2018

A study on interaction between pile and prefabricated vertical drain in soft clay

Dongli Zhu
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UNIVERSITY OF WOLLONGONG

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Recommended Citation

A STUDY ON INTERACTION BETWEEN PILE AND PREFABRICATED VERTICAL DRAIN IN SOFT CLAY

A thesis submitted in fulfilment of the requirements for the award of the degree

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University of Wollongong

By

Dongli Zhu

Department of Civil, Mining and Environmental Engineering 2018
I, Dongli Zhu, declare that this thesis, submitted in fulfilment of the requirements for the award of Doctor of Philosophy, in school of Civil, Mining & Environmental Engineering, Faculty of Engineering and Information Sciences, University of Wollongong, is wholly my own work unless otherwise referenced or acknowledged. The document has not been submitted for qualification at any other academic institution.

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Dongli Zhu

3rd July 2018
ABSTRACT

Pile foundation and prefabricated vertical drain (PVD) are two well established techniques used by geotechnical practitioners when dealing with soft compressive foundation materials. Driven stakes or piles were adopted since man first attempted to build secure dwellings near streams and rivers, as mentioned by Fleming et. al., (2008). Indraratna et. al., (2015) pointed out that PVDs has been adopted by the industrial practice to form radial drainage in low permeability soil for a fast consolidation. Even though both pile foundation and pre-consolidation with PVDs are widely adopted as geotechnical solutions in many projects, limited attempts of combined use of PVDs and piles were reported in the literature (Bradshaw and Baxter, 2006; Holtz and Boman, 1974; and Tefera et. al., 2011). The potential benefits of combining two ground improvement methods were not studied sufficiently by researchers and engineers.

In this research, a full-scale field test which compares the foundation soil and pile behaviour, before and after pile driving, under two circular embankments was carried out. Both embankments were built on soft compressible clays and PVDs were installed under one embankment. Two embankments were both left for consolidation before one pile was installed at centre of each embankment. The porewater pressure, lateral soil movement, surface settlement and strain of pile shaft were monitored. Pile capacity was tested immediately and 3 hours after pile installation. The monitoring and testing results indicated that pre-consolidating the clay layer with PVDs before piling can effectively reduce excess porewater pressure, lateral soil movement and downdrag due to pile soil interaction

The generation and dissipation of porewater pressure due to pile installation was studied in the laboratory, an empirical equation was developed to predict the generated porewater pressure considering different initial states of soil. The laboratory test results
indicated that the inclusion of PVDs facilitated dissipation of excess porewater pressure after pile installation.

A parametric study was carried out using numerical modelling software package PLAXIS. The model was validated using measurements obtained during large-scale field testing. After validation of the numerical model, influence of soft soil thickness, consolidating time prior to pile installation and effect of vertical drain were studied. An index which can be used to measure the efficiency in reducing negative skin friction is defined and used to compare efficiency of various consolidating arrangement prior to pile installation.
ACKNOWLEDGEMENT

I would like to express my sincere gratitude to my supervisor Prof. Buddhima Indraratna for his support and guidance. Without his effort, I would not have the privilege to carry out the full-size field test at the National Field Test Facility, Ballina, NSW. He has provided guidance and encouragement throughout my research and made my research project a mostly enjoyable one. I would also like to thank my co-supervisors: Prof. Harry Poulos and A/Prof. Cholachat Rujikiatkamjorn for all the help they provided, technical and non-technical, during my candidature. I considered myself extremely lucky that three great supervisors had led me to the destination with the least detours.

I also wish to acknowledge the contributions made by technical officers during my filed and laboratory experiments. The lab manager Alan Grant, technical officer Ritchie McLean and Duncan Best and others helped me overcome many issues during the experiments. In addition, I would like to thank Dr. Mojitaba, Dr. Rui and Dr. Pei for their help during my field and laboratory test program. I would like to extend my sincere thanks to Golder Associate for the flexible work arrangement and financial support it provided to me, so that I can study full time.

Last but certainly not least, I would like to thank my family members for the sacrifice they made to help me finish my research. My parents Zhiqiang Zhou and Daohua Zhu and my wife Lan Cao supported me selflessly and I am deeply indebted to them. Finally, thank you, my daughter, Charlotte Zitong Zhu, you have been a source of my courage, inspiration and happiness.
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<table>
<thead>
<tr>
<th>Symbol</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>A</td>
<td>Area of a unit cell around a vertical drain</td>
</tr>
<tr>
<td>$A_s$</td>
<td>Area of smear zone in a unit cell around a vertical drain</td>
</tr>
<tr>
<td>a</td>
<td>Radius of cavity</td>
</tr>
<tr>
<td>$a_0$</td>
<td>Immediate settlement on a typical consolidation settlement vs time curve</td>
</tr>
<tr>
<td>$a_{50}$</td>
<td>50% primary consolidation settlement on a typical consolidation settlement vs time curve</td>
</tr>
<tr>
<td>$a_{90}$</td>
<td>90% primary consolidation settlement on a typical consolidation settlement vs time curve</td>
</tr>
<tr>
<td>$a_{100}$</td>
<td>100% primary consolidation settlement on a typical consolidation settlement vs time curve</td>
</tr>
<tr>
<td>$a_s$</td>
<td>Settlement at the start of a typical consolidation settlement vs time curve</td>
</tr>
<tr>
<td>$a_f$</td>
<td>Settlement at the end of primary consolidation stage on a typical consolidation settlement vs time curve</td>
</tr>
<tr>
<td>$A_f$</td>
<td>Pore water pressure coefficient $A$ at failure</td>
</tr>
<tr>
<td>b</td>
<td>Radius of media which has a cavity inside</td>
</tr>
<tr>
<td>$c_c$</td>
<td>Virgin compression index</td>
</tr>
<tr>
<td>$c_r$</td>
<td>Recompression index</td>
</tr>
<tr>
<td>$c'$</td>
<td>Effective cohesion</td>
</tr>
<tr>
<td>$c_v$</td>
<td>Coefficient of consolidation</td>
</tr>
<tr>
<td>$c_\alpha$</td>
<td>Secondary compression index</td>
</tr>
<tr>
<td>$C_k$</td>
<td>Permeability index</td>
</tr>
<tr>
<td>$d_b$</td>
<td>Diameter of pile base</td>
</tr>
<tr>
<td>$d_e$</td>
<td>Effective diameter of the drain</td>
</tr>
<tr>
<td>$d_w$</td>
<td>Equivalent diameter of the drain</td>
</tr>
<tr>
<td>$e_0$</td>
<td>Initial void ratio</td>
</tr>
<tr>
<td>$e_1$</td>
<td>Void ratio of clay under 1 kPa consolidation stress</td>
</tr>
<tr>
<td>$e_n$</td>
<td>Void ratio of the clay before piling</td>
</tr>
<tr>
<td>$E'$</td>
<td>Effective elastic modulus</td>
</tr>
<tr>
<td>$F$</td>
<td>Safety factor</td>
</tr>
<tr>
<td>$G$</td>
<td>Shear modulus of the soil</td>
</tr>
<tr>
<td>$G_L$</td>
<td>Shaft soil shear modulus at pile base level</td>
</tr>
<tr>
<td>H</td>
<td>Thickness of consolidation layer</td>
</tr>
<tr>
<td>$H_d$</td>
<td>Drainage length</td>
</tr>
<tr>
<td>$I_r$</td>
<td>Rigidity index</td>
</tr>
<tr>
<td>$I_i$</td>
<td>Bessel function of first kind of $i^{th}$ order</td>
</tr>
<tr>
<td>$k$</td>
<td>Earth pressure coefficient</td>
</tr>
<tr>
<td>$k_0$</td>
<td>Earth pressure coefficient at rest</td>
</tr>
<tr>
<td>$k_a$</td>
<td>Active earth pressure coefficient</td>
</tr>
<tr>
<td>$k_h$</td>
<td>Horizontal permeability of the soil</td>
</tr>
<tr>
<td>$k_{h,\text{ring}}$</td>
<td>Equivalent horizontal permeability of soil in axisymmetric model</td>
</tr>
<tr>
<td>$\bar{k}_{hs}$</td>
<td>Average horizontal permeability of soil in smear zone</td>
</tr>
<tr>
<td>$k_p$</td>
<td>Passive earth pressure coefficient</td>
</tr>
<tr>
<td>$k_v$</td>
<td>Vertical permeability of the soil</td>
</tr>
</tbody>
</table>
L: Length of pile  
LL: Liquid limit  
MC: Moisture content  
\( m_v \): Volume compressibility of the soil  
\( p \): Pressure applied to the inner surface of the cavity  
\( p_0 \): Uniform outside pressure  
\( P_t \): Vertical load acting on pile head  
PL: Plastic limit  
Q: Surcharge at ground surface  
\( q_b \): Pile base resistance  
\( q_n \): Negative skin friction  
\( q_w \): Discharge capacity  
r: Radius of pile  
R: Radius of plastic zone in cavity expansion problem  
S: Spacing between two adjacent vertical drains  
\( s_u \): Undrained shear strength of the soil  
\( t_0 \): Time correspond to \( a_0 \)  
\( t_{50} \): Time correspond to \( a_{50} \)  
\( t_{90} \): Time correspond to \( a_{90} \)  
T: Time factor  
\( T_h \): Time factor regarding horizontal consolidation  
\( T_v \): Time factor regarding vertical consolidation  
\( T_r \): Time factor regarding radial consolidation  
\( U \): Degree of consolidation  
\( U_r \): Degree of consolidation by radial flow only  
\( U_v \): Degree of consolidation by vertical flow only  
\( w_t \): Pile head settlement  
\( Y_i \): Bessel function of second kind of \( i^{th} \) Order.  
z: Depth of soil  
\( \alpha \): Ratio between vertical drain spacing and effective diameter  
\( \alpha_u \): Coefficient to calculate pore water pressure during piling  
\( \beta \): Ratio between \( \alpha \) and vertical drain effective diameter  
\( \beta_p \): Pile-soil adhesion factor in terms of effective stress  
\( \gamma_w \): Unit weight of the water  
\( \delta \): Angle of friction between soil and pile  
\( \Delta e \): Incremental void ratio  
\( \Delta u_m \): Maximum excess pore water pressure at pile surface  
\( \Delta \sigma \): Incremental stress  
\( \eta \): Group efficiency  
\( \theta \): Ratio between area of smear zone and area of unit cell  
\( \Theta \): Ratio between horizontal permeability of soil in smear zone and outside smear zone  
\( \kappa \): Cam Clay compression index
κ* Modified Cam Clay compression index
λ Cam Clay recompression index
λ* Modified Cam Clay recompression index
ξ Shaft stress softening factor
ρ Settlement of pile
σᵢ Initial pressure
σᵢ Pre-consolidation pressure
σᵢ Final pressure
σᵢ Vertical stress
σᵢ Effective vertical stress
σᵢ Effective overburden stress
τₛ Shaft resistance
漕 Effective friction angle
θᵢ Cylinder function of ith order.
CHAPTER 1. INTRODUCTION

1.1 General
Soft soils are often encountered in Australia, particularly in the coastal areas. Their low strength and high compressibility are considered by engineers to be problematic as foundation material. This issue of soft soil foundations is more pronounced as our society develops, because civil projects such as infrastructure, residential and commercial towers, and industrial compounds, apply much higher superstructure loads on the foundation. Another example is high speed rail which is more sensitive to differential settlement than normal railway lines.

Soft soil foundations have two major problems; the foundation material does not have enough strength, so it is likely to induce bearing capacity failure, unstable slopes, and higher active pressure on the retaining structures, etc. Furthermore, as Indraratna et al. (1992) stated, the high compressibility of soft clay can lead to excessive settlement and/or differential settlement under service loads, and therefore compromise the integrity and serviceability of superstructure. In addition, most soft soil has low permeability, which means that long term post-construction deformation can be significant.

For the reasons mentioned above, the engineering property of soft soils must be improved before most permanent structure can be founded onto them, which is why engineers introduce ground improvement processes. Ground improvement is not a new concept; it has been applied in most successful land-based projects since ancient times. It can be as simple as compacting the surface with a hammer or as complicated as using multiple techniques to improve foundation material
below the ground surface. Although there are several definitions of ground improvement, the one proposed by Mitchell & Jardine (2002) is relatively concise: "Ground improvement is the controlled alteration of the state, nature or mass behaviour of ground materials in order to achieve an intended satisfactory response to existing or projected environmental and engineering actions." According to their definition, the pile foundation and prefabricated vertical drain (PVD) assisted consolidation explored in this thesis are ground improvement techniques.

Pile foundation and PVD assisted consolidation are two prevalent techniques for treating soft soil as foundation material; a pile foundation adopts piles as a rigid inclusion which "bridges over" the soft soil and transfers the load from the superstructure directly onto the bearing layer, such as bedrock. The PVD assisted consolidation method is used to improve the soft soil itself to achieve the desired engineering properties.

This chapter explains the concept and application of a pile foundation under a vertical load and PVD assisted consolidation. Various alternatives and/or improvements to pile foundations and PVD consolidation are included, as well as attempts to combine pile foundations and PVD, albeit very few cases are presented in the literature.

1.2 Pile foundation
The method of installing piles into soft ground has been used for thousands of years. Adding stronger reinforcement into weak ground is probably the most intuitive way of strengthening natural foundation soil. As stated by Bowles (1996), piles are like structure members used in superstructures, but instead of bridging
the load between slabs, piles bridge the load between the superstructure and the bearing foundation layer. Although the concept is ancient, it has never been obsolete, and it is still the most frequently used technique when natural ground is not suitable for foundations. Das (2010) pointed out that piles can be used for more than just carrying the structural load, they can also resist horizontal loads such as retaining structures, as well as countering the uplift load in offshore engineering, etc. However, in this thesis it refers to vertically downward loaded piles.

A pile is essentially a tool that redistributes the load into the foundation in a desired fashion; as Figure 1.1 shows, there are two types of piles in terms of load transfer mechanism, end bearing piles and floating piles. Although most piles transfer load through both the shaft and the tip, there is a difference in the percentage of load transferred. End bearing piles generate most of their resisting force at the tips which sit on a relatively strong, less compressive layer such as rock or dense sand. Floating piles, however, do not extend to “bearing layer” and therefore tip resistance is only a part of the load capacity. Both types of piles have their “strength and weakness;” end bearing piles generally have a higher geotechnical capacity and settle less than floating piles. On the other hand, end bearing piles normally cost more than floating piles, especially when “strong” material that piles sit on lays deep under the ground.
As Fleming et al. (2008) suggested, piles are also classified by how they are installed, that is, displacement piles and non-displacement piles. Displacement piles are driven into ground, so the surrounding soil is pushed aside and compressed, hence the term “displacement”. Another common method is to excavate a hole in the ground and then pour concrete into the hole to form a pile. In this method the natural ground material is retrieved, not pushed away, and therefore they are called non-displacement piles. Figure 1.2 shows how
displacement and non-displacement piles are installed. This thesis will only
discuss displacement piles.

