paper to create a streamlined body to reduce the soil friction.
Additional volume at the back of the sampler due to the bracing may provide additional lateral stress to push the tube sampler against the window during sampling. The edges of the half-tube sampler were lined with 1 mm thick black silicon tape to ensure a watertight interface between the half-tube sampler and the window surface. U50 Shelby tubes with a sampler head were obtained from a local supplier and the sampler head was modified to fit into the actuator. The dimensions of the tube samplers used in the physical model are summarized in Table 7-1. It includes values of the area ratio (AR), the cutting toe angle (OCA) as well as the ratio B/t, where B is the outer sampler diameter and t refers to the thickness of the tube wall. Ladd and DeGroot (2003) suggested a value of B/t > 40 in order to minimize sampling disturbance. The ratio B/t in this case is equal to 34 and therefore specimens of good quality could be (a priori) expected.

Figure 7-1: Three-dimensional view of the laboratory sampling physical model
Table 7-1: Dimension of tube sampler

<table>
<thead>
<tr>
<th>Tube sampler</th>
<th>Outside diameter, B (mm)</th>
<th>Thickness, t (mm)</th>
<th>Cutting toe angle, OCA (°)</th>
<th>Internal diameter of cutting shoe, D_i (mm)</th>
<th>Area ratio, AR ((B^2-D_i^2)/D_i^2)</th>
<th>B/t</th>
</tr>
</thead>
<tbody>
<tr>
<td>U50</td>
<td>51</td>
<td>1.5</td>
<td>15</td>
<td>48</td>
<td>0.128</td>
<td>34</td>
</tr>
</tbody>
</table>

(i) Side view

(ii) Top view

(iii) Front view

Figure 7-2: Schematic diagram of the half-tube sampler design (the filled gaps with epoxy between the rods is not shown)

To prevent deflection of the window during consolidation, hardened glass of 19 mm in thickness was used instead of the traditional Perspex window. The reference markers to be placed on the window for PIV analysis were developed by printing 2 mm black dots on a transparency screen and gluing 4 mm white circular dots behind the black dots, as shown in Figure 7-3. The transparency screen was sandwiched between the interior surface of the hardened glass window and a 2 mm thick sheet of Perspex, to protect the markers and provide a scratch resistance surface during sampling. The reference markers with black and white background are necessary to correlate the measurement from pixels to mm (White et al. 2003). This method provides the flexibility to change the markers’ arrangement without damaging the hardened glass and prevent the markers from being erased during soil consolidation.
Figure 7-3: 4 mm plastic white dots to be glued on transparency screen with printed black dots for PIV analysis

One of the main walls of the physical model was fitted with six pore pressure transducers (TE Connectivity Measurement Specialties-86-015G-C) as shown in Figure 7-4. Each of the transducers has a measuring range up to 103.2 kPa which coupled with a filter made from sintered bronze. To ensure full saturation of pore pressure transducers, the filters were fitted very closely to the pressure sensor diaphragm. The pressure transducers were calibrated using a special chamber as shown in Figure 7-5 which allow the application of controlled pressure by Druck Pressure Calibrator as well as measurement of voltage by the pressure sensor. Additionally, this chamber was used to saturate the embedded pore pressure transducers before testings.
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Figure 7-4: Six embedded pore pressure transducers

Figure 7-5: Special chamber used to calibrate and saturate the pore pressure transducers

7.3 SOIL TESTED AND SAMPLE PREPARATION

Tube specimens retrieved from the North-West Shelf, Australia were collected and mixed together to create a slurry for soil reconstitution. The tube samples were recovered from water depths between 400 m up to 1500 m and soil depths between 1 m up to 4 m.
All the soil samples were extruded from the tube with the top and bottom parts removed to prevent wax contamination. Extruded samples were soaked with artificial sea water (electrical conductivity EC=50 mS/cm) to soften the soil for mixing. The soaked soil samples were transferred into a mechanical mixing chamber with artificial sea water added to form a slurry. A vacuum was applied during the mixing process for at least two days to de-air the mixture. Afterwards, the slurry was transferred into the consolidation chamber overlying a 5 mm thick geotextile. Due to the large volume of soil, the transfer of soil was done using a hose attached to the bottom of the mixing chamber, while the chamber was lifted by a forklift (see Figure 7-6) to provide a height difference of 1.2 m. The soil was slowly layered using the hose under a layer of seawater (see Figure 1-5).