1.2.1 Negative skin friction, drag down load and downdrag

As discussed before, a portion of pile foundations' geotechnical capacity come
from shaft resistance. When the foundation is loaded, the piles mobilise an
upward force on the shafts due to the downward movement relative to the
surrounding soil. This force constitutes most of the geotechnical capacity for
floating piles. However, in some cases, the surrounding soils move downward
relative to the piles, so the piles are also pulled down by the surrounding soil. In
these cases, the friction acting on the shaft is called negative skin friction.
Following the terminology used by Fellenius (1984), negative skin friction
accumulates into a force called drag down load or dragload, so the process

Figure 1.2 Installation of displacement and non-displacement piles
whereby settling soils “drag” piles downward is called downdrag. Obviously, negative skin friction will cause settlement and reduce the total geotechnical capacity of piles. In extreme cases the downdrag forces are high enough for the pile to fail structurally, as reported by Johannessen and Bjerrum (1965), Bjerrum et. al. (1969) and Bozozuk (1972). Figure 1.3 shows the generation of positive and negative skin friction.

1.2.2 The installation of displacement piles

As mentioned above, displacement piles push the soil away from its original location and also compresses it laterally, consequently, changes the engineering properties of the surrounding soil mass including but not limited to: stress, void
ratio, and other aspects. One significant change is an increase in the strength of the soil mass over a certain period, in most displacement cases. It is widely accepted across the industry that this phenomenon is caused by the generation and dissipation of pore water pressure in the soil mass, which will be discussed in a later chapter. Other changes such as ground heaving and lateral soil movement are also commonly observed and have their engineering significance.

1.3 Consolidation
Consolidation was first studied by Karl Terzaghi. Terzaghi et al., (1996) pointed out that consolidation is a process whereby a saturated soil is loaded and then the layer is compressed, and excess pore water drains out. From the description above, it is easy to understand that one important assumption of classic consolidation theory is that voids in the soil body are assumed to be filled with liquid, water in most cases. Conceptually, if there is no way for the pore fluid to escape, then the sample is in a “undrained” condition. Zero volumetric strain of sample is expected under undrained condition. On the other hand, if pore fluid is allowed to flow through drainage paths and dissipates, volume of sample will decrease. Once the pore water pressure in a sample reaches equilibrium, the sample is said to be drained. The volume change due to consolidation is always equal to the volume of water that flows out of a sample. In this thesis, unless specified, all soils under consolidation are saturated and the pore water fluid is water. A sketch of the consolidation process is shown in Figure 1.4.
Figure 1.4 Concept of consolidation

To geotechnical practitioners, two aspects of consolidation are of particular interest; settlement/deformation of the ground due to consolidation and the rate of consolidation with regards to time. As described by Craig (2004), consolidation settlement has three parts. The first part is immediate settlement that occurs straight after a load been applied. In this stage the soil is under undrained conditions, so no pore water is expelled from the soil body. The applied load is fully carried by pore water, which means the excess pore water pressure equals the increase in total pressure. Immediate settlement is elastic and can be
calculated with elastic theory. Compared to the next two stages, the magnitude of immediate settlement is insignificant and is often ignored in engineering practice. The second part is primary consolidation settlement which commences when pore water starts to seep out of the soil body. As a result, excess pore water pressure starts to dissipate and stress on the soil skeleton gradually increases. Primary settlement is considered to have both elastic and plastic component, and the volume change of the soil body is equal to the volume of pore water expelled. Primary settlement could take years to complete in cohesive soil with low permeability, and the final vertical strain can be quite significant (more than 20%). Although many decades have passed since Terzaghi first proposed his consolidation theory, there is still no universal agreement on when secondary settlement, the third stage of settlement, actually begins. The most widely accepted theory in practice is that secondary consolidation starts after 90% of excess pore water pressure has dissipated. Although researchers such as Robinson (2003), have proposed other theories about the beginning of secondary consolidation, secondary consolidation in this thesis is assumed to start at $t_{90}$, when 90% of consolidation is complete. Figure 1.5 presents a typical consolidation settlement versus log time plot, in which settlement is divided into the three parts, as mentioned above. Here, secondary consolidation settlement is linear on the log time plot and the slope is defined as the coefficient of secondary consolidation $c_z$. 
Highly compressible soft clay is always a challenge to geotechnical engineers. Without treatment, the infrastructures founded on thick soft clays, could settle more than 1 metre in the long term, which is generally way beyond the tolerance of post construction settlement. Furthermore, due to ununiform clay thickness and load distribution, the resulting differential settlement will potentially lead to severe serviceability problems such as cracking in superstructures, instable slopes, or even progressive failure of the whole foundation. Apart from its high compressibility, the low strength of soft clay also demands treatment to serve as suitable foundation material. It is no surprise that without improvement, some natural soft grounds are too weak to even support construction plants.

It is well established that pre-consolidation is an effective way of treating soft soil. Consolidation decreases the void ratio which reduces the potential of soil compression, and according to effective stress theory, the strength of soil increases as the pore water drains out. In engineering practice, some of the pre-
consolidation surcharge is often removed after designed consolidation period and the foundation soil becomes over consolidated, which generally has better engineering properties compared with normally consolidated soil. This is why pre-loading with a surcharge is a widely used method to improve soft ground.

1.3.1 Vertical drain assisted consolidation

Although soft clay can be treated with pre-consolidation, there is one critical issue: the extended period of time before consolidation is finished, due to the slow rate of consolidation. The rate of consolidation depends primarily on the permeability of the soil and the length of the drainage path. As stated before, soft clays generally have low permeability. The thicker the soft clay layer is, the longer the drainage length, consequently the slower the consolidation rate. As an example, primary consolidation of a 10 m thick soft clay layer with permeability of $1 \times 10^{-9}$ m/s can take years, or even decades to finish. Consolidation can, however, be facilitated by increasing the permeability and/or shortening the drainage path; this is the principal of the vertical drain technique. As shown in Figure 1.6, when consolidation takes place in nature soil, the pore water at the base of the soil layer needs to travel upwards to the top, whereas consolidating with vertical drains allows the pore water to travel vertically and horizontally. It is obvious that the horizontal drainage paths are much shorter than the vertical ones, and as Mitchel (1956) suggested, the horizontal permeability for most sediments is generally larger than vertical permeability. Therefore, the rate of consolidation can be accelerated significantly by using vertical drains.
Sand drains were first adopted as vertical drains in early attempts to reduce consolidation time. Sand drains are installed by excavating a borehole and fill the borehole with poorly graded sand. With relatively high permeability, the backfilled sand columns form vertical drainage inside the soil layer. Although this technique was proven useful, it was soon replaced by wick drains. Compared to the cost of borehole drilling and backfilling, installing wick drains, which are made of cupboard paper, is much cheaper. Later on, wick drains are replaced by modern PVDs, which has a plastic core wrapped with geosynthetic filter. PVDs come with different cross-sectional geometry to cater for different projects; PVDs are also easy to install with PVDs rigs, as shown in Figure 1.7.
1.3.2 Theory of PVD assisted consolidation

1D consolidation theory is applicable to most cases where only the vertical dissipation of pore water is considered, whereas the consolidation with vertical sand drains and PVD includes vertical and horizontal drainage of pore water. The general 3D consolidation theory developed by Biot (1941) can be used to solve most consolidation problems, but it is too complicated for most projects in the geotechnical industry. Instead, the radial consolidation theory developed by Barron (1948a) for drain wells and the simplified solution provided by Hansbo (1979) is widely adopted in projects which involve PVD assisted consolidation. In ground improvement practice, PVDs are normally used in conjunction with surcharge preloading. Surcharge preloading is easy to apply in most cases. However, to ensure the stability of the preloading embankment, stage
construction methodology has to be adopted. In most cases, multiple thin lifts of embankment are constructed with sufficient time interval allowed between the constructions of two lifts. In some cases, the long construction/consolidation time induced by stage construction and other issues associated with surcharge preloading justifies the adoption of vacuum preloading. In the recent decade, vacuum preloading has been increasingly applied in Asia and Australia. Unlike surcharge preloading, vacuum preloading creates negative pressure inside PVDs and “suck” the pore water out of soil mass. During this process, soil move towards PVDs and no outwards lateral displacement occurs. Consequently, stability of the foundation is not a problem. In addition, the vacuum pressure can propagate deeper than embankment surcharge which makes vacuum preloading more suitable for deep soft improvement. Furthermore, vacuum preloading can and normally are used in conjunction with surcharge preloading to optimize efficiency.

1.4 Combined use of ground improvement techniques

It is often desirable to adopt more than one ground improvement techniques. As an example, a combination of soil reinforcement (geosynthetics), pre-consolidation, and rigid inclusion is often used to form Mechanically Stabilised Embankment (MSE), as shown in Figure 1.8.
In MSE, soil reinforcement increases the stability of the embankment and form a load transfer platform which minimize the differential settlement. Rigid inclusion increases the foundation bear capacity by transfer loads to stronger soil layers and reduce embankment settlement by increase the stiffness of foundation. PVDs facilitate the consolidation, which reduces the post construction settlement and increase the soil strength, by shortening the drainage length. These three techniques are used together to overcome their individual shortcomings and form a better solution for geotechnical problems.

Other than MSE, PVDs have been used in conjunction with pile foundations to reduce the effect of pile installation on adjacent sensitive structures. Other type of rigid inclusion, such as jet mixing columns, has been used in conjunction with
PVDs to facilitate consolidation and improve strength and stiffness of soil. Details of these combined techniques will be introduced in Chapter 2.

1.5 Objective of current study
There is no current systematic study on the benefit of combining pile foundations and PVDs. Therefore, this research investigated several potential positive outcomes of this combination of techniques and provide relatively simple tools for geotechnical engineer to analyse the behaviour soil and piles during and after piling. The following aspects were focused on:

1. To reduce the negative skin friction/downdrag on pile foundations due to the settlement of soil.
2. To reduce the excess pore water pressure and lateral movement of soil due to piling.

1.6 Structure of dissertation
Following Chapter 1, a comprehensive literature review on relevant topics is presented in Chapter 2. This chapter has three sections. Literatures regarding PVD assisted consolidation are discussed in the first section; this includes factors such as the smear effect, patterns of PVD installation, and the method of preloading. The second section presents literature about pile foundations with a focus on negative skin friction and the effects of installation; this section also reviews studies on the combined use of PVD and pile foundations.

Chapter 3 includes details of the proposed large-scale field test to study the combined use of piles and PVD. The test setup and test procedure are described. In the Laboratory model tests, factors such as CSR, frequency, clay strength,
effective confining pressure, and drainage condition were studied. The
development of pore pressure and stress redistribution were monitored.
Chapter 4 introduces the design, procedure, and results of laboratory model tests
to gain some understanding of the pore water pressure generated in clay due to piling.
Chapter 5 provides a detailed consolidation theory of unit cell including a pile and surrounding PVDs. Smear effect will be included, and the results of field and lab tests will be used to verify the proposed theory. The results of numerical simulation are presented in this chapter as well.
Chapter 6 concludes the research outcomes and provide recommendations for future work.
CHAPTER 2. LITERATURE REVIEW

In this chapter, an extension of the introductory material presented in Chapter 1 is provided via a comprehensive literature study on the following topics: 1. Consolidation theories, including the effect of smear zone, well resistance, varying soil properties, vacuum preloading, etc. 2. Existing theories used to predict the behaviour of pile foundation and surrounding soil, with a focus on the behaviour of soil during piling and negative skin friction caused by settling soil. 3. Latest developments in the combined use of different ground improvement techniques, both theory and practice.

2.1 Consolidation theories
Existing consolidation theories are either analytical or numerical; an analytical solution can be further separated into 1D and 3D cases. The most frequently used theory is the 1D consolidation theory first proposed by Terzaghi et al., (1996) for simple geometry and load conditions, after which more complicated solutions for various loading schemes were developed. Despite its popularity, in cases where vertical drainage exists, 1D consolidation theory will not solve the problem by itself, which is why the radial consolidation theories developed by Barron (1948a) and Hansbo (1979) are often used for analysis. For more complicated problems, numerical methods can be used.

2.1.1 One dimensional consolidation theory

For a geotechnical engineer, the deformation and strength of foundation is always of concern. When foundations must be constructed on soft soil, then without treatment, excessive defamnation and a foundation with a low bearing capacity is
expected. The pre-consolidation of soft soil has proven to be an effective way of treating foundation soil. 1D consolidation theory is a basic and yet effective tool to predict the deformation and strength of soil after pre-consolidation as well as the time needed to finish this process.

Terzaghi et al., (1996) points out that consolidation is a process whereby saturated soil is loaded, and the layer is compressed, and excess pore water drains out. Although there are three components of consolidation settlement, for most projects, only primary consolidation settlement is of interest. In addition, the horizontal scale is much larger compared to the vertical scale, Terzaghi’s one dimensional consolidation theory which can predict primary consolidation settlement and the rate of settlement is the most widely used theory in engineering practice.

There are several essential assumptions that must be known when using Terzaghi’s (1943) theory:

- Soils are homogeneous and fully saturated, and pore fluid is incompressible;
- Pore water dissipation is one-dimensional and only in a vertical direction;
- It is a small strain theory so any changes in geometry caused by compression in the soil is insignificant.
- Pore fluid is Darcian fluid;
- There is a linear relationship between the void ratio and log scale of effective stress that is independent of time and stress history;
- The coefficient of soil permeability does not change during the consolidation process; and
Only primary consolidation is of concern.

The rate of consolidation, according to Terzaghi’s theory, is expressed in terms of the average degree of consolidation (U), which is expressed as:

\[ U = 1 - \sum_{m=1}^{\infty} \frac{2}{M^2} e^{-M^2 T_v} \]  
Equation 2-1

Where,

\[ M = \frac{\pi (2m - 1)}{2} \]  
Equation 2-2

\[ T_v = \frac{c_v t}{H^2} \]  
Equation 2-3

c_v is the coefficient of consolidation, which can be calculated from

\[ c_v = \frac{k_v}{m_v \times \gamma_w} \]  
Equation 2-4

Where

k_v is the vertical permeability of the soil

m_v is the volume compressibility of the soil

\( \gamma_w \) is the unit weight of the water

m_v can be obtained from 1D consolidation (oedometer) test. m_v is stress dependant, and can be calculated as

\[ m_v = \frac{\Delta e}{(1 + e_0) \times \Delta \sigma} \]  
Equation 2-5

An example of evaluating m_v from oedometer test results is shown in Figure 2.1.
As an alternative, $c_v$ can also be determined from oedometer test results using the root time and log time methods. As Figure 2.2 and Figure 2.3 shows, the time to achieve 50% and 90% degrees of consolidation is estimated from settlement vs log time and root time plot respectively. Then the corresponding time factor $T_v$ is calculated based on Equation 2-1 and $c_v$ is obtained from Equation 2-3.
Figure 2.2 Log time method to determine $c_v$ (after Craig, 2004)
Figure 2.3 Root time method to determine $c_v$ (after Craig, 2004)

As suggested by Craig (2004), Equation 2-1 can be represented by the much simpler empirical equations listed below

\[
\text{for } U < 0.6, \quad T_v = \frac{\pi}{4} \times U^2 \quad \text{Equation 2-6}
\]

\[
\text{for } U > 0.6, \quad T_v = -0.933 \times \log(1 - U) - 0.085 \quad \text{Equation 2-7}
\]

Although there are three components of consolidation settlement, only primary consolidation settlement is crucial in most projects. 1D primary consolidation
settlement is calculated from the ultimate primary settlement and degree of consolidation. Equation 2-8 is generally used to calculate the ultimate primary consolidation settlement $S_c$.

$$S_c = \frac{H \times \left( c_c \times \log\left(\frac{\sigma_f}{\sigma_p}\right) + c_r \times \log\left(\frac{\sigma_p}{\sigma_i}\right)\right)}{(1 + e_0)}$$  
Equation 2-8

Where

- $H$ is the thickness of consolidation layer
- $c_c$ is the virgin compression index
- $c_r$ is the recompression index
- $\sigma_i$ is the initial pressure
- $\sigma_p$ is the pre-consolidation pressure
- $\sigma_f$ is the final pressure
- $e_0$ is the initial void ration

Using Equation 2-6 to Equation 2-8, primary consolidation settlement at any time after loading can be calculated.

Another well accepted approach for ultimate settlement estimation was first introduced by Asaoka (1978). Asaoka method has been frequently adopted in daily engineering activities due to its simplicity and reasonable accuracy. First, the $n^{th}$ settlement reading is plotted against $(n-1)^{th}$ settlement reading. Then, a linear section close to the end of the data set is found and extend to intersect a line, on which $n^{th}$ settlement reading is equal to $(n-1)^{th}$ settlement reading. The point of intersection is predicted ultimate settlement.
2.1.2 Improvement of Terzaghi's one-dimensional consolidation theory

Since Terzaghi’s solution, many researchers developed more general one-dimensional consolidation theory by relaxing some of the assumptions in Terzaghi’s theory. Singh (2005) provided solutions for initial pore pressure distributions that are triangular; the consolidation curve for different triangles is shown in Figure 2.4.

![Image of consolidation curves]

Figure 2.4 Degree of consolidation vs time factor for different triangle loading (after Craig, 2004)

Some of the assumptions of 1D consolidation were also looked at by researchers. Morris (2005) relaxed the original small strain assumption and developed analytical solutions to one dimensional consolidation by assuming a finite strain. Vaziri and Christian (1994) studied the assumption of fully saturated material and proposed a solution which allows for slightly unsaturated ground conditions.
2.1.3 Three-dimensional consolidation theory

Due to the low permeability of soft clay, when thick layers are encountered, as stated by Terzaghi et al., (1996), consolidation may last for years. To accelerate the process, vertical drains may be installed. Vertical drains are very effective in reducing the consolidation time, especially in stratified layers of thick clay where the horizontal permeability is greater than the vertical permeability. In these cases, 1 D consolidation theory will not solve the problem single handed.

Although vertical drains have been widely adopted, predicting the performance of PVD assisted consolidation is still the most challenging problem in geotechnical engineering. Currently, there are several analytical and numerical tools are available for engineers to predict soil behaviour under radial consolidation. Analytical solutions with reasonable assumptions can consider the smear affect, well resistance, and change of permeability with effective stress and multi-layer consolidation. When dealing with more complicated conditions, numerical methods such as the finite difference and finite element techniques may be preferable.

In the category of analytical methods, the first three dimensional consolidation theory was proposed by Biot (1941). It is a comprehensive solution to 3D consolidation problem, but compared to radial consolidation theory, it is less attractive to geotechnical engineers because it is more difficult to apply in real projects. The fundamental radial consolidation theory was initially proposed by Carrillo (1942), who demonstrated that the excess pore water pressure and degree of consolidation can be obtained by solving the compression by vertical...
flow and radial flow separately and then combining them together, as shown in Equation 2-9

\[ 1 - U = (1 - U_r)(1 - U_v) \]

Equation 2-9

Where

- \( U_r \) is degree of consolidation by radial flow only
- \( U_v \) is degree of consolidation by vertical flow only.

Zhu and Yin (2001) compared this result with their rigorous solutions and indicated that Carrillo’s equation over predicts the degree of consolidation by less than 10%.

Barron (1948) introduced the first conventional procedure for predicting radial consolidation where the governing equation of combined consolidation is given in cylindrical coordinates as shown below:

\[
\frac{\partial u}{\partial t} = c_v \left( \frac{\partial^2 u}{\partial z^2} \right) + c_h \left( \frac{\partial^2 u}{\partial r^2} + \frac{1}{r} \frac{\partial u}{\partial r} \right)
\]

Equation 2-10

To solve this equation Barron (1948) proposed two extreme boundary conditions: “free strain” and “equal strain”, where "equal strain" assumes that the load is applied by a rigid foundation and hence surface settlement is uniform across the loaded area and any horizontal surface within the soil body remains horizontal. A "free strain" condition implies that a load is applied by a flexible foundation, hence the surface stress is uniform under foundation and differential settlement occurs in soil body. Barron (1948) provided a rigorous solution for "free strain" conditions and an approximate solution for "equal strain" conditions when he found that both conditions give very close results. Based on the "equal strain" condition, he also included the smear zone effect and well resistance.
In Equation 2-10, $c_h$ is the horizontal coefficient of consolidation of the soil, and according to Lunne et. al. (1997), $c_h$ is normally determined from in-situ dissipation tests. The most widely accepted method to determine $c_h$ is the graphic method proposed by Robertson et. al., (1992).

Torstensson (1975) proposed an analytical solution based on cylindrical cavity expansion theory, and which can be used to calculate $c_h$. Houlsby and Teh (1988) proposed a solution which incorporated large strain finite element analysis and the strain path method. Time factor is calculated as equation 2-11

$$T^* = \frac{c_h \times t}{r^2 \times \sqrt{I_r}}$$

Equation 2-11

Where

$r$ is radius of the cone, typically 25.7 mm

$I_r = \frac{G}{s_u}$ is rigidity index

$G$ is the shear modulus of the soil

$s_u$ is the undrained shear strength of the soil

It is generally accepted that without test information, the horizontal permeability of clay can be estimated from the vertical permeability. Rixner (1986) suggested that the relationship between horizontal and vertical permeability generally ranges from 1 to 5 (as shown in Figure 2.5), and in some case can be as high as 15.
Figure 2.5 Typical ratio between horizontal and vertical permeability for various clays (after Rixner, 1986)

Barron’s solution includes the Bessel function terms which are not easy for engineers to handle, so to provide a simplified procedure, Hansbo (1981) assumed equilibrium between the volume change of a soil annulus and the pore water flowing through it; based on this assumption, he obtained the most widely used solution for pore water pressure in the industry as:

\[
\bar{u} = \Delta \sigma \left( \frac{-8T_h}{\mu} \right)
\]

Equation 2-12

Where

\[
\mu = \ln \left( \frac{n}{s} \right) + \left( \frac{k_h}{k_s} \right) \ln(s) - 0.75 + \pi z (2l - z) \left( \frac{k_h}{q_w} \right)
\]

Equation 2-13

\[
T_h = \frac{c_h t}{4d_e z^2}
\]

Equation 2-14

\[
n = \frac{d_e}{d_w}
\]

Equation 2-15

\[
s = \frac{d_s}{d_w}
\]

Equation 2-16

\(d_s\) is the equivalent diameter of the smear zone
$d_e$ is the effective diameter of the drain (diameter of a unit cell)

d_w is the equivalent diameter of the drain

Chai et al. (2001) took a different approach to derive their consolidation theory with vertical drain. They considered the soil with a vertical drain as a new material with an increased average vertical permeability. This vertical permeability was derived based on an equal average degree of consolidation under a one-dimensional condition. They proposed an approximation to Terzaghi's one dimensional consolidation theory.

$$U_v = 1 - e^{-C_d T_v}$$  \hspace{1cm} \text{Equation 2-17}

They considered $C_d = 3.54$ as the best value compared to Terzaghi's theory and the expression of equivalent vertical permeability is

$$k_{ve} = k_v + \frac{8L^2}{C_d \mu D^2} k_h$$  \hspace{1cm} \text{Equation 2-18}

Where

$\mu$ is the same as that proposed by Hansbo, 1981.

Other researchers such as Lei et. al., (2015) provided analytical solution for consolidation with PVDs under multi-ramp loading.

2.1.4 Factors that affect the performance of PVD

Many factors affect the performance of PVD, such as the property of the PVD itself, the process of installation and even the consolidation deformation of soil mass. It is intuitive to think that the properties of the PVD have a direct influence on consolidation. As stated in section 2.1.3, to consider the radial drainage, it is necessary to know $d_w$, the equivalent diameter of the drain. There are two general types of cross sections of PVDs, but in most instances a rectangular
cross section is used due to its cost efficiency. Several methods have been developed by researchers such as Hansbo (1981), Atkinson and Eldred (1981), and Fellenius and Castonguay (1985), to convert a rectangular shape into an equivalent circular shape. Indraratna and Redana (2000) and Welker et al., (2000) concluded there is little difference in their equations; some of which are listed below in Table 2.1.

Table 2.1 Some existing methods used to calculate the equivalent drain diameter

<table>
<thead>
<tr>
<th>Author</th>
<th>Equations</th>
<th>Equation</th>
</tr>
</thead>
<tbody>
<tr>
<td>Hansbo (1981)</td>
<td>[d_w = 2 \times \frac{(a + b)}{\pi}]</td>
<td>Equation 2-19</td>
</tr>
<tr>
<td>Atkinson and Eldred (1981)</td>
<td>[d_w = \frac{(a + b)}{2}]</td>
<td>Equation 2-20</td>
</tr>
<tr>
<td>Fellenius and Castonguay</td>
<td>[d_w = \left(\frac{4ab}{\pi}\right)^{0.5}]</td>
<td>Equation 2-21</td>
</tr>
</tbody>
</table>

In recent decades as the trend for applying vacuum preloading PVD consolidation has increased, PVD with a circular cross section are used more frequently. For circular PVDs the diameter of the drain is equivalent to the drain diameter \(d_w\); in this research circular cross section PVDs are used.

For PVD to maintain its discharge capacity throughout the design life, a filter coat is needed to prevent soil particles from entering the drain while allowing the pore water to flow through the vertical drain. It is apparent that the filter coat must be more permeable than the soil which needs to be pre-consolidated. In general, the filter coat must have a minimum of twice the permeability of soil. In addition, to
effectively stop soil particles moving through filter coat, the apparent opening size (AOS) must be less than 90 μm. Christopher and Holtz suggested that to avoid clogging, the AOS must be equal to or larger than 3 times D15 which is the diameter of the clay particles corresponding to 15% passing through.

The discharge capacity $q_w$ is another property of PVDs which is crucial to their functionality. Since 1980 many researchers have studied the discharge capacity of PVDs and the specified $q_w$ value for design purposes. Bergado et. al., (1996a) summarised the recommended $q_w$ value from many researchers and concluded that under lateral earth pressure ranging from 10 to 350 kPa, $q_w$ value is 10 to 1580 m$^3$/year. Some research results also indicate that the discharge capacity of the drain may also be under the influence of the length of PVDs. The $q_w$ value suggested by Kremer et al., (1982) is based on tests carried out on a 400 mm long drain. Jamiolkowski et al., (1983) proposed that the $q_w$ value can be used for PVDs as long as 20 m. Hansbo (1987) pointed out that for long drains the $q_w$ value is crucial when it is less than 50 to 100 m$^3$/year.

Bo (2004) summarised the $q_w$ specified in projects carried out in various areas around the world. Which suggested that the $q_w$ value ranges from 150 to 3150 m$^3$/year. Holtz et. al. (1991) summarised the major factors that the discharge capacity of PVDs depended on:

- The effective cross section area of PVDs available for pore water flow (free volume). The free volume of some PVDs from different manufactures was summarised by Rixner et al., (1986).
- The influence of the cross-sectional geometry on PVDs from lateral earth pressure; Rixner et al., (1986) presented a relationship between $q_w$ and lateral earth pressure.

- Folding and bending of PVDs caused by soil deformation. Lawrence and Koerner (1988) studied the deformation of strip drains due to soil settlement.

- Clogging of PVD; the clogging effect of PVDs was also considered by Bergado et al., (1996), they recommend the following equation:

$$q_{w\text{(design)}} = (F_t)(F_c)(F_{fc})q_{w\text{(required)}} \quad \text{Equation 2-22}$$

Where

$q_{w\text{(design)}}$ is the specified discharge capacity in design

$q_{w\text{(required)}}$ is the required discharge capacity which can be determined by Equation 2-23 proposed by Kamon et al., (1984).

$(F_t)$ is the reduction factor due to time, which is suggested to be 2

$(F_c)$ is the reduction factor due to the deformation of PVDs, which can be found in Error! Reference source not found..

$(F_{fc})$ is the reduction factor due to clogging, which is suggested to be 3.5

$$q_{w\text{(design)}} = \frac{0.1 \times S \times H \times C_h \times \pi}{4 \times T_h} \quad \text{Equation 2-23}$$

Where

$S$ is the final settlement

$H$ is the depth of improved soil layer
Table 2.2 Percentage in reduction of $q_w$ due to deformation of PVDs (after Bergado et. al., 1996)

<table>
<thead>
<tr>
<th>Type of drain</th>
<th>Bent (10%)</th>
<th>Bent (20%)</th>
<th>Twist (90°)</th>
<th>Twist (180°)</th>
<th>One-clamp 20% Bent</th>
<th>Two-clamps 30% Bent</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alidrain</td>
<td>34</td>
<td>34</td>
<td>42</td>
<td>45</td>
<td>52</td>
<td>51</td>
<td>43</td>
</tr>
<tr>
<td>Amerdrain</td>
<td>18</td>
<td>32</td>
<td>32</td>
<td>50</td>
<td>47</td>
<td>75</td>
<td>42</td>
</tr>
<tr>
<td>Fiberdrain</td>
<td>66</td>
<td>78</td>
<td>78</td>
<td>79</td>
<td>84</td>
<td>86</td>
<td>79</td>
</tr>
<tr>
<td>Flodrain</td>
<td>14</td>
<td>26</td>
<td>23</td>
<td>44</td>
<td>52</td>
<td>81</td>
<td>40</td>
</tr>
<tr>
<td>Geodrain</td>
<td>28</td>
<td>36</td>
<td>31</td>
<td>45</td>
<td>62</td>
<td>70</td>
<td>45</td>
</tr>
<tr>
<td>Mebradrain</td>
<td>38</td>
<td>40</td>
<td>59</td>
<td>59</td>
<td>61</td>
<td>67</td>
<td>54</td>
</tr>
<tr>
<td>Average</td>
<td>33</td>
<td>41</td>
<td>44</td>
<td>54</td>
<td>60</td>
<td>72</td>
<td>51</td>
</tr>
</tbody>
</table>

Other than the properties of PVDs, the installation process can also affect their performance. The installation process of PVDs, which normally includes the following steps:

- Pass the PVD through a hollow steel mandrel, which is an attachment of the PVD driving rig. At the end of the mandrel, an anchor plate with an area larger than the cross section of the PVD, is fixed onto the end of the PVD. The mandrel is then driven into the ground by the mast on the driving rig.