Figure 7-6: Process of filling up the consolidation chamber with freshly mixed slurry.
A loading rig was installed on top of the consolidation frame to consolidate the slurry in multiple stress increments (Figure 1-6). The first stress increment was 3 kPa and subsequent stress was applied up to a maximum stress of 50 kPa, with a stress increment ratio of 2. Due to the height limitation of the consolidation chamber, the box was refilled three times with the same slurry to achieve the required final height for sampling simulation. For the first two refills, the soil was consolidated up to 35 kPa, while for the final refill, the soil was consolidated up to 50 kPa. The displacement reading from a linear variable displacement transducer (LVDT) and pore pressure readings from the pore pressure transducers were used to ensure the consolidation process of each stress increment was completed before moving to the next increment.
Surface texture on the soil against the glass window needs to be created to perform DIC/PIV analyses, due to the fine-grained nature of the soil. Once the soil had been consolidated to 50 kPa, the soil was unloaded to a zero-stress condition and the loading frame was removed. The chamber was then tilted at 45° before removing the window panel and the surface of the soil was spread with seeding particles. The spread of seeding particles on the soil followed the recommendation by Stanier and White (2013) by using the artificial seeding ratio of 0.5 to optimise the PIV analysis as shown in Figure 7-9. The soil was reconsolidated up to a maximum stress of 50 kPa using the same loading frame, which was then swapped with a compact hydraulic jack consolidometer with load cell measurement to hold the consolidation stress. The compact consolidometer illustrated in Figure 7-10 was used to allow additional space for the actuator support frame to be secured above the consolidation chamber.
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Figure 7-9: Using seeding particles to create texture on the soil surface

Figure 7-10: Compact hydraulic jack consolidometer with load cell during sampling simulation

Figure 7-11 shows the particle size distributions of five soil samples taken at different depths of the consolidated soil sample. All the particle size distributions are similar, indicating that the consolidated soil mass is uniform and homogeneous in terms of particle size distribution from the top to the bottom. This reconstituted soil comprised of more than 70% of fine-grained particles (55% silt and 15% clay) and 30% of fine sand. It had a water content (w) of 46%, liquid limit (w_L) of 58% and plasticity index
(Ip) of 20. The material can be classified as a silty sand with high plasticity based on AS 1726:2017 (Standards Australia 2017).

Additional, Figure 7-12 shows the correlated undrained shear strength profiles at two different locations obtained using T-bar penetrometer tests, which were carried out.
after sampling simulations. The undrained shear strength determined from an established correlation (Chung et al. 2006; Low et al. 2011) was based on the tip resistance load and the single bearing factor, N =10.5. Similar soil profiles were obtained from these two tests, which suggests that the soil mass was reasonably homogeneous. T-bar penetrometer tests were performed under a zero-load condition and an average $s_u$ of 23 kPa was estimated, with peak and residual values of 29 kPa and 20 kPa, respectively. The undrained shear strength reduced linearly with depth which is consistent with the state of the soil in an over consolidated state. At the start of penetration, initial soil friction developed around the penetrometer surface is normally higher before it reduces with subsequent penetration resulting in a higher resistance (higher $s_u$) at shallow depths (see Figure 7-12).

![Undrained shear strength vs depth](image.png)

**Figure 7-12:** Correlated undrained shear strength from T-bar penetrations at two different locations

### 7.4 EXPERIMENTAL PROCEDURE

#### 7.4.1 Procedures for sampling in the laboratory

The sampling procedure consisted of three stages as follows: (i) two tube samplings using half-tubes against the hardened glass window for PIV analysis, (ii) two tube
samplings using half-tubes against the aluminium wall with embedded PPT along the tube centreline and (iii) two tube samplings using full tubes close to the centre of the consolidation chamber. Two penetration rates were selected to simulate two different drainage conditions: (i) 2 mm/s to simulate undrained conditions and (ii) 0.4 mm/s to simulate partially drained conditions. The penetration rates were determined based on the normalised velocity parameter, \( V \), defined by Finnie & Randolph (1994) and Randolph et al. (1998) as:

\[
V = \frac{vD_e}{c_v}
\]  

(1)

where \( v \) is the velocity of the penetrating object, \( D_e \) is the equivalent diameter of the penetrating object and \( c_v \) is the coefficient of consolidation of the soil. \( D_e \) is the diameter of a hypothetical cylinder, which has the same planar area as the penetrating object (Cassidy 2012). For the half-tube sampler of outer diameter \( B \) and thickness \( t \), \( D_e \) is defined as:

\[
D_e = \sqrt{2Bt}
\]  

(2)

Using \( c_v = 25 \text{ m}^2/\text{year} \), a value determined based on previous CRS data on similar material, \( V = 40 \) for undrained condition while \( V = 8 \) for partially drained condition.

A high-speed camera was set up with a lighting system secured by lighting stand kits (see Figure 7-13) during the penetration of half-tubes against the transparent window. The camera was programmed to capture soil movements around the tube at a frequency rate of 13 frames per second (fps) during undrained penetration and 3 fps for partially drained penetration. The frequency rate was set to capture minimal differences of soil movements between two successive images, in order for the PIV/DIC technique to give adequate results. After setting up the actuator with the attached half-tube sampler, the corresponding cover on the top consolidation plate was removed to allow the penetration of the half-tube sampler into the soil mass. The half-tube sampler was pushed at the selected rate for a distance up to 420 mm. A resting period of five minutes was allowed at the end of penetration to simulate the field process, before retrieving the tube at the same penetration rate. At the end of the retrieval process, any soil retrieved that came as a block was placed back into the same cavity to minimise soil displacement around it and the semi-circular cover on the top plate was placed back in position. Next, the location of
the actuator with half-tube sampler was moved to the next sampling location, the same process was repeated but at a slower penetration and camera frequency rate.