- Figure 2.6 and Figure 2.7 show the mast and the PVD rig used for the site experiment carried out for the current research.

- Once the desired depth is reached, the mandrel is extracted from the ground and the PVD is anchored at depth by the anchor plate.
The mandrel is extracted from the ground until the end is approximately 500 mm above the ground, and then the PVD is cut with a 300 mm stick-out.

PVDs are difficult to install in stiff to very stiff material so they must be vibrated or hammered into the ground, or predrilling might be needed. However, it is uncommon to install PVD in cohesive soil has a consistency of stiff to hard. This installation process disturbs the surrounding soil; this disturbance in the soil surrounding the PVD is called smear, and the region of disturbed soil is called the smear zone. Inside the smear zone, horizontal/radial permeability has decreased because the soil adjacent of PVD has been remoulded and compressed, resulting a lower void ratio value. Sharma and Xiao (2000) carried out large scale lab tests to study the characteristics of the smear zone and found that reconsolidation had more effect on the soil property than remoulding; they also suggested that the smear effect decreases as OCR increases, as shown in Figure 2.8.
Figure 2.6 Mast used for PVD installation in current research
Figure 2.7 Driving rig used for PVD installation in current research

Figure 2.8 Horizontal permeability versus radial distance (after Sharma and Xiao, 2000)
Indraratna and Redana (1998a) also carried out large scale lab tests which revealed that the radius of the smear zone was approximately 100 mm, which was about four to five times larger than the radius of the central drain; Sharma and Xiao (2000) agreed and also concluded that only at the drain soil interface, the horizontal permeability was close to the vertical permeability, as shown in Figure 2.9. This differs from the assumption that a constant reduced horizontal permeability exists in the smear zone, as adopted by Hansbo et al., (1981). A similar conclusion can be drawn from studies by other researchers (Chai and Miura, 1999; Hawlader et al., 2002; Hird and Moseley , 2000; Indraratna and Redana, 1998b; Madhav et al., 1993; Bergado et al., 1991). The decrease in horizontal permeability towards the soil drain interface is assumed to be either linear or parabolic. Rujikiatkamjorn and Indraratna (2009) extended their work (Rujikiatkamjorn and Indraratna, 2007) to produce design charts, such as Figure 2.10, which include the linear variation of horizontal permeability in the smear zone.
Figure 2.9 Ratio of horizontal over vertical permeability (after Indraratna and Redana, 1998a)
Sathananthan (2008) studied the smear affect by using cavity expansion theory and proposed an arbitrary criterion for determining the extent of the smear zone, which is that the ratio between the pore water pressure and the in-situ overburden pressure is larger than unity. This implies that the size of smear zone would change over depth since the overburden stress would also change; Figure 2.11 shows the change in the extent of the smear zone over its depth.
2.1.5 Installation pattern of PVD

Two patterns are commonly used to install PVD; either square or triangular (Figure 2.12). The area of soil improved by a single drain is known as the influence zone of PVD. It is general practice to use circular influence zone in analysis, therefore two different influence zones which correspond to two patterns of PVD installation must be converted to equivalent circular zones based on the equal area method. For a square pattern the equivalent influence radius \( R = 0.564 \times S \), for a triangular Pattern \( R = 0.525 \times S \), where \( S \) is the PVD spacing.
2.1.6 Numerical methods

There has been a considerable advancement in consolidation theory using numerical methods since 1953 when Gibson and Lumb proposed a numerical solution to solve three dimensional consolidation problems. Fox and Berles (1997) adopted a finite difference method and proposed correction factors for Terzaghi 1-D consolidation to account for changes in the thickness of drainage layers. They showed that less time is required to reach a certain degree of consolidation by reducing the thickness of each layer during consolidation. Compressible pore fluid was considered by Fox and Qui (2004) using the finite difference method; this relaxed the original assumption that pore fluid is incompressible.

Traditional radial consolidation theory that combined vertical and radial flow together to study vertical drains began with Barron (1948a). Two different
fundamental assumptions, equal strain and free strain, lead to different solutions, but by considering the well resistance and smear effect, the consolidation theory of vertical drains became closer to reality (Hansbo, 1981).

Randolph and Worth (1979) proposed an analytical solution for consolidation around a pile after pile driving; here the pore water pressure is expressed in equation 4.1 to 4.4

\[
\begin{align*}
  u &= \sum_{n=1}^{\infty} B_n e^{-\alpha_n t} \theta_0(\lambda_n r), \quad r_0 \leq r \leq r^* \quad \text{Equation 2-24} \\
  u &= 0, \quad r > r^* \quad \text{Equation 2-25} \\
  B_n &= \frac{4C_u}{\lambda_n^2} \frac{[\theta_0(\lambda_n r) - \theta_0(\lambda_n R)]}{[r^2 - r_0^2 \theta_0^2(\lambda_n r^*) - r_0^2 \theta_0^2(\lambda_n r_0)]} \quad \text{Equation 2-26} \\
  \theta_i(\lambda r) &= J_i(\lambda r) + \mu Y_i(\lambda r) \quad \text{Equation 2-27}
\end{align*}
\]

Where

The radius \( r^* \) was taken as between 5 and 10 times the radius of pile R.

\( J_i \) is the Bessel function of first kind of \( i^{th} \) order

\( Y_i \) is the Bessel function of second kind of \( i^{th} \) Order.

\( \theta_i \) is the cylinder function of \( i^{th} \) order.

Randolph and Worth's solution is in the form of an infinite series that involves Bessel functions. Which is difficult to be adopted in projects. In addition, Randolph and Worth's model also assumed that the deformation of soil around a driven pile
is only radial, and if this is true, no negative skin friction should develop during consolidation, which contradicts the observation made by Fellenius and Broms (1971) and Fellenius (1972).

For the consolidation of soil around one single PVD, it is straightforward to simulate the problem using an axisymmetric model. However, when dealing with multiple PVDs, axisymmetric model is no longer valid. Indraratna and Redena (2000) proposed an equivalent Plane strain model for finite element analysis where a traditional cylindrical unit cell is converted to a drain wall, as shown in Figure 2.13. The equivalent plane strain permeability to be used in finite element simulation can be expressed as per equations 4.5 to 4.7

\[
\frac{k'_{hp}}{k_{hp}} = \frac{\beta}{\ln \left( \frac{n}{s} \right) + \left( \frac{k_h}{k_{hp}} \right) \ln(s) - 0.75 - \alpha}
\]

Equation 2-28

\[
\alpha = \frac{2}{3} - \frac{4b_s^3}{3B^3} + \frac{2b_{w1}^2}{B^2} - \frac{2b_{w1}}{B}
\]

Equation 2-29

\[
\beta = \frac{b_{w1}^2}{B} + \frac{b_s^3}{3B^3} - \frac{2b_{w1}^3}{3B^3} - \frac{2b_{w1}b_s}{B^2} + \frac{b_{w1}^2}{B^2} + \frac{b_{w1}b_s}{B^3}
\]

Equation 2-30
Walker and Indraratna (2006) considered the parabolic distribution of horizontal permeability in the smear zone; in this theory, \( \mu \) as in Hansbo's theory (1981) is modified to be:

\[
\mu_p = \ln \left( \frac{s}{s_0} \right) - \frac{3}{4} + \frac{\kappa (s-1)^2}{(s^2 - 2ks + k)} \ln \left( \frac{s}{\sqrt{k}} \right) - \frac{s(s-1)\sqrt{k(k-1)}}{2(s^2 - 2ks + k)} \ln \left( \frac{\sqrt{k} + \sqrt{k-1}}{\sqrt{k-1}} \right) + \frac{k_h}{q_w} \pi z (2l - z)
\]

Equation 2-31

Where

\( \kappa = \frac{k_h}{k_0} \) is the ratio of smear permeability

Figure 2.13 Unit cell for plane strain model (after Indraratna and Redena, 2000)
$k_u$ is the horizontal permeability at an equivalent radius of drain

Indraratna et al., (2008) proposed a solution for consolidation under circular loading with PVDs where the PVDs are modelled as concentric circular drain walls.

The proposed equivalent horizontal permeability is expressed in Equation 2-32.

$$k_{h,ring} = \frac{2}{3} \alpha^2 k_h \left[ \ln \left( \frac{n}{S} \right) + \frac{k_h}{k_s} \ln(s) - \frac{3}{4} \right]$$  \hspace{1em} \text{Equation 2-32}$$

For drains installed in a square pattern $\alpha = 0.887$, and for drains installed in an equilateral triangular pattern $\alpha = 0.952$ (Holtz et al. 1991)

2.2 Introduction to pile foundation

Pile foundations have always been an important part of geotechnical research. After centuries of study, analytical solutions are well established for estimating capacity and settlement of single pile and pile group. Moreover, there are numerical tools can be adopted to analyse pile soil interactions. The status quo of research into pile foundation that is relevant to this thesis is discussed in this section.

A pile foundation is probably one of mankind’s earliest attempts to treat soft ground, indeed literature indicates that piles have been used as early as 4,000 years ago (Fleming et al., 2008). In China, timber pile were used to support heavy infrastructure such as mason bridges as early as the Han Dynasty, which is about 2,000 year ago (Tomlinson and Woodward, 1993). Even though our present knowledge and techniques are probably far beyond the imagination of our
ancestors, their original concept of using rigid inclusions to improve soft ground is still valid and it will probably be the same in the foreseeable future.

Poulos and Davies (1980), mentioned that the earliest modern literatures on pile foundations can be attributed to Piles and pile driving edited by Wellington in 1893. Since then, a great volume of field tests, lab tests, and analytical and numerical studies have been published. The following section includes some of the literature, with a focus on negative skin friction and the pile driving process.

2.2.1 Types of piles

Piles are generally categorised based on their material, their function, and their method of installation. The most common materials for piles are timber, steel and concrete with and without reinforcement. In terms of their functions, piles can be used to carry foundation loads, reduce settlement, resist uplift forces, and support excavation or slopes. Based on their method of installation, piles can be classified into displacement and replacement piles, as shown in Figure 2.14.

Figure 2.14 Types of piles based on their installation methods
Other than tradition pile foundations, there are some ground improvement techniques which can be considered as piles, such as stone columns. Stone columns can be used to carry surface loads and reduce settlement, as well as serve as vertical drains. The downside with stone columns is the uncertainty with regards to their properties.

2.2.2 Load capacity of a single pile

It is generally accepted that the load capacity of a pile consists of the end bearing and shaft friction, as shown in Figure 2.15.

\[ Q = Q_s + Q_b \]

Figure 2.15 Vertical load carried by a single pile

Fleming et al. (2008) indicate that shaft resistance is considered easier to mobilise than the base capacity because only 0.5 to 2\% of the pile diameter of pile soil relative movement is needed to fully mobilise the shaft capacity whereas 5 to 10\% of a pile’s base diameter is needed to fully mobilise the base capacity.
Simple method exists for estimating the pile load capacity. For non-cohesive material the end-bearing capacity can be evaluated as

\[ q_b = N_q \sigma'_v \]  
 Equation 2-33

The value of \( N_q \) can be found in chart produced by Berezantzev et. al., (1961)

Shaft friction can be calculated as

\[ \tau_s = K \sigma'_v \tan \delta \]  
 Equation 2-34

Where \( K \) is the coefficient between the effective vertical pressure and horizontal pressure, and \( \delta \) is the angle of friction between soil and pile.

For cohesive soil, the end-bearing can be assessed as

\[ q_b = N_c S_u \]  
 Equation 2-35

where an \( N_c \) value of 6 to 9 is typically used. The shaft friction can be calculated as

\[ \tau_s = \alpha S_u \]  
 Equation 2-36

Where the value of \( \alpha \) can be obtained from charts prepared by various authors.

2.2.3 Settlement of a single pile

The settlement of a single pile is more complex than the bearing capacity. The earlier solution to this problem is mainly empirical, so Meyerhof (1961) proposed the following equation:

\[ \rho = \frac{d_b}{30F} \]  
 Equation 2-37

Where

\( \rho \) is the settlement of pile

\( d_b \) is the diameter of pile base
Even though this formula seems very crude, it is still the most popular tool for estimating pile settlement due to its simplicity, especially in preliminary design. During the detail design stage, Mayerhof’s method may not be accurate enough, so in most projects today, numerical methods are essential tools for the design process. A 2D/3D analysis of the capacity and/or deformation of pile foundations can be done with various commercial numerical analysis software packages such as PLAXIS, Flac, Phase 2, and Abaqus. However, not every engineer has access to numerical tools due to their high cost, in fact even if those software packages are available, they are not used in most projects due to budget constraints; in these cases, elastic analysis is very useful. Existing elastic solutions are briefly discussed below.

The load transfer method was initially proposed by Coyle and Reese (1966). This method needs iteration after an initial base settlement has been assumed. This assumed settlement will be compared with the calculated settlement and if they do not agree, the assumed settlement is replaced by the calculated settlement and the process is repeated until convergence is achieved. Coyle and Sulaiman (1967) produced a design load-transfer chart for piles in sand.

Randolph (1978) developed an expression for the load settlement ratio of the pile head as Equation 2-38. Design charts have also been produced for this method.

\[
\frac{P_t}{w_t d G_L} = \frac{2\eta}{(1-v)\xi} + \frac{2\pi \rho \tan(\mu L) L}{\xi \mu L d} + \frac{8\eta}{\pi \lambda (1-v)\xi} \frac{\tan(\mu L) L}{\mu L d}
\]

Equation 2-38

Where
$P_t$ is vertical load acting on pile head

$w_t$ is pile head settlement

$G_L$ is shaft soil shear modulus at pile base level

$\xi$ is a shaft stress softening factor

$\eta$ is group efficiency

Guo and Randolph (1997) extended the above solution to include Gibson’s soil, the modulus of which is a function of depth $z$ in the following form

$$G = mz^n$$  \hspace{1cm} \text{Equation 2-39}$$

Where $0 \leq n \leq 1$.

Based on Mindlin’s (1936) solution for loading within a semi-infinite mass, Poulos and Davies (1980) developed an elastic analysis method in which piles are separated into many shorter sections. This method is more rigorous and computer program is made to obtain the results. Both methods (Randolph, 1977, Poulos and Davies, 1980) can be extended to pile groups.

2.2.4 Negative skin friction

When a pile foundation is installed in consolidating soil, and according to Terzaghi et al. (1996), the pile is not as compressible as the surrounding soil, so the soil moves downwards relatively to the pile, and the load is transferred to the pile by skin friction.

Bozozuk (1972) studied a 49 m long steel pipe pile in marine clay and concluded that:

- The fill had settled 53 cm during the observation period, dragging the pile down with it.
- The peak compressive load is about 140t
- The neutral point is 22 m below the top of the pile
- The skin friction loads are proportional to the in-situ horizontal effective stresses, and

the negative skin friction $q_n$ can be expressed as

$$q_n = \beta \sigma_v'$$

Equation 2-40

Bjerrum et al. (1969) field tested steel pipe piles in Norway. In his program 40-metre-long by 500 mm diameter piles were driven into bedrock. The soft clay layer is 27 m thick and lies under a 13-metre-thick layer of fill. After 2 years post installation, a 56.3 mm difference in settlement was observed between the pile head and ground surface around the pile; in fact, the measured maximum dragload is about 4, 000 KN, and the back calculated $\beta$ ranges from 0.2 to 0.3.