For the penetration of half-tubes against the aluminium wall with embedded PPT, the actuator with the attached half-tube sampler was set-up on the back wall. A similar process was followed as the first sequence, but with excess pore pressure monitoring along the centreline of the half-tube sampler. The final part of the sampling process was carried out by pushing full tubes into the soil mass at the allocated penetration rates. Due to the limited space for the actuator support frame to push the tube at the allocated position, the soil mass was unloaded to adjust the hydraulic jack consolidometer beam to be perpendicular from its previous position. The soil mass was loaded again up to 50 kPa before placing the actuator with attached 50 mm Shelby tube into its position. The whole process of adjusting the position of hydraulic jack consolidometer took less than an hour, preventing swelling of the soil mass. The full tubes were pushed at their respective penetration rates but with no retrieval at the end of penetration. The tubes were left in the soil mass and they were manually removed when the whole system including the consolidation chamber was disassembled. After removal of the top consolidation plate, T-bar penetrometer tests were carried out to characterise the soil strength at two different locations under zero load condition (T-bar penetrometer on Section 7.3). Subsequently, all the side panels of the chamber were removed. A block of soil was cut in the undisturbed soil mass using a large soil wire saw, placed on to a rigid plate and wrapped with plastic film and aluminium foil while the tubes were manually removed from the soil mass, covered with plastic film and plastic caps. All the soil samples were stored in a humid room before performing element tests.
7.4.2 Procedure for soil element tests

Three one-dimensional constant rate of strain (CRS) compression tests and three anisotropically consolidated undrained (CK₀U) triaxial tests were carried out to evaluate soil compressibility and strength of specimens trimmed from the two tube samples and the cut block. For CRS tests, a specimen of 24 mm in height and 43 mm in diameter was loaded up to a maximum effective stress of 1500 kPa, after saturating using a back pressure of 200 kPa for at least 24 hours with 5 kPa of effective stress as seating load. The strain rate used for CRS tests was 1.85 %/hr (0.005 mm/min) to ensure the maximum pore water pressure ratio was less than 15 % in the normally consolidated range.

Triaxial tests were conducted on specimens of 100 mm in height and 50 mm in diameter. All specimens were consolidated to a vertical effective stress of 40 kPa and horizontal effective stress of 24 kPa, after saturation with a B-value of 0.98 was obtained. The vertical effective stress was lower than the applied vertical effective of consolidation stress to have the specimen in over-consolidated state at the end of consolidation. At the

Figure 7-13: Schematic diagram for camera and light set-up during half-tube sampling for PIV analysis
end of consolidation, the specimens were sheared at a constant rate of 0.007 mm/min, approximately 10 % per day. The triaxial apparatus was equipped with bender elements to determine the small strain stiffness (at strain below 0.001 %) by measuring the shear wave velocity.

7.5 RESULTS

7.5.1 Strain contours from GeoPIV_RG

Figure 7-14 illustrates the shear strain contour (maximum shear strain of 20 %) for the undrained and partially drained penetrations for the same tube geometry. It is apparent that the shear strain for the soil inside the tube is less than 8 % during undrained sampling. On the contrary, larger shear strain can be observed for soil sampled in a partially drained penetration. This observation is in agreement with the initial investigation at small-scale described in Chapter 6, where it was demonstrated that sampling at a fast penetration rate (undrained conditions) produces less deformation inside the tube compared with sampling where partial drainage is allowed. However, the deformation of soil sampled in a partially drained condition is not symmetrical with respect to the vertical axis. This due to the tip of the tube sampler was slightly bent during the tube sampling process, which may have induced a larger soil movement inside the tube. A further analysis and comparison was not performed because the outcomes will not represent the effects of different drainage conditions accurately. Nevertheless, the bent tip of the tube sampler during partially drained condition can represent a condition when a damaged tube is used to sample soil in the field. A damaged tip can increase the shear strain of soil along the tube and soil interface.
7.5.2 Pore-pressure measurement along the tube centreline during tube penetration

Only four pore pressure transducers were below the surface of the soils at the end of consolidation as illustrated in Figure 7-15. The distance from the surface to the first row of pore pressure transducers (PP2 and PP4) was approximately 110 mm while the second row of pore pressure transducers (PP1 and PP3) was approximately 200 mm away from the first row. Only three pore pressure transducers (PP1, PP2 and PP3) performed normally during penetration, as PP4 stopped working during the consolidation process. Furthermore, it was not possible to replace the faulty pore pressure transducer before the penetration stage.

Figure 7-14: Shear strain contours from DIC/PIV analysis of half tube samplings
Figure 7-15: Location of pore pressure transducers before tube sampling

Figure 7-16 and Figure 7-17 show the excess pore pressure, $\Delta u$, profiles measured by three pore pressure transducers at their respective locations for undrained and partially drained samplings. Additionally, both figures also include sampling paths with reference to the events of penetration into the soil mass, five minutes resting period at the end of penetration and retrieval of half-tube sampler. In undrained penetration, PP2 and PP1 measured the $\Delta u$ of soil inside the tube, while in partially drained penetration, PP3 measured $\Delta u$ of soil inside the tube. At the start of tube penetration, a very small soil suction was measured regardless of drainage conditions. This small suction may be due to the water seeping out from the porous stone and flowing downward with the sliding soil.

During undrained penetration, PP2 and PP1 measured maximum $\Delta u$ values of 55 and 48 kPa, respectively. A drop of 5 kPa was measured by PP2 at about 325 mm penetration, which may suggest a leakage between the edges of the tube sampler and the wall (due to loss of perfect contact) at this particular location, while a gradual increase of $\Delta u$ was measured by PP1. At the end of penetration, $\Delta u$ measured by PP2 dropped drastically to 35 kPa in a few minutes, in contrast to $\Delta u$ measured by PP1, which continued to increase until stabilisation. This increase in $\Delta u$ during the resting period may be due to water flow from the tube/soil interface, where higher shear strain have
developed, towards the tube centreline (this flow may not have occurred for PP2 due to some leakage between the edges of the tube and the wall). During the retrieval process, a strong suction measured by PP2 and PP1 can be observed; PP2 measured a maximum suction of 30 kPa, whereas PP1 measured a suction of 10 kPa. Suction measured by PP1 reduced gradually compared to suction measured by PP2, which fell steeply midway during the retrieval process.

The results for partially drained penetration are shown in Figure 7-17. During partially drained penetration, the $\Delta u$ for soil inside the tube (measured by PP3) developed gradually without any unusual response until the end of penetration. The response was similar to the response measured by PP1 during undrained penetration, except that a larger maximum $\Delta u$ of 57 kPa (compared to 48 kPa in the undrained case) was measured at the end of penetration. This result might seem surprising, however, it may be explained by the larger shear strain measured during partially drained penetration. A gradual drop of 7 kPa was recorded during the five minutes resting period. During retrieval, a maximum suction of 30 kPa was measured and it continued to decrease with subsequent retrieval.

![Figure 7-16: Excess pore pressure profiles for undrained penetration](image-url)
Figure 7-17: Excess pore pressure profiles for partially drained penetration

7.5.3 Compressibility characteristics

Table 7-2 shows CRS specimen properties including the change of void ratio at different experimental stages. The initial void ratios for all specimens based on the initial water content are similar, with the UD specimen having the lowest void ratio followed by PD specimen and block specimen. Figure 7-18 shows the compressibility curves from CRS tests performed on specimens trimmed from two tube specimens sampled in either undrained (UD) or partially drained (PD) condition, as well as a block specimen. During saturation with a back pressure of 200 kPa and effective vertical stress of 5 kPa as seating pressure, it appeared that both tube specimens, UD and PD underwent large deformation, with a larger deformation observed for UD (shown by the dashed lines).

Table 7-2: CRS experimental details and results

<table>
<thead>
<tr>
<th>Specimens</th>
<th>UD</th>
<th>PD</th>
<th>BLOCK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial bulk density, $\rho_{\text{bulk}}$ (g/cm$^3$)</td>
<td>1.79</td>
<td>1.78</td>
<td>1.77</td>
</tr>
<tr>
<td>Initial dry density, $\rho_{\text{dry}}$ (g/cm$^3$)</td>
<td>1.23</td>
<td>1.22</td>
<td>1.21</td>
</tr>
<tr>
<td>Initial void ratio, $e_0$ (-)</td>
<td>1.24</td>
<td>1.25</td>
<td>1.28</td>
</tr>
<tr>
<td>Void ratio after saturation, $e_s$ (-)</td>
<td>1.05</td>
<td>1.12</td>
<td>1.25</td>
</tr>
<tr>
<td>Void ratio at in-situ stress, $e_{\text{in-situ}}$ (-)</td>
<td>1.04</td>
<td>1.11</td>
<td>1.24</td>
</tr>
<tr>
<td>Sample quality assessment, $\Delta e/e_0$</td>
<td>0.16</td>
<td>0.11</td>
<td>0.02</td>
</tr>
<tr>
<td>Initial water content, $w_{\text{initial}}$ (%)</td>
<td>44.1</td>
<td>44.4</td>
<td>46.6</td>
</tr>
<tr>
<td>Final water content, $w_{\text{final}}$ (%)</td>
<td>32.0</td>
<td>32.2</td>
<td>39.0</td>
</tr>
</tbody>
</table>
Yield stress, $\sigma'_\text{yield}$ (kPa) & 140 & 90 & 50  
Swelling index, $C_s$ (-) & 0.02 & 0.02 & 0.02

Both UD and PD specimens showed a sudden change in inclination around the yield stress, compared to the block specimen, which showed a gradual decrease of void ratio with stress increase. One interesting observation is the yield stress of UD specimen (140 kPa) and PD specimen (90 kPa) appeared to be higher than the consolidation stress applied to the soil mass, which was approximately 50 kPa, while the yield stress for the block specimen was approximately 50 kPa, similar as the consolidation stress. The sample quality for all the specimens was assessed using Lunne et al.'s (1997) approach by using the change in void ratio parameter, $\Delta e/e_0$ during saturation and consolidation as shown in Table 7-2. The block specimen shows very good to excellent quality, while tube specimens, UD and PD rank as poor quality. The initial oedometric modulus reduces from 23 MPa (UD specimen) to 4 MPa (block specimen), showing a similar trend as the yield stress as shown in Figure 7-19. Even though the oedometric modulus can be used to assess sampling disturbance, this assessment may not be applicable, as disturbed specimens were subjected to higher densification during saturation.