Not only consolidation due to preloading/surcharge can cause high dragload, the reconsolidation of soil adjacent to the pile due to the disturbance of pile driving can incur significant dragload. Fellenius and Broms (1969) and Fellenius (1972) reported a case where 300 mm diameter concrete piles were driven through a 40 metre thick clay deposit founded on an underlying sand layer; immediately after driving the piles the load on them was insignificant, but after 5 months post installation, the dragload due to negative skin friction reached around 300 KN to 350 KN, and the corresponding $\beta$ was almost 0.10.

There is always a point where the settlement of a pile is equal to the settlement of the surrounding soil, as mentioned by Fellenius (1984). At this point the maximum compression load can be detected in the pile; called the neutral point. Above the neutral point the soil moves relatively downwards compared to the pile
so negative skin friction is generated, but below the neutral point the pile settles more than the surrounding soil and a positive skin friction is generated. Guo (2012) suggests that for end bearing piles, the ratio between the distance of the neutral point to the ground surface and the length of the pile is close to one, whereas for floating piles the ratio is close to 0.7. Indraratna et al., (1992) reported a field study on the downdrag acting on driven piles in soft Bangkok clay. They studied two driven piles, one with bitumen coating, and another without. The thickness of soft clay was between 10 to 15 metres, and there was a 2 m embankment on top of the ground surface. The performance of those two piles was monitored for 9 months; the recorded data indicated that the maximum dragload reached 30 t and 12 t for uncoated and coated piles respectively. They concluded that negative skin friction can be predicted by the effective stress approach and using values of $\beta$ between 0.1 and 0.2. They also suggested it could be beneficial to let the ground to settle, say for 1 month, after the surcharge load has been applied and then install the piles. Since consolidation settlement on the ground surface due to embankment construction is generally large and dragload can be rapidly mobilised within a short period of time.

2.2.5 The effect of installation

Compared to a replacement pile, a displacement pile has more influence on the property of the surrounding soil. Poulos and Davis (1980) suggest that the installation method can affect the behaviour of the pile and the surrounding structures due to the reaction of the soil. The load capacity of a pile will gradually increase over time due to disturbance of the soil during piling. Compared to non-
cohesive material, cohesive material is more sensitive to pile driving, and the strength can be altered significantly as piles are installed. Randolph & Wroth (1979) pointed out that an increase in the undrained shear strength of surrounding clay, known as "set up", is often encountered after piles have been installed, and this increase in strength contributes to the increase of geotechnical capacity of piles over time.

Soderberg (1962) pointed out that the shape of the increased load capacity of piles over time is similar to the shape of the dissipation of pore water pressure over time. Therefore, it is assumed that the generation and dissipation of pore water pressure around driven piles during and after installation can be used to predict the change in the geotechnical capacity of piles over time.

Many field and lab tests have been carried out to investigate the pore water pressure generated by pile driving and subsequent dissipation of excess pore water pressure over time. Poulos and Davies (1980) reviewed work from previous researchers (Bjerrum and Johannessen, 1961, Soderman and Milligan, 1961, Lo and stermac, 1965, Airhart et al., 1969 and D'Appolonia and Lambe, 1971) and proposed a procedure to estimate excess pore water pressure generated by pile driving.

D'Appolonia and Lambe (1971) derived a formulation to predict the maximum pore pressure developed near the pile surface.

$$\frac{\Delta u_m}{\sigma'_{vo}} = \left[ (1 - K_0) + \frac{2s_u}{\sigma'_{vo}} \right] A_f$$  \hspace{1cm} \text{Equation 2-41}

Where

$\Delta u_m$ is the maximum excess pore water pressure at pile surface

$A_f$ is the pore water pressure coefficient A at failure
Lo and Stermac (1965) proposed a constant excess pore water pressure inside the failure zone which is equal to $\Delta u_m$, but outside the failure zone the pore pressure drops significantly. They suggested that the radius of the failure zone is about 4 times the radius of the pile which interestingly, is similar to the smear zone predicted for PVD installation. This is probably because the failure zone and smear zone are both created by the vertical insertion of rigid object into clay material, and they have both been analysed with cavity expansion theory in these two cases.

Yu (2013) stated that cavity expansion theory has developed significantly over the past 20 years (1980 - 2000). Cavity expansion theory has been applied to a wide range of geotechnical problems such as pressuremeter tests, cone penetration tests, the behaviour of tunnels and underground excavations, the performance of pile foundations, and the effect of pile driving. Figure 2.16 shows a sketch of the cavity expansion problem.
Figure 2.16 Sketch of cavity expansion problem

A brief description of this problem is that: a cavity with a radius "a" inside a media with a radius "b" has initially been subjected to a uniform outside pressure "\( P_0 \)". When a pressure "\( p \)" is applied to the inner surface of the cavity the stress and deformation is studied.

Cavity expansion theory is the most popular way of estimating the effect of installation. It has been used to study the installation of PVDs, pile foundations, and sand piles. A lot of work has been done to verify the effectiveness and accuracy of this method by comparisons to experiments (single gravity and multi-gravity environment) and field observations.

Ladanyi (1963) is one of the pioneers who use cavity expansion theory to predict excess pore water pressure during pile driving. He proposed an inverse variation
outside the failure zone between the excess pore water pressure and the square of the distance from the pile.

$$\Delta u = \frac{\Delta u_m}{(r/R)^2}$$  \hspace{1cm} \text{Equation 2-42}

Where

$R$ is the radius of the failure zone which is proposed to be $3\alpha$ to $4\alpha$ for insensitive clay and $8\alpha$ for sensitive clay (Nishida, 1962).

Randolph and Worth (1979), based on Gibson and Anderson's (1961) solution for a pressuremeter, proposed a distribution of excess pore water pressure.

The pressure immediately after installation is expressed as:

Inside the plastic zone ($r_0 \leq r \leq R$)

$$\Delta u = 2c_u \ln \left( \frac{R}{r} \right)$$  \hspace{1cm} \text{Equation 2-43}

Outside the plastic zone $\Delta u$ is zero

Where $R$ is the radius of the plastic zone

$$R = r_0 \sqrt{\frac{G}{c_u}}$$  \hspace{1cm} \text{Equation 2-44}

At the pile surface where $r_0 = r$, $\Delta u = c_u \ln \left( \frac{G}{c_u} \right)$

Cylindrical cavity expansion was used by Asaoka et al., (1994) to simulate the installation effect of sand pile driving. The results were compared with a series of triaxial compression tests in which the disturbance-consolidation-undrained shearing sequence was followed. Although the similarity of comparison is not clear, the theoretical and experimental analysis shows proximate load-settlement curves.
A semi-empirical approach was proposed by Lee et al., (2004) based on cylindrical cavity expansion theory, and verified by measuring the change of total stress and pore pressure of clay in centrifuge tests during the installation of sand compaction piles. It was suggested that the plain strain cavity expansion theory gives good results of predicting peak stress and the whole process, however, may overestimate the residual stress after installation.

2.3 Combined use of PVD and pile foundation

In recent decades the theories which can analyse the effect of combined ground improving techniques has been pursued by many researchers. Significant achievements have been made in predicting mechanically stabilised embankments by combining soil mixing and PVD, etc. Currently there is no research focus on reducing the undesirable response of soil during piling and pile soil interaction by adopting PVD techniques.

The combined use of pile and geosynthetic reinforcement has proven to be an effective method to improve foundations for embankments. Researchers such as Han and Akins (2002) have demonstrated the benefits and design considerations of geogrid-reinforced and pile-supported earth structures. Nashed, et. al., (2004) studied the combined use of stone column and dynamic compaction to prevent the liquefaction of silty soil

Combined use of PVD and Dry Jet Mixing (DJM) has attracted the attention of researchers and practitioners in recent years, especially in China where the combined use of PVD and DJM has gained popularity in high speed railway projects. Ye, et al., (2012) studied the consolidation of a composite foundation with soil–cement columns and PVD, and proposed an analytically solution. Liu,
et al., (2008) studied the performance of DJM-PVD in soft clay based on a highway project in Jiangsu Province, China. They concluded that:

- The inclusion of PVDs reduced the magnitude of excess pore water pressure and increased the rate of dissipation.
- The use of PVDs improved the quality and strength of the DJM columns.
- The use of PVDs could lead to cost effective design compared to the conventional DJM method.

In order to solve the problems of excessive pore water pressure and soil heave associated with pile driving, attempts to combine PVD and piles to improve soft ground have also been made by geotechnical engineers and researchers. Holtz and Boman (1974) and Fellenius (1975) discussed a technique to reduce excess pore water pressure induced by pile installation by nailing PVDs to the pile shaft before driving; test results indicated that the maximum pore water pressure decreased by at least 50%. Holtz and Boman (1974) suggested that this new technique was the most cost-effective method for the particular project, and they also think a potentially cheaper and more effective procedure is to install the drains before piling.

Tefera, et. al., (2011) presented a bridge construction project in Drammen in the south eastern part of Norway about 50km from Oslo. Soft sensitive clay was found on this site, so to reduce excess pore water pressure during driving, predrilling, open-end pile, and PVDs attached to the pile shafts were used. Although the authors did not conclude that a significant reduction in pore water pressure had been achieved, the measured pore water pressure was much lower than those in the literature, as shown in Figure 2.17.
Figure 2.17 Measured pore water pressure during piling (after Tefera, et. al., 2011)

Unsuccessful cases can also be found in literature. Bradshaw and Baxter (2006) presented a project in the US where a line of PVDs had been installed between the piling area and adjacent sensitive building to reduce the pore water pressure, and to reduce lateral soil movement and soil heave due to pile installation. However, the results indicate that installing PVDs around the piling area may not effectively reduce the pore water pressure, lateral soil movement and soil heave due to pile driving.

2.4 Summary
Both PVDs and piles have been the focus of research in the last couple of decades. The consolidation theory considering PVDs was developed by Barron in early 1950s (Barron, 1950). Then many researchers studied the effect of smear zone, drain resistance and factors has impact on PVD efficiencies. Pile
foundations were extensively studied in terms of capacity and settlement under various loading conditions by researchers, effect of pile installation including negative skin friction was recognised by researchers, (Poulos and Davis, 1980). Some attempts were made to reducing ground movement and excess pore water pressure due to piling by combing PVDs and piles. No research conducted on improve the ground by preloading with PVDs before pile installed.
CHAPTER 3. LARGE SCALE FIELD TESTS

3.1 General
Full scale field tests have always been one of the most reliable methods for studying geotechnical problems; Wood (2003) stated that full scale field tests can be used to validate analytical, empirical or numerical theories. In addition, it can be used to study problems which are highly site dependant, for example, trail embankments are one common form of full-scale field tests. In this research the following aspects of PVD and pile improved soft soil will be investigated.

1. Pore water pressure generated during piling in treated and untreated soft clay.
2. Dissipation of pore water pressure due to piling with and without PVDs.
3. Negative skin friction on piles due to soft soil consolidation with and without PVDs improvement.
4. The lateral movement of soil around a pile while driving with and without PVDs improvement.

The results of these full-scale field tests were used to validate the analytical and numerical predictions of those four aspects listed above.

3.2 Previous large-scale field test on piling
Full scale model tests have been widely accepted and adopted by generations of geotechnical engineers and researchers to investigate the performance of various types of foundations. Since the 1960s, many researches into the behaviour of pile foundation were carried out by full scale field tests (Bjerrum and Johannessen, 1961, Koizumi and Ito, 1967, Holtz and Boman, 1974, Bozozuk 1972, Bozozuk et al., 1978, Roy et. al., 1981, Indraratna et al., 1992, Hwang et
al., 2001, Pestana et al., 2002, and Jeong et al., 2014). Table 3.1 presents a brief introduction of some of these tests.

Fellenius (2006) reviewed several full scale, long term tests on instrumented piles and concluded that:

- The dragload is important for piles where $L \geq 100D$ and where $L$ is the length of the pile and $D$ is the diameter.
- Settlement of pile due to dragload is very important for low-capacity short piles.
- Reconsolidation of soil around a pile after installation will cause significant dragload.

3.3 Introduction to the test area

The Australian Research Council Centre of Excellence for Geotechnical Science and Engineering (CGSE) was established in 2011. Since its formation, CGSE has provided a platform for fundamental research into geotechnical engineering. To understand the behaviour of the natural estuarine soft clays often encountered in the coastal regions and flood plains in Australia, which commonly serve as the foundation material for much of its geotechnical infrastructure, CGSE established a National Soft Soil Field Testing Facility (NFTF) at Ballina, NSW. As introduced by Kelly (2013), the NFTF allowed for high quality in situ testing, sampling, and full scale geotechnical model testing to be carried out.

The soft clays in the Ballina region are known to have high moist content, low undrained shear strength, and high compressibility. In addition, Ballina clays also have relatively high organic content and expansive minerals and weak cementation, and present challenges for civil projects founded on them. The
NFTF is located approximately 5 km northwest of the Ballina-Baron airport. A map of the NFTF site is shown in Figure 3.1.

Figure 3.1 Location of NFTF in Ballina, NSW
Table 3.1 Brief description of some large-scale pile tests

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Pile type</th>
<th>D (mm)</th>
<th>L (m)</th>
<th>Data collected</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft marine clay</td>
<td>Square concrete end bearing pile</td>
<td>300 (side)</td>
<td>To bed rock</td>
<td>Excess pore water pressure during pile driving</td>
<td>Bjerrum and Johannessen (1961)</td>
</tr>
<tr>
<td>Soft silty clay with shell</td>
<td>Group of closed end steel tube floating pile</td>
<td>300</td>
<td>5.5</td>
<td>Excess pore water pressure, lateral stress in soil and axial strain on pile</td>
<td>Koizumi and Ito (1967)</td>
</tr>
<tr>
<td>Organic clay / silt</td>
<td>Timber end bearing piles with vertical drain attached</td>
<td>180</td>
<td>16-25</td>
<td>Excess pore water pressure during and after pile driving</td>
<td>Holtz and Boman (1974)</td>
</tr>
<tr>
<td>High plasticity sensitive silty clay</td>
<td>Precast concrete closed end bearing hexagon pile</td>
<td>300</td>
<td>26</td>
<td>Compressibility, strength, pore pressures and surface heave</td>
<td>Bozozuk (1978)</td>
</tr>
<tr>
<td>Soft silty clay / very soft clayey silt</td>
<td>Group of closed end floating pile</td>
<td>219</td>
<td>7.5</td>
<td>Excess pore water pressure, point resistance, skin friction during driving and soil strength around pile</td>
<td>Roy et. al. (1981)</td>
</tr>
<tr>
<td>Soft Bangkok clay</td>
<td>Precast concrete pile (1 coated, 1 uncoated)</td>
<td>400</td>
<td>26</td>
<td>Excess pore water pressure, pull-out shaft resistance, negative skin friction, ground settlement</td>
<td>Indraratna et. al. (1992)</td>
</tr>
<tr>
<td>Loose sand underlain by soft clay</td>
<td>Group of precast concrete close end floating</td>
<td>800</td>
<td>34</td>
<td>Excess pore water pressure, vertical ground surface movement, lateral soil movement</td>
<td>Hwang et. al. (2001)</td>
</tr>
<tr>
<td>Very soft to soft clay</td>
<td>Steel pipe closed end tip bearing pile</td>
<td>610</td>
<td>36.6</td>
<td>Lateral deformation and excess pore water pressure</td>
<td>Pestana et. al. (2002)</td>
</tr>
</tbody>
</table>
Three field test embankments were constructed simultaneously; one large 40m diameter embankment was constructed for vacuum preloading tests, and two smaller embankments were constructed to study the interaction between piles and PVDs. Although this thesis focused on the two small embankments, data acquired from the vacuum preloading embankment were used to facilitate the analyses. All three embankments were situated at the North-West corner of the NFTF, as shown in the map. The terrain rises towards the North and West and drops slightly towards the East, and there was a stream flowing along the Eastern boundary which seasonally floods the NFTF site. As Figure 3.2 shows, there were two test embankments at the southern part of the NFTF, about 50 metres away from the proposed field test, and the vacuum preloading test embankment was about 20 metres Northwest of the proposed field test.
Before carrying out any type of geotechnical field study, the geological condition of the area of interest must first be modelled and interpreted. The two essential parts of a working geotechnical model are soil stratigraphy and soil parameters; soil stratigraphy is normally interpreted from a geological survey, the soil classification from Cone Penetration Tests (CPT), Bore Holes (BH) results, and...
local experience, whereas the soil parameters are generally interpreted from the results of field and lab tests.

3.3.1.1 Soil stratigraphy for geotechnical model

The geological map 1:250,000 Geological series sheet SH 56-3 published by Geological Survey of N.S.W., Department of Mines, Sydney, (Brunker et. al., 1972) indicates that the Ballina area is covered by river gravels, alluvium, and sand and clay, followed by shale, siltstone and sandstone. Indraratna et al. (2012) described a test embankment for the Ballina Bypass Project (BBP) located about 500 m North-west of NFTF, in which they suggested that the soil profile consisted of a 10 m thick layer of soft silty clay over a firm layer of silty clay that extends to 25 m below ground level (BGL). Pineda et al., (2016) indicated that at NFTF, the clay fraction was predominant between 2 m and 11 m BGL and shell fragments are distributed randomly along the soil above 4.5 m BGL. Figure 3.3 shows the field test location on a geological map.

Before the test embankments were constructed, a site investigation (SI), including 6 CPTs and 2 BHs, were carried out inside the test area. The locations of CPTs and BHs are shown in the outcome of the site investigation, which were discussed below, and generally agree with the conclusions in literature, albeit
with slight localised variations.
Figure 3.7 present the raw data measured from CPT tests; they reveal that cone resistance is between 1 to 2 MPa above 2 m BGL and drops below 0.5 MPa from 2 m deep to 12 m deep, and then rise up to a maximum of 25 MPa between 12 m to 20 m BGL. Below depth of 20 m the cone resistance values drop back to roughly 1 mPa. Only CPT 105 went below 25 m deep and \( q_c \) increased to an average value of 3 MPa. Base on the \( q_c \) vs depth results, the underground soil above 25 m deep can be assumed to have 4 different layers: Layer 1: 0-2 m deep, layer 2: 2-12 m deep, layer 3: 12 – 20 m deep, and layer 4: 20 to 25 m deep. Plots of skin friction and pore water pressure versus depth also confirm this assumption.
Figure 3.3 Location of field test on geological map (after Brunker et. al., 1972)
Figure 3.4 CPTs and BH location

Figure 3.5 Measured cone resistance versus depth
Figure 3.6 Measured skin friction versus depth
Figure 3.7 Measured pore water pressure behind cone tip versus depth

Figure 3.8 Soil behaviour index classification from CPTs
Soil classification was based on CPT results following the method proposed by Robertson (2010). Soil behavior type index ($I_{SBT}$) was calculated and plotted against the depth below natural ground in Figure 3.8. It is obvious that between 0 to 2 m depth, the soil is a mixture of sand, silt, and clay, and between 2 to 7 m depth there is almost no other content than pure clay. From 7 to 12 m depth there is an increasing percentage of organic content, but this diminishes from 12 m to 15 m depth. From 12 to 15 m deep there is less clay and silt and more sand particles in the soil, but below 20 m the clay becomes predominant again.

The borehole logs presented in Figure 3.9 and Figure 3.10 generally agree with the stratigraphy interpreted from the CPT results. Photos of the soil samples taken from the boreholes are attached in the appendix.
Figure 3.9 Borehole log for BH01
Figure 3.10 Borehole log for BH02
According to the current SI, a 3D geotechnical model can be built and is shown in Figure 3.11 and Figure 3.12.

Figure 3.11 3D geotechnical model looking from the southeast.

Figure 3.12 3D geotechnical model looking from the northwest.

Although a comprehensive 3D model can better represent the site conditions, it is generally too complicated for most of the projects and studies when the spatial variation of underground material is insignificant, 3D models are often simplified into 2D models. By considering the SI results and the information from literature (Indraratna et al., 2012 and Pineda et al., 2016), the geotechnical model at the field test site above 25 m can be simplified and generalized as a 2 m thick crust,
followed by a 10 m thick very soft to soft Holocene clay layer with a high moisture content; the crust is clayey silt with some sand in the mix. The Holocene clay can be further divided into an upper sub layer of Holocene clay, where from 2 to 7 m BGL is predominately clay, and a lower sub layer of Holocene clay with an increasing amount of organic material along the depth. Bands of shell fragments are found within the Holocene clay layer. Immediately after the Holocene clay is a 3 m thick transition zone which consists of sandy/silty clay with an increasing sand fraction along the depth. Beneath the transition zone is a 5 m thick layer of silty sand with traces of clay which is loose to median dense and underlain by Pleistocene clay of firm to stiff consistency.

3.3.1.2  Soil parameters for geotechnical model

While geological models focus on the stratigraphy, geotechnical models are more concerned on material properties. The soil properties are determined by insitu and lab tests such as a Cone penetration test (CPT/CPTu), Dissipation test (DT), Standard penetration test (SPT), Field vane test (FVT), Pocket penetration test (PPT), a triaxial test and an oedometer test. Table 3.2 summarizes the representative basic soil properties for each layer. Moisture content, dry density, void ratio, and the plastic limit and liquid limit were obtained from specimens prepared for triaxial and oedometer tests; the ground water table (GWT) was generally very high at this site. During the dry season the GWT was about 0.2 m BGL, however in wet season the site was often flooded, leaving the GWT temporally above (less than 0.3 m) the natural ground. This high GWT rendered fully saturated soil with a high nature Moist Content (MC) on site. In the crust layer, the MC was relatively low (around 30%), but still higher than Plastic Limit.
(PL), which was less than 30%. Between 2 to 12 m BGL, where the clay fraction was predominant, the MC was much higher (100% to 125%) and very close to LL (110% to 130%). Below 12 m BGL, the MC drops rapidly to less than 30% due to increasing amounts of coarse material. The void ratio along the depth follows the same trend: a moderately high crust layer (1.1), which increases to around 3 in the clay layer and drops below 1 in the transition and sand layer. The dry density matches the soil classification from the CPTs results. Above 2 m BGL and between 12 to 15 m BGL, within the crust and transition zone respectively, where a mix of clay, silt, and sand coexist, the dry density is around 1.27 to 1.34 t/m$^3$, whereas between 2 to 12 m BGL, within the clay layer, the dry density of the soil is estimated to be 0.75 to 0.8 t/m$^3$. Below 15 m BGL the dry density increased to approximately 1.48 t/m$^3$; this is reasonable because diminishing amounts of fines was confirmed by the CPT test results and the split spoon samples retrieved from BH1. Between 15 and 20 m BGL, the porewater pressure measured during CPTs were less than the hydrostatic pressure, which suggested that this layer is potentially dilatant. Dilatant frictional soil generally has high consistency and was selected as bearing stratum in the field experiments.

The undrained shear strength ($S_u$) of different layers of soil are interpreted from CPT results by the method proposed by Lunne et al., (1997). The $N_{kt}$ value is determined by comparing the CPT results with corrected FVT results based on the method proposed by Bjerrun (1973) and the reported $S_u$ values from nearby projects. It is concluded that with a $N_{kt}$ number of 13, the interpreted $S_u$ matches the reported values in the literature and is used to produce a continuous $S_u$ profile along depth. The $S_u$ profile is plotted together with corrected FVT results and
previous SI results from Kelly et. al. (2014) in the same area, as shown in Figure 3.13.

Note that above 2 m and below 12 m BGL, the Su is more than 60 kPa. In the clay layer the minimum Su value is about 10 kPa at 2 m BGL and increases approximately linearly with depth to around 20 kPa. The Su values from the pocket penetrometer tests conducted on tube samples on site also falls into the range of 12 to 25 kPa in the soft clay layer.

Series of lab tests, including oedometer tests, unconsolidated undrained triaxial tests and consolidated undrained tests were carried out to obtain the effective strength and consolidation parameters of the curst and soft clay; the results are similar to those reported by Pineda (2012). Table 3.2 and Table 3.3 presents the parameters selected by the author for analytical and numerical studies. The virgin compression index (Cc) of the clay layer is between 1.2 and 1.4, which suggests that the clay layer is highly compressible. The secondary compression index (Cα) is 0.04 and 0.06 for soft soil layers 1 and 2 respectively. The ratio between Cα/Cc ranging from 0.033 to 0.043 agrees with the value of 0.04±0.01 for soft clays reported by Mesri and Castro (1987); this ratio also matches most of the value reported by Pineda et al. (2016). The over consolidation ratio (OCR) in the curst and the first and second layers of soft clay are 6, 1.7, and 1.2 respectively.
Figure 3.13 Su from CPTs and FVTs
Table 3.2 Basic soil parameters

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (m)</th>
<th>MC (%)</th>
<th>Dry density (t/m³)</th>
<th>Void ratio</th>
<th>PL</th>
<th>LL</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crust</td>
<td>0-2</td>
<td>32</td>
<td>1.34</td>
<td>1.1</td>
<td>29.3</td>
<td>46.5</td>
</tr>
<tr>
<td>Soft soil 1</td>
<td>2-7</td>
<td>105.5</td>
<td>0.75</td>
<td>2.9</td>
<td>38.1</td>
<td>111.4</td>
</tr>
<tr>
<td>Soft soil 2</td>
<td>7-12</td>
<td>124.1</td>
<td>0.80</td>
<td>3.1</td>
<td>49.6</td>
<td>126.7</td>
</tr>
<tr>
<td>Transition zone</td>
<td>12-15</td>
<td>29.6</td>
<td>1.27</td>
<td>0.8</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Sand layer</td>
<td>15-20</td>
<td>25.8</td>
<td>1.48</td>
<td>0.5</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
### Table 3.3 Soil strength and consolidation properties

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (m)</th>
<th>C_u (kPa)</th>
<th>c'(kPa)</th>
<th>Ø'(degree)</th>
<th>C_c</th>
<th>C_r</th>
<th>C_a</th>
<th>k_h (m/s)</th>
<th>k_v (m/s)</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Crust</td>
<td>0-2</td>
<td>60</td>
<td>5</td>
<td>30</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>7×10^{-8}</td>
<td>6×10^{-8}</td>
<td>6</td>
</tr>
<tr>
<td>Soft soil 1</td>
<td>2-7</td>
<td>12.5</td>
<td>10</td>
<td>25</td>
<td>1.2</td>
<td>0.13</td>
<td>0.04</td>
<td>3×10^{-8}</td>
<td>1.5×10^{-8}</td>
<td>1.7</td>
</tr>
<tr>
<td>Soft soil 2</td>
<td>7-12</td>
<td>17.5</td>
<td>10</td>
<td>25</td>
<td>1.4</td>
<td>0.16</td>
<td>0.06</td>
<td>3×10^{-9}</td>
<td>1.5×10^{-9}</td>
<td>1.2</td>
</tr>
<tr>
<td>Transition zone</td>
<td>12-15</td>
<td>60</td>
<td>5</td>
<td>30</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>5×10^{-8}</td>
<td>4×10^{-8}</td>
<td>N/A</td>
</tr>
<tr>
<td>Sand layer</td>
<td>15-20</td>
<td>N/A</td>
<td>1</td>
<td>35</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>5×10^{-6}</td>
<td>5×10^{-6}</td>
<td>N/A</td>
</tr>
</tbody>
</table>
The consolidation parameters were determined from the field dissipation tests (DT), and the lab triaxial and oedometer tests; the field DTs were continued during CPT, but once the cone tip reached the desired depth, penetration was suspended and the pore water pressure behind the cone tip was recorded. The tests finished after 50% of the maximum generated pore water pressure had dissipated. To determine soil permeability from DT, $t_{50}$ is needed, and the method proposed by Sully et al.,(1999) was adopted. Once $t_{50}$ is determined, the $C_n$ can be determined as discussed in Literature review. A typical DT interpretation chart from site is shown in Figure 3.14.

![Figure 3.14 Interpretation of DT from CPT 102 at 13 m depth](image)

Oedometer tests were carried out on undisturbed Shelby tube samples according to AS 1289.6.6.1 (1998), and triaxial tests were carried out on undisturbed Shelby tube samples according to AS 1289.6.4.2 (2016). Field vane shear tests were
carried out at each CPT location within the soft clay layer, and the peak and residual strengths were obtained. Undisturbed U50 Shelby tube samples were collected at each bore hole with a 1.5 m interval in depth. It was noted by Bjerrum (1973) that the shear strength of clay assessed from shear vane testing varied with the plasticity index, and therefore the raw data from the vane shear tests must be corrected using the plasticity index, PI. Bjerrum also suggested the following correction to assess the undrained shear strength:

\[
c_u = [1.7 - 0.54 \log(PI)]s_v
\]

Equation 3-1

where \( PI \) is the soil plasticity index and \( s_v \) is Vane shear test results; charts are also available for this correction, as shown in Figure 3.15.

Excess pore water coefficient A at failure was also determined from the triaxial tests, and then used to check the proposed modified formula for predicting excess pore water pressure induced by pile driving.

![Figure 3.15 Field vane correcting chart (after Bjerrum, 1973)](image)
3.3.2 Design and construction of the field test

Two embankments were constructed at the site, embankment A was built on a foundation with PVDs and embankment B was built on natural ground. The ground was left to consolidate for 3 months before one pile was driven into each embankment. The proposed field test has 3 stages, as shown in Figure 3.16 to Figure 3.18.