Figure 7-18: Compressibility curves from CRS tests: (a) Void ratio against vertical effective stress and (b) Volumetric strain against vertical effective stress.
Chapter 7: Physical modelling of tube sampling in calcareous silty soils

Figure 7-19: Oedometric modulus versus effective vertical stress

Figure 7-20 illustrates the variation of the compression index, \( C_c = \frac{\partial e}{\partial \log(\sigma'_v)} \), with the vertical effective stress normalized by the appropriate \( \sigma'_\text{yield} \) for each specimen, in the normal compression range only. Both tube specimens show the same response of a rapidly rising \( C_c \) to \( \sim 0.25 \) after passing the yield stress, whereas \( C_c \) for the block specimen increases gradually. However, all specimens reached almost similar compression indices at higher effective vertical stress.

Figure 7-20: Variation of compression indices with normalised effective stress level
Figure 7-21 compares the excess pore pressure responses during consolidation, derived hydraulic conductivity and coefficient of consolidation for all specimens. For vertical effective stress less than 300 kPa, obvious differences of consolidation responses can be seen, while for vertical effective stress above 300 kPa it appears that all specimens behaved similarly. The more densified specimen of UD developed the highest excess pore pressure as it has the lowest hydraulic conductivity compared to PD and block specimens.
Figure 7-21: CRS compression results: (a) $\Delta u$ response during consolidation; (b) derived hydraulic conductivity and (c) $c_v$ with effective vertical stress

7.5.4 Stress-strain behaviour in CK0C tests

Table 7-3 shows properties of triaxial specimens including the change of void ratios at different experimental stages. The initial void ratio based on the initial water content for all specimens have slight differences between them but only the tube specimens underwent further reduction of void ratio during saturation. Sample quality based on Lunne et al.’s (1997) approach shows that all specimens appeared to be good to excellent quality with better quality for the block specimen.

Table 7-3: Triaxial experimental details

<table>
<thead>
<tr>
<th>Specimen</th>
<th>UD</th>
<th>PD</th>
<th>BLOCK</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial bulk density, $\rho_{\text{bulk}}$ (g/cm$^3$):</td>
<td>1.79</td>
<td>1.77</td>
<td>1.78</td>
</tr>
<tr>
<td>Initial dry density, $\rho_{\text{dry}}$ (g/cm$^3$):</td>
<td>1.26</td>
<td>1.24</td>
<td>1.22</td>
</tr>
<tr>
<td>Initial void ratio, $e_0$ (-)</td>
<td>1.21</td>
<td>1.24</td>
<td>1.26</td>
</tr>
<tr>
<td>Void ratio after saturation, $e_s$ (-)</td>
<td>1.19</td>
<td>1.23</td>
<td>1.26</td>
</tr>
<tr>
<td>Void ratio after consolidation, $e_{ac}$ (-)</td>
<td>1.17</td>
<td>1.20</td>
<td>1.23</td>
</tr>
<tr>
<td>Sample quality assessment, $\Delta e/e_0$ (-)</td>
<td>0.037</td>
<td>0.035</td>
<td>0.028</td>
</tr>
<tr>
<td>Initial water content, $w_{\text{initial}}$ (%)</td>
<td>44.6</td>
<td>44.9</td>
<td>47.5</td>
</tr>
<tr>
<td>Final water content, $w_{\text{final}}$ (%)</td>
<td>42.1</td>
<td>41.9</td>
<td>46.2</td>
</tr>
</tbody>
</table>
Figure 7-22 shows the stress-strain curves and evolution of excess pore pressure during undrained compression for both tubes and block specimens. The stress-strain curve for the UD specimen shows a continuous increase in strength corresponding to strain hardening behaviour. The PD specimen also shows an increase in strength up to an axial strain of ~10%, followed by moderate strain softening behaviour. The Block specimen achieved peak strength at very low axial strain and thereafter the deviator stress remained approximately constant with continued shearing. Figure 1-20 (b) shows that all specimens initially built up excess pore pressure indicating a contractant behaviour, which was followed by a dilatant response in the case of the tube specimens only (decrease in $\Delta u$).

The block specimen had a lower $s_u$ (~50% lower) compared to both tube specimens. The peak $s_u$ of approximately 16 kPa measured for the block specimens is in relatively good agreement with the average $s_u$ of 22 kPa correlated from T-bar penetrometer tests, whereas the tube specimens exhibit higher $s_u$ values. Comparing stress-strain curves for both tube specimens, the specimens sampled under undrained and partially drained conditions showed similar response during undrained compression up to 10% axial strain.

The stress path diagram in Figure 7-23 shows that both tube specimens were dilative, whereas the block specimen was contractive during undrained compression. The effective stress paths for all tests terminate on a unique critical state line, with a slope $M = 1.72$, corresponding to a friction angle $\varphi = 42^\circ$. Therefore the sampling disturbance and drainage condition during tube sampling has minimal effects on the critical state line of the tested soil.
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Figure 7-22: (a) Stress-strain curves and (b) Excess pore pressure response during undrained compression

Figure 7-23: Stress paths for all specimens during shearing
Chapter 7: Physical modelling of tube sampling in calcareous silty soils

Figure 7-24: Stiffness decay curves for specimens during undrained shearing

Figure 7-24 illustrates the stiffness decay curves, $G_{u-sec} = Δq/3Δε_s$, obtained from specimens sheared under triaxial compression and values of the small-strain shear modulus obtained using bender elements. Both tube specimens have slightly higher stiffness compared to block specimens. The values measured in BE is always higher than the values calculated using stress-strain response of triaxial tests, which is to be expected because the BE measurements were made at an axial strain of only 0.001%.

7.5.5 Shear stiffness using bender elements

The small strain shear modulus, $G_{max}$, deduced from the shear wave velocity, $V_s$, measured using bender elements ($G_{max} = \rho V_s^2$, where $\rho$ is the total mass density of the specimen) was determined at five different intervals: after setting up the specimen on the base just before saturation, at the end of consolidation just before shearing and at three different strain intervals during shearing. The values of shear modulus after setting up on the base just before saturation, reported in Table 7-4, indicate that shear modulus for both tube specimens are similar to each other, but they are slightly lower than the block specimen. However, after consolidation, tube specimens appear to be stiffer than the block specimen, which is consistent with their higher density. During shearing, all specimens showed a consistent reduction of shear modulus at the same strain intervals.
Table 7-4: Small strain shear modulus at different stages

<table>
<thead>
<tr>
<th>Stage/Specimens</th>
<th>Shear Modulus, G (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>UD</td>
</tr>
<tr>
<td>After setting up and before saturation</td>
<td>3.1</td>
</tr>
<tr>
<td>At the end of consolidation</td>
<td>24.4</td>
</tr>
<tr>
<td>After 0.01 mm shearing ($\varepsilon_a = 0.01%$)</td>
<td>24.1</td>
</tr>
<tr>
<td>After 0.1 mm shearing ($\varepsilon_a = 0.1%$)</td>
<td>22.3</td>
</tr>
<tr>
<td>After 1.0 mm shearing ($\varepsilon_a = 1%$)</td>
<td>18.7</td>
</tr>
</tbody>
</table>

### 7.6 DISCUSSION

In this study, the mechanical disturbance due to tube sampling was investigated through establishing the strain field inside a tube sample using PIV analysis, measuring the excess pore pressure along the tube centreline during tube sampling and evaluating the mechanical properties of sampled soils through CRS compression tests and triaxial tests with bender elements. Based on PIV results, larger shear strains in the soil inside the tube were observed for a slower penetration rate (partially drained conditions) compared to a higher penetration rate (undrained conditions). This observation is in agreement with previous work presented in Chapter 6 and with the earlier study by Hvorslev (1949) who suggested the best sample quality can be obtained when the tube sampler is driven into the soil mass at a very fast pace.

Drainage conditions influence the excess pore pressure generated during tube sampling and affect the change of effective stress in the specimen. During the experiment, a change of excess pore pressure was observed during both penetration (positive excess pore pressure) and retrieval (suction) of the tube sampler. Based on the limited data available, it was observed that higher pore pressure was generated during sampling in the partially drained case, compared to the undrained case. This may be explained by the higher shear strain developing during partially drained sampling. Indeed, if the soil is contractant (as shown by the triaxial response of the block specimen), further shearing may induce higher excess pore pressure. It was also observed that contrary to partially drained sampling, during undrained sampling the maximum pore pressure was not reached at the end of penetration but after some resting period. This may be due to some pore pressure redistribution from the zone near the soil/tube interface towards the tube.
centreline. This redistribution would have occurred during tube penetration in the case of the partially drained case. During retrieval of a tube specimen, suction of a similar magnitude to the excess pore pressure was generated, which may induce a rearrangement of the soil structure and water redistribution in the specimen. This study suggests that measurement of stresses inside the tube would be valuable, to be combined with pore pressure measurements, in order to determine the effective stress in the soil and assess whether the soil reaches some “meta-stable” state or “near liquefaction” state at very low effective stress.

The effects of tube sampling on soil mechanical properties measured in the laboratory have been analysed by comparing the results of tests carried out on tube specimens and block specimen. All specimens show similar values of compression index, $C_c$, in the normally consolidated range, however, significant increase in yield stress was measured for the tubes specimens compared with the block specimen, with the tube sampled in undrained conditions having the highest value of yield stress. These responses may be due to the process of retrieving the tube samples in the laboratory. Due to the large suction measured during the retrieval of the half tubes, the process of retrieving the full tube specimens was done manually, instead of mechanically using the actuator, to reduce the risks of specimens remaining stuck in the soil mass. The tube specimens were removed a few hours later by disassembling the chamber, cutting the soil blocks into smaller blocks and sealing the tube. The excess pore pressure generated during the penetration may have caused a ‘meta-stable’ state of soil, which is more prone to collapse during subsequent loading or saturation. This collapse may be the cause of the significant reduction in void ratio observed during saturation of the CRS specimens. Although the triaxial tube specimens showed a less drastic change in void ratio during saturation, they are still denser than the block specimen, which leads to a significant overestimation of the soil undrained strength and overestimation of the small strain stiffness.