Stage 1: After constructing an excess road a 300 mm thick working platform (WP) was laid on site on 22/10/2015; and then PVDs were installed where embankment A was to be built on the same day. Circular type of PVD was selected for this research. PVDs were installed in a triangular pattern with 1.5 m spacing and to a depth of 16 metres. This was followed by the installation of monitoring instruments on 26/10/2015. Piezometers were also installed at 4 locations, two at each embankment, approximately 2d and 5d from the centre of the embankment where “d” is the diameter of the Cylindrical hollow Steel (CHS) pile, which is 406 mm; 3 piezometers were installed at 2, 5, and 8 metres below the natural ground. Two inclinometers were installed on each embankment, roughly 2d and 5d from the centre of the embankment. According to ASTM (2013) the inclinometer casing should extend at least 5 metres below the expected zone of soil movement, and therefore the inclinometer casing was installed 6 metres below the level of the pile toe. Settlement plates were installed on top of the natural ground, two on each embankment, and again they are about 2d and 5d from the centre of the embankment. After installing the instrumentation, the first layer of the embankment (700 mm) was constructed on top of the WP on 28/10/2015 and then left untouched for 143 days; the 1m thick second layer of
the embankment was constructed on 17/03/2016. A sketch of stage 1 is shown in Figure 3.16, and the location of the instrumentation is shown in Figure 3.17
Stage 2: On 17/08/2016 one instrumented cylindrical hollow steel pile was driven into the centre of each embankment with a diesel hammer and then dynamic analyses of the piles were carried out. The instrumented piles used on site have a closed end; they are 406 mm in diameter, have wall thickness of 9 mm, and are 17 m long. The CHS was separated into 4 sections so that strain gauges could be attached to the inside surface of the piles at two ends of each section. Electronic resistance strain gauges were used on site. Silicone and Araldite were applied to the surface of the strain gauges as a water proof cover. The design of Instrumented piles is shown in Figure 3.18. Two sets of dynamic analysis were carried out on each pile; the first set occurred immediately after pile driving and the second set occurred 3 hours after pile driving.
Figure 3.18 Design of instrumented pile

406 mm OD, 9 mm thick CHS pile
Figure 3.19 Stage 2 of proposed field testing

Stage 3: After constructing the piles, the monitoring stage commenced according to the monitoring schedule introduced below.
3.3.1 Construction of the embankments and installation of the instruments

The access road and drainage layer were constructed with a 20t excavator. The drainage layer consists of poorly graded fine-grained sand. Following the placement of drainage layer, PVDs are installed at proposed location of embankment A. Then piezometers, inclinometer tubes and settlement plates are installed at locations shown in Figure 3.17. After the installation of the instruments, a mini excavator was used to construct the trench for cables and the embankments. Figure 3.20 to Figure 3.23 shows the embankment construction activities.

Figure 3.20 Setout PVD locations on drainage layer
Figure 3.21 Installation of piezometers
Figure 3.22 Installation of inclinometer tubes
The stain gauges are installed on the inner shaft of CHS piles, and the piles were driven into the ground with a diesel hammer. Same hammer was used for PDA tests. Figure 3.24 to Figure 3.27 shows the activities for piling activities.
Figure 3.24 Install strain gauges

Figure 3.25 Pre-drill before pile installation
Figure 3.26 Welding CHS sections on site
3.3.2 Proposed monitoring schedule

The generation and dissipation of pore water pressure are crucial indicators of the behavior of foundation soil. Compared to the excess pore water pressure generated by surcharge due to the self-weight of embankments, the excess pore water pressure incurred by piling is much higher, and the rate of generation and dissipation of pore water pressure is much more rapid in the piling case than in the embankment construction case. For the reasons mentioned above, two monitoring intervals were set for the piezometers. During embankment construction phase, one piezometer reading was logged every hour and in the
piling case, readings were taken every two minutes, the highest reading frequency acceptable by the system.

The lateral movement of soil is also an essential part of foundation behavior, especially during and after piling. The lateral movements of soil are measured by inclinometers. The reading interval is generally about two weeks. However, before, immediately, and one day after piling, one set of inclinometer readings were taken to capture the reaction of the foundation to piling in terms of lateral soil movement; the reading interval was then changed back to every two weeks.

Compared to excess pore water pressure and lateral soil deformation, the vertical settlement of foundation soil is mainly induced by surcharge loading from the embankment material, which is why settlement readings were taken more frequently after constructing the embankments; after two weeks of embankment construction, the frequency reverted to one reading every two weeks.

The strain gauges attached to the surface of the piles were monitored with a portable device which gives micro-strain of the pile shaft and automatically compensated for changes in temperature; readings were obtained every two weeks after the piles were installed.
3.4 Monitoring results of the field test

The monitoring results are presented in this section, but due to various incidents that occurred during construction some instruments were damaged so the results from these damaged sensors are not presented. Moreover, some of the results from vacuum preloading embankments were used to back calculate the properties of soil on site and calibrate the test results from two embankments with CHS piles installed.

A brief introduction to the construction and monitoring of vacuum preloading embankment is given below. To be cost efficient, a vacuum preloading embankment is designed to be constructed together with pile-PVDs embankments. The site investigation, PVDs, and installation of instrumentation for these two projects were carried out simultaneously, but the complicity of vacuum preloading technology means that the vacuum preloading embankment is currently half finished. A 600 mm thick WP and a 900 mm thick sand blanket were placed over the ground, and then the pore water pressure and soil deformation results were back analysed. The current state of vacuum preloading embankments and pile-PVD embankments is shown in Table 3.4. A sketch of instrumentation location is shown in Figure 3.28.
Table 3.4 Construction status of vacuum and pile-PVD embankments by December 2016

<table>
<thead>
<tr>
<th>Project</th>
<th>VP embankment</th>
<th>pile-PVD embankments</th>
</tr>
</thead>
<tbody>
<tr>
<td>Status</td>
<td>Under construction</td>
<td>Finished</td>
</tr>
<tr>
<td>Site investigation</td>
<td>Finished</td>
<td>Finished</td>
</tr>
<tr>
<td>Working platform</td>
<td>0.6 m thick, constructed on 20/10/2015</td>
<td>0.3 m thick, Constructed 21/10/2015</td>
</tr>
<tr>
<td>Instrumentation</td>
<td>constructed on 26/10/2015</td>
<td>constructed on 26/10/2015</td>
</tr>
<tr>
<td>First layer</td>
<td>0.9 m thick, constructed on 03/10/2016</td>
<td>0.7 m thick, constructed on 28/10/2015</td>
</tr>
<tr>
<td>Second layer</td>
<td>Not yet</td>
<td>1 m thick, constructed on 17/03/2016</td>
</tr>
<tr>
<td>Piling</td>
<td>N/A</td>
<td>Finished on 17/08/2016</td>
</tr>
<tr>
<td>Apply vacuum preloading</td>
<td>Not yet</td>
<td>N/A</td>
</tr>
</tbody>
</table>
3.4.1 Instrumentation details

Although maximum effort was given to ensure the instruments were installed at the intended location and depth, there was a discrepancy between the as built and design locations for most instrumentation. Table 3.5 summarises the design and as built locations for the instrumentation installed at two pile-PVD embankments.

Except for inclinometer 4, and settlement plates 2 and 4, all the other instrumentation was installed with an offset of no more than 0.1 m from the designed locations; how the locations of this instrumentation affected the monitoring results will be discussed in later sections.
Table 3.5 Designed and as-built location for instruments

<table>
<thead>
<tr>
<th>Instrument</th>
<th>Designed locations</th>
<th>As built locations</th>
</tr>
</thead>
<tbody>
<tr>
<td>Piezometer set 1 (P1)</td>
<td>0.8 m from centre of the pile</td>
<td>0.8 m from centre of the pile</td>
</tr>
<tr>
<td>Piezometer set 2 (P2)</td>
<td>2 m from centre of the pile</td>
<td>2.1 m from centre of the pile</td>
</tr>
<tr>
<td>Piezometer set 3 (P3)</td>
<td>0.8 m from centre of the pile</td>
<td>0.75 m from centre of the pile</td>
</tr>
<tr>
<td>Piezometer set 4 (P4)</td>
<td>2 m from centre of the pile</td>
<td>1.85 m from centre of the pile</td>
</tr>
<tr>
<td>Inclinometer 1 (I1)</td>
<td>0.8 m from centre of the pile</td>
<td>0.75 m from centre of the pile</td>
</tr>
<tr>
<td>Inclinometer 2 (I2)</td>
<td>2 m from centre of the pile</td>
<td>1.85 m from centre of the pile</td>
</tr>
<tr>
<td>Inclinometer 3 (I3)</td>
<td>0.8 m from centre of the pile</td>
<td>0.7 m from centre of the pile</td>
</tr>
<tr>
<td>Inclinometer 4 (I4)</td>
<td>2 m from centre of the pile</td>
<td>1.85 m from centre of the pile</td>
</tr>
<tr>
<td>Settlement plate 1 (SP1)</td>
<td>0.8 m from centre of the pile</td>
<td>0.9 m from centre of the pile</td>
</tr>
<tr>
<td>Settlement plate 2 (SP2)</td>
<td>2 m from centre of the pile</td>
<td>2.46 m from centre of the pile</td>
</tr>
<tr>
<td>Settlement plate 3 (SP3)</td>
<td>0.8 m from centre of the pile</td>
<td>0.8 m from centre of the pile</td>
</tr>
<tr>
<td>Settlement plate 4 (SP4)</td>
<td>2 m from centre of the pile</td>
<td>2.35 m from centre of the pile</td>
</tr>
</tbody>
</table>
3.4.2 Settlement data from VP embankment

Figure 3.29 shows the settlement data measured from surface settlement plates; here the rate of settlement was rapid before 80 days post WP construction and then it flattened out, which suggests that primary consolidation finished around 80 days after the WP was constructed. An analytical prediction of the degree of consolidation after WP construction was made by adopting the method proposed by Hansbo (1981), where the degree of consolidation is calculated from:

\[ U = 1 - (1 - U_v) \times (1 - U_r) \quad \text{Equation 3-2} \]

Where \( U_v \) is the average degree of vertical consolidation calculated from 1-D consolidation theory, and \( U_r \) is the average degree of radial consolidation calculated from Equation 3-3.

\[ U_r = 1 - e^{-\frac{8T_h}{\ln\left(\frac{d_e}{d_w}\right) + \frac{K_h}{K_s} \ln\left(\frac{d_e}{d_w}\right)}} \quad \text{Equation 3-3} \]

Where \( T_h \) is the horizontal coefficient of consolidation, and \( d_e \) is the effective diameter of one vertical drain, i.e. diameter of influencing zone (soil cylinder); where vertical drains are installed in a square pattern at 1.2 m spacing,

\[ d_e = 1.128 \times 1.2 = 1.35 \text{ m} \]

\( d_w \) is the diameter of the vertical drain, which is 0.034 m in this case, and therefore \( \frac{d_e}{d_w} = 39.8 \). Indraratna et. al. (2012) suggested a value of 3 for \( \frac{d_e}{d_w} \) at BBP site, which was adopted in this paper. A detailed report on the vertical coefficient of consolidation \( c_v \) is given by Pineda et. al. (2016); it suggests that the \( c_v \) value is between 4 and 20 m²/year in soft clay. A conservative \( c_v \) value of 5 is selected.
and the ratio $\frac{c_z}{c_v}$ is assumed to be 2, and $\frac{k_h}{k_z}$ is assumed to be 3. The calculated degree of consolidation (DoC) is plotted together with measured settlement. Figure 3.29 shows that for the predicted and measured data, a 90% degree of consolidation was reached before 80 days after the placement of WP for the clay layer; this confirmed that the soil permeability value assumed in the theoretical and numerical analyses, and that any settlement which occurred 80 days after the construction of WP can be treated as secondary settlement by assuming that secondary consolidation only takes place after 90% of primary consolidation is finished.

Figure 3.29 Measured surface settlement data vs predicted DoC

Figure 3.30 shows the settlement measured at various depths from extensometers after 85 days post WP construction, and the predicted secondary consolidation settlements, where most of the settlements occurred between 5 to
The oscillation is potentially due to “cyclic loading” caused by saturated-dry cycle of crust and WP material; similar behaviour occurred in the lateral deformation data measured by the inclinometer. As Figure 3.31 shows, when the rainfall is less than evaporation, the working platform dries out, the vertical overburden stress decreases and renders unloading in the foundation soil. Hence the lateral movement of soil is negative. On the contrary, when the rainfall is more than evaporation, the bulk unit weight of the embankment and the foundation soil increases, applying additional load on the foundation, hence the lateral soil movement is positive.

![Figure 3.30 Measured and predicted secondary consolidation at various depths](image)

Theoretical secondary consolidation settlement is calculated based on Equation 3-4 suggested by Das (2007).
\[ S_s = H \frac{C_a}{1 + e_p} \log\left(\frac{t_2}{t_1}\right) \]

Equation 3-5

Where

- \( S_s \) is settlement due to secondary consolidation
- \( H \) is thickness of the soil layer
- \( C_a \) is coefficient of secondary consolidation
- \( e_p \) is void ratio at start of secondary consolidation
- \( t_2 \) is time at which secondary consolidation settlement is calculated
- \( t_1 \) is time at which secondary consolidation starts

Figure 3.31 Inclinometer reading and rainfall minus evaporation data
3.4.3 Pore water pressure data from VP embankment

Monitoring of the pore water pressure began 85 days after the working platform was in place. The pore water pressure data from all 11 piezometers and the rain fall data from nearby weather stations are plotted in Figure 3.32.

Note that the pore water pressure at every location fluctuated with the same pattern and a similar magnitude, while the daily rain fall data indicates that when there was a rain fall event the pore water pressure spiked. This is because piezometers measure the total pore water pressure (\(u_{\text{total}}\)) which consists of hydrostatic pore water pressure (\(u_{\text{static}}\)) and excess pore water pressure (\(u_{\text{excess}}\)). The GWT rises during rain fall events, which causes \(u_{\text{static}}\) to increase, hence the spikes in \(u_{\text{total}}\) readings from piezometers. The magnitude of these spikes was similar, except for 3 piezometers at 8 m deep which all experienced
down scaled oscillations. This observation suggests a change of marital of around 8 m BGL. Pineda et. al. (2016) indicated that a change of soil structure occurred around 8.5 m at one of the BHs. The retarded pore water pressure to rainfall events near 8 m BGL is possibly due to the soil having a lower permeability. The piezometers were surrounded by sand packs and the time needed to reach an equilibrium between $u_{\text{static}}$ within the sand pack and surrounding soil when GWT changes, is affected by the permeability of the surrounding soil. When the permeability of the soil around the sand pack was low, the change of $u_{\text{static}}$ inside the sand pack was delayed and therefore the retarded response. Since no dummy piezometer reading which would reflect the change of GWT is available, an alternative method is needed to correct the vibration of $u_{\text{total}}$. Base on the assumption that the fluctuation of $u_{\text{total}}$ is mainly due to a change of GWT, the accumulative daily rainfall minus evaporation data was plotted together with change of $u_{\text{total}}$ in Figure 3.33. The rainfall and evaporation data were obtained from Bureau of Meteorology (2017) at Ballina station. It is worth noting that these data cannot be directly related to any change in $u_{\text{static}}$ due to various uncertainties such as the catchment area, topographic conditions, and tidal influences.
The corrected $u_{total}$ is plotted in Figure 3.34, and suggests that the impact from the change of GWT on $u_{total}$ can be minimised by considering the average piezometer reading near 10 m BGL as “dummy” readings that represent the change of GWT. Figure 3.34 and Figure 3.35 show that after correcting, all the readings are approximately linear and stable, except for $u_{total}$ near 8 m BGL due to the reason discussed earlier, despite a slightly increasing trend which will be discussed later.
Figure 3.34 Corrected pore water pressure data

Figure 3.35 Change of pore water pressure after correction
Figure 3.35 shows the corrected change in $u_{total}$ versus time at 2 m and 5 m BGL and the increasing trend in pore water pressure is evident. From previous analyses, 90% of consolidation had finished around 80 days after constructing the WP; in other words, $u_{total}$ should decrease very slowly or remain unchanged. This increasing trend of $u_{total}$ is probably due to the secondary settlement of the piezometer tip.