This study has found that tube sampling induces densification in the sampled soil, which leads to different responses observed in element tests between block and tube specimens. These laboratory results seem to be consistent with other research, which found that the effects of sampling disturbance on silt material is not the same as for clay (Long 2006; Long et al. 2010). However, the limited data presented in this study do not allow to deduce strong outcomes on the effects of penetration rates on sample quality.
7.7 CONCLUSIONS

A physical model was designed and manufactured in order to (i) measure strains developed during sampling using PIV/DIC analysis, (ii) evaluate effective stresses generated during sampling by measurement of excess pore pressures along the tube centreline and (iii) compare geotechnical parameters derived from element tests carried out on tube and block specimens. Due to the time required for developing the physical model and for consolidating the soil sample, only one set of simulations was performed using this physical model. The effects of drainage conditions during tube sampling was studied in the experimental campaign by varying the penetration rate. From this set of simulations, the following conclusions can be drawn:

a. Based on PIV/DIC analyses, the soil inside the tube specimen experienced a higher level of disturbance (i.e., larger shear strain) when sampled under partially drained conditions compared to under undrained conditions.

b. During tube sampling, the excess pore pressure developed during tube sampling may be greater than the total stress of the specimen. This may induce a state of local failure or a metastable state for soil inside the tube specimen. This state causes de-structruction of the soil fabric during subsequent saturation.

c. This study has shown that tube sampling of silt material may result in densification, leading to higher values of undrained shear strength, small strain stiffness and yield stress. This outcome agrees well with previous studies on silt material subjected to sampling disturbance.

Further simulations need to be done using the same physical model to establish the effects of sampling disturbance in silt materials, using different dimensions of tube samplers and different consolidation stresses. All these data will allow proper correlation between sampler geometry, penetration rates, strains, effective stress state during sampling and geotechnical parameters obtained from element tests on material extracted from the tubes or blocks.
7.8 REFERENCES


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Lim, GT, Pineda, JA, Boukpeti, N & Carraro, JAH, Fourie, A 2018, “Effects of sampling disturbance in geotechnical designs”. In preparation


CHAPTER 8. CONCLUDING REMARKS

8.1 CONCLUSIONS – MAIN FINDINGS

This thesis has presented a comprehensive experimental study of the effects of sampling disturbance in fine-grained materials, with particular emphasis on a natural soft clay and an offshore calcareous silty soil. Two approaches were considered: i) determination of soil mechanical properties from laboratory tests on specimens obtained with different samplers, and ii) physical modelling of the sampling process using the digital image correlation technique to determine displacements and strains in the soil. Several aspects were considered in the study such as the presence of natural heterogeneities, the type of samplers used to retrieve tube specimens, and tube penetration rates (i.e. drainage condition during tube penetration). In addition, the consequences of sampling disturbance in geotechnical design were analysed and discussed through selected design examples such as predicting the behaviour of an embankment and shallow foundation on soft Ballina clay.

The study includes laboratory tests such as one-dimensional compression tests, triaxial tests, simple shear tests with cell pressure confinement and small strain stiffness measurement using bender elements and a resonant column apparatus. Additionally, two sets of physical models were developed, one small-scale and one large-scale, to analyse the strains that develop in the soil during tube sampling via the use of digital image correlation. In addition, the pore pressure response during sampling was examined in the large-scale model as well as by retrieving soil samples for element tests.

Only the main conclusions are presented in this final chapter as specific conclusions from previous chapters have been discussed and presented throughout the thesis.

8.1.1 Determination of soil mechanical properties from laboratory tests on specimens obtained with different samplers

Tube specimens of soft estuarine clay containing natural heterogeneities proved to be challenging to test in the laboratory. Not only the distribution of the heterogeneities within the tube specimens affected the measured properties, but their particle sizes and shapes also contributed to the variability of test results. Due to the limited extent of this
particular aspect of the research, no clear conclusion could be drawn on the effect of sampler geometry (varying outer cutting angle) on the determination of soil properties for specimens that contain variable amounts of shells. This is due to the fact that the effects of natural heterogeneities and the effects of sampler geometry could not be isolated and may cancel each other out; this particular study did not allow independent variation of these factors. However, a trend of increasing sampling disturbance, using the sample quality index $\Delta e/e_0$ proposed by Lunne et al. (1997), with increasing volumetric shell fraction was observed.

A sampling campaign had been carried out at the Ballina field testing facility using a wide variety of tube samplers (Shelby, free-piston and fixed-piston samplers) and a Sherbrooke sampler (block specimen which provides the highest specimen quality in soft soils) to evaluate the effects of sampling disturbance in soft clay. The results of the laboratory testing programme performed on the recovered soft clay samples led to the following conclusions: undrained shear strength, shear stiffness, pre-consolidation pressure and soil compressibility all decreased with increasing sample disturbance. Conversely, the strain at peak deviatoric stress increased with increasing sample disturbance. These outcomes are in accordance with observations reported previously for other natural soft clays. The laboratory results from disturbed soil samples affected the interpretation of geotechnical parameters for engineering designs such as an embankment or footing structure. Predictions of total settlement as well as pore pressure response underneath embankments is strongly affected by the interpretation of geotechnical parameters from laboratory tests using samples that may be subjected to sampling disturbance. These predictions can be assured to be unbiased as the methods used to predict embankment behaviour had been compared with a real embankment prediction exercise. Analyses of the performance of footings are, relatively speaking, less likely to be affected by sampling disturbance effects.

8.1.2 Physical modelling of the sampling process using the digital image correlation technique to determine displacements and strains in the silts

Offshore calcareous silts have characteristics such as large intra-voids between particles, irregular particle shapes and a strong potential for particle breakage during soil compression. Depending on water depths, offshore calcareous silts have different
Chapter 8: Concluding remarks

microstructures due to different geological deposition environments. The percentage of microfossils found in such specimens increased with increasing water depths. These characteristics contribute to the peculiar characteristics of offshore calcareous silts (such as their high friction angle) when they are subjected to sampling disturbance.

Analysis of strains determined from the physical model shows that offshore silts in the tube deformed mainly in two zones, namely the top of the tube and the zone near to the soil-tube interface. These deformation zones are similar to what had been observed in clay samples. Using a thicker tube during sampling of offshore silts induced a larger shear strain along the perimeter of the tube. By creating undrained conditions during tube sampling using a higher penetration rate, it appears that the shear strain developed inside the soil is less compared to the partially drained condition. Less disturbance and soil deformation are observed in undrained conditions, resulting in a better-quality tube specimen, as observed in the small-scale physical modelling. However, based on the element tests using soil samples from large-scale physical modelling, sampling disturbance in calcareous silts causes soil densification resulting in stiffer specimens. The densification process increases the undrained shear strength, the small strain stiffness, and the yield stress.

8.1.3 Recommendation for characterisation of fine-grained soils related to sampling method, laboratory testing procedure and determination of engineering properties.

Tube sampling using U50 Shelby tubes is still a very common practice in retrieving intact soil samples, especially in Australia. Based on this study, it is recommended to avoid using U50 Shelby tubes, even though they are cheap and easy to operate. If the use of tube sampling is necessary to minimise costs for site investigation, it is recommended that the Osterberg piston tube sampler be used instead of the U50 Shelby tube or even the U75 Shelby tube. Tube sampling should be carried out under undrained conditions to reduce the possible shear strain development between the sampled soil and the tube interface. This means that when sampling silty materials, the minimum sampling rate required to ensure undrained conditions would need to be determined. Additionally, during a sampling campaign, it is recommended that the shear wave velocity be measured somehow (e.g. using a seismic dilatometer, SDMT) to
estimate the small-strain stiffness of the soil *in-situ*, so that a comparison can be made with measurements performed on soil specimens tested in the laboratory.

Before performing element tests on retrieved tube specimens, it is strongly advisable to perform an X-ray scan to determine the presence of natural heterogeneities such as shell fragments, as well as fissures and cracks in the specimens. This step allows the selection of a soil specimen section that has lesser disturbance due to tube sampling. Even though shear wave velocities can be used to assess sampling disturbance, the assessment needs to be done cautiously as the presence of natural heterogeneities may affect the measurement of shear wave velocity. An assessment based on Lunne et al. (1997)’s approach can be used to evaluate the degree of sampling disturbance (or sample quality) in soils that are prone to de-structuration such as clays. However, this approach may not apply to silty materials due to their tendency to densify when subjected to sampling disturbance.

8.2 FUTURE RESEARCH

This study has developed a physical model that is able to simulate the tube sampling process in a well-controlled environment, as well as an option to establish preferred boundary conditions during the simulation. This physical model can provide insights into the excess pore pressure development along the centerline during tube sampling, shear strain development between the soil and tube interface using digital image correlation techniques and mechanical properties measured from elements tests using retrieved tube samples in the same soil mass. Due to the time constraints in developing the physical model, additional tests to establish correlations between sampler geometry, shear strains and the development of excess pore pressure could not be performed. Hence, further physical modelling work will have to be conducted to fully understand the tube sampling process in fine-grained soils using these three different insights. The following recommendations and topics are proposed for future studies on sampling disturbance in fine-grained soils using the physical model.

1. A range of penetration rates should be used to establish the effects of drainage conditions on the shear strain developed along the interface between the sampled soil and tube and how it affects the measured strength and stiffness.
2. Different tube diameters, outer cutting angles and thicknesses of tube should be used to determine the effects of using different tube geometries and dimensions in offshore calcareous silts.

3. Effects of tube sampling on the excess pore pressure development along the centerline of a tube sampler in normally consolidated soil and over-consolidated soil should be studied, as the change of effective stress affects the soil strength and stiffness.

4. Further development of the physical model to include bender elements to measure the small strain stiffness at the end of consolidation and how the stiffness varies after tube sampling simulation would be extremely beneficial.