As stated by Chu et. al. (2003), the pore water pressure needs to be corrected for settlement of the piezometer tip.

Figure 3.36 Change of pore water pressure measured as the piezometer settles during consolidation
Before the surcharge loading has been applied, $u_{\text{total}} = u_{\text{static}} = H \times \gamma_w$, where $H$ is the distance from the tip of the piezometer to GWT and $\gamma_w$ is the unit weight of water, immediately after the surcharge loading has been applied.

$$u_{\text{total}} = u_{\text{static}} + u_{\text{excess}} = H \times \gamma_w + h \times \gamma_s \quad \text{Equation 3-6}$$

Where $h$ is the height of surcharge and $\gamma_s$ is the unit weight of the surcharge, then primary consolidation takes place,

$$u_{\text{total}} = u_{\text{static}} + u_{\text{excess}}$$

$$= (H + S_p) \times \gamma_w + h \times \gamma_s \times (1 - U) \quad \text{Equation 3-7}$$

Where $S_p$ is the primary consolidation settlement at the level of the piezometer tip and $U$ is the degree of consolidation. As primary consolidation continues $S_p$ and $U$ increases, so $u_{\text{static}}$ increases while $u_{\text{excess}}$ decreases. This increasing $u_{\text{static}}$ is often undetected or ignored during primary consolidation because the rate of dissipation of $u_{\text{excess}}$ is generally much more rapid than the rate of increase of $u_{\text{static}}$ caused by settlement of the piezometer tips. However, when the compressibility and thickness of the consolidating layer is significant enough, the difference between the measured and calculated pore water pressure (residual pore water pressure) will be noticeable. As Figure 3.37 shows, if the measured pore water pressure is not corrected for settlement, the calculated $u_{\text{excess}}$ using $u_{\text{total}}$ subtract $u_{\text{static}}$ will be higher than the theoretical value. This is because the piezometer will pick up higher $u_{\text{excess}}$ as it settles, and the higher static pore water pressure contributes to higher $u_{\text{total}}$. Hence the higher calculated $u_{\text{excess}}$ with the assumption that $u_{\text{static}}$ is constant. In addition, a slight
increase of measured pore water pressure may be observed after majority of the excess pore water pressure has dissipated, which can be attributed to creep settlement.

However, once the degree of consolidation approaches unity and secondary consolidation commences, as normally accepted in practice, the remaining $u_{\text{excess}}$ is insignificant and can be ignored. Assuming that the piezometer still settles with the surrounding soil, $u_{\text{total}}$ increases due to developing secondary consolidation settlement ($S_s$), while consolidation theory predicts that no change should occur in the pore water pressure. As the following equation indicates:

$$u_{\text{total}} = u_{\text{static}} + u_{\text{excess}} = (H + S_p + S_s) \times \gamma_w + 0 \quad \text{Equation 3-8}$$
Figure 3.37 Comparison between total pore water pressure calculated from 1-D consolidation theory by considering and ignoring the settlement of the piezometer

A numerical analysis using PLAXIS 2D axisymmetric model was carried out to simulate the foundation soil under surcharge loading. The PVDs were converted to concentric drain walls, as stated in Indraratna et. al., (2008). The properties of soil used the in analysis are listed in Table 3.2 and Table 3.3. A soft soil creep model was used for the layer of soft clay to include secondary consolidation. To reflect the impact of settling piezometer tips on the total pore water pressure, updated mesh and pore water pressure options were activated during the consolidation phase. The settlement and total pore water pressure were compared to the measured values.

Figure 3.38 shows a comparison between numerically predicted and measured surface settlement at SP1 which is located at the centre of the test area. It can be concluded that the numerical analysis simulated the field settlement very well, except for the early stage of consolidation, which has been suggested by H.G. Poulos (1972) that the predictions on immediate settlement are less satisfactory than total settlement. In Figure 3.39 to Figure 3.41, the predicted and measured pore water pressure and settlement data 85 days post WP construction are plotted together for different depths, and show the influence of secondary consolidation on \( u_{total} \). The theoretical secondary consolidation was calculated from \( s_s = H \times \frac{c_u}{1+e} \times log(t - t_{90}) \) which assumed that secondary consolidation started when 90% of DoC was reached. Once \( s_s \) has been calculated, the change
of $u_{total}$ can be calculated as $u_{total} = s_s \times \gamma_w$. Note that the increasing $u_{total}$ measured by piezometers and settlements measured by extensometers matched the predicted values very well.

Figure 3.38 Comparison of predicted and measured settlement for SP1
Figure 3.39 Predicted and measured pore water pressure and settlement at 2 m BGL during secondary consolidation

Figure 3.40 Predicted and measured pore water pressure and settlement at 5 m BGL during secondary consolidation
Figure 3.41 Predicted and measured pore water pressure and settlement at 8 m BGL during secondary consolidation

Many other projects had similar observations of pore water pressure. Moh and Woo (1987) reported a field test program in Bangkok, Thailand where the site has a 10.5 metre thick layer of very soft to soft clay. The field test program used sand drains and surcharge preloading to consolidate the ground. The, 0.26 m diameter sand drains were installed in a triangular pattern at 2 m spacing. Settlement and pore water were measured. The subsurface settlement was measured with a Sondex settlement gauge installed at the crest of the embankment and the pore water pressure was measured under the centre of the embankment. No numerical analysis was reported in the paper.
Although the authors did not show the degree of consolidation at the end of the monitoring scheme, the settlement readings are quite stable 100 days after final loading which indicates high degree of consolidation. Four piezometer measurements, PP21A to D, were presented. PP21B was installed in a very soft layer of clay. There was approximately 10 kPa of residual pore water at 100 days after final loading in PP21B, whereas settlement at the same level under the centre of the embankment can be estimated as 0.6 m. In PP21A, which was installed near the boundary of curst, there was a 10 kPa difference between the pore water pressure 100 days after loading and the lowest pore water pressure can also be established. Here the pore water pressure kept decreasing even after 200 days of post embankment construction, and according to Moh and Woo (1987) this is due to an extensive extraction of underground water at the Bangkok area and nearby vacuum PVD testing activities. In other words, the measured pore water pressure, and hence the residual pore water pressure, would be even higher if no underground water pumping had occurred in this region.

Redana (1999) reported a test embankment at a naval Dockyard in Thailand. This site has approximately 12 m of very soft to soft clay (undrained shear strength less than 30 kPa). The vertical drains used in this project are sand wick drains which consist of a permeable fibrous hose filled with sand. Sand wick drains were installed in a square pattern at two different spacing: 1.5 m in the T1 area and 2.5 m in the T2 area. A comparison of the measured and predicted date of settlement and the excess pore water pressure are shown. The excess pore water pressure in both areas is higher than the predicted values, especially for the T1 area where the spacing of the sand wick drain is 1.5 m. The settlement in T1 area is higher
than the T2 area, which has a 2.5 m-spacing sand wick drain. In the T1 area, while the predicted excess pore water pressure had almost fully dissipated, there is a residual pore water pressure (7-8 kPa) in the measured data set. In the T2 area, due to larger drain spacing, the measured excess pore water pressure had only partially dissipated, and the measured rate of dissipation was slower than prediction. The difference between the measured and predicted pore water pressure can be attribute to the increase of hydrostatic pore water pressure due to the settlement of the piezometer.

Others such as Indraratna et. al., (1994) and Yan and chu, (2003) also reported projects in which residual pore water pressure can be found.

3.4.4 Pore water pressure data from pile-PVD embankments

The pore water pressure data shown in Figure 3.42 are the uncorrected piezometer readings beneath the pile-PVD embankments. As stated before, 4 sets of piezometers were installed, with 2 sets under each embankment. These pore pressures were influenced by precipitation and fluctuation in the ground water table. A similar technique for the VP embankment monitoring data is adopted here. The static pore water pressure measured by most piezometers is lower than expected, possibly due to installation errors, but the difference is generally less than 10 kPa and since the change of pore water pressure is the main concern here, the difference between the measured and predicted static pore water pressure after correction, is still acceptable.

There is no data recorded for WP and first list of embankment construction since the data logging system was not functional at that time. However, Figure 3.42
indicates that the response of pore water pressure was much more significant during piling (around 18/08/2016) than during the second lift of embankment construction (21/03/2016). Since the applied loads are similar during first and second lift of embankment construction, it is safe to conclude that pile driving generates much higher maximum access pore water pressure in the soil than surcharge loading does.

By comparing Figure 3.42 with Figure 3.43 to Figure 3.45, the change of pore water pressure due to embankment construction after correction, is much more pronounced.

Figure 3.42 Piezometer readings under pile-PVD embankments
Figure 3.43 Corrected piezometer readings at 3 m below the natural ground level

Figure 3.44 Corrected piezometer readings at 5m below the natural ground level
As discussed before, the generation and dissipation of pore water pressure due to vertical surcharge loading and piling are two different processes. When vertically loaded, the generated excess pore water pressure can be calculated using the well accepted method based on the change of stress in the soil and the pore water coefficients. Consolidation takes place while the total increment of stress remains unchanged and the direction of soil movement is the same as the applied load, in this case downwards. However, the current methods used to calculate the generated excess pore water pressure due to piling are either empirical or based on cavity expansion theory. Furthermore, these methods are quite case sensitive, so there are no general methods yet available that can easily be adopted in real projects. In terms of consolidation, soil movement after piling is mainly radial/horizontal and the direction of displacement (inwards) is opposite.
to the applied stress due to piling (outwards). At the same time the total applied stress decreases as the inward movement of soil develops.

Due to these fundamental differences, the pore water pressure data are separated into two phases: before and after piling. Pore water pressure data before piling was used to back calculate the properties of soil and validate the numerical models at a later stage, as shown in Figure 3.46. The data after piling was used to study the effect of PVD improvement on excess pore water pressure due to piling; the settlement data was divided in the same way.

![Figure 3.46 Corrected piezometer readings before piling](image)
Changes of pore water pressure due to second lift of embankment construction are shown in Figure 3.47, in which the influence of static pore water pressure is excluded. The increase of pore water pressure after a 1 m thick embankment was placed across the site was relatively consistent. Depending on the depth of the piezometer, the excess pore water pressure due to embankment surcharge ranged from 7 to 10 kPa.

Due to the construction sequence, the pore water pressure between the first and second lifts of embankment construction is not available. In Figure 3.48 to Figure 3.50, the degree of consolidation after the second lift of embankment construction shows that 90% of DoC was reached within 100 days of PVD being installed, whereas without PVD, less than 70% of DoC was achieved about 150 days after constructing the embankment, and before piling took place. The DoC plots confirm that:

1. The inclusion of PVD facilitated the dissipation of excess pore water pressure due to surcharge.
2. The horizontal permeability of clay at 8 m is probably lower than at depths of 5 m and 3 m.
Figure 3.47 Change of piezometer readings before piling

Figure 3.48 Degree of consolidation between the second lift and piling at 3 m below the natural ground level
Figure 3.49 Degree of consolidation between the second lift and piling at 5 m below the natural ground level

Figure 3.50 Degree of consolidation between the second lift and piling at 8 m below the natural ground level
After 153 days of consolidation when 90% and 50% of DoC was expected for embankment A and B respectively, piling was carried out at two embankments. On 16/08/2016 piling on embankment B was carried out after the ground was pre-bored to a depth of 1m. The first section of pile was lifted and placed inside the pre-bored hole, its inclination was checked, and a temporary casing was used to ensure the pile would be driven vertically during the first few blows. Once the pile reached a 2 m embedded length, the casing was removed, and pile was driven to the designated depth. Four sections were driven into the ground with a 1.12-ton diesel hammer. The sections were welded on site and two sets of dynamic analyses were carried out on the piles. At the end of the second PDA, the toe of the pile was driven to 15.85 m below the crest of the embankment (13.85 m below natural ground level) with a 1.15 m stick out. One day later, the same procedure was carried out when another pile was installed on embankment B, and the final stick out was also 1.15 m. Due to limited space between the pile and inclinometers 1 and 3, the sticking out part of inclinometer 1 and 3 casing were forced to move away from the piles, which created an unrealistic displacement later within the embankment and crust; the data are presented in section 3.4.5. Figure 3.51 shows the piezometer readings during, and one to two days after piling. Note that the maximum excess pore water pressure generated while the tip of the pile was driven past the piezometer depth; this was the first time the soil had been pushed aside as the pile was driven deeper into the embankment. The pore water pressure response was much less obvious during the restrike, which suggests that most of the excess pore water pressure was due to cavity expansion.
Figure 3.51 Measured pore water pressure due to piling

Figure 3.52 and Figure 3.53 show that with and without PVD, the pore water pressure measured by the piezometers closer to the face of the pile was higher than the pore water measured by piezometers further away from the pile during pile driving, which agrees with predictions made with cavity expansion theory.

To study the dissipation of excess pore water, the change in pore water pressure measured by each piezometer during and after piling was calculated as shown in Figure 3.54. The degree of consolidation after piling was based on the pore water pressure before piling and the maximum pressure reached during piling, at both embankments. Figure 3.55 shows a comparison between reading from piezometers about 2D and 5D away from the face of the pile respectively, with and without PVD. The DoC plots suggest that PVDs facilitated the dissipation of pore water pressure in a radial direction, however the influence of PVDs is not
significant within the first 3 hours after pile driving. The difference in terms of DoC with and without PVD was generally less than 40%. Moreover, at 3 m and 5 m below ground level, 70% of excess pore water pressure had dissipated within 1 day, and at a depth of 8 m depth, other than piezometer 4, 50% of excess pore water pressure had also dissipated within 1 day. It is therefore safe to say that the dissipation of excess pore water pressure due to piles is quite rapid initially, then slows down.

Figure 3.52 Measured pore water pressure due to piling with PVDs
Figure 3.53 Measured pore water pressure due to piling without PVD

Figure 3.54 Change of pore water pressure due to piling
Figure 3.55 Degree of consolidation after piling from piezometers about 2D from the pile

Figure 3.56 Degree of consolidation after piling from piezometers about 5D from the pile
3.4.5 Inclinometer data from pile-PVD embankments

Other than excess pore water pressure, the lateral movement of soil is also a product of piling activities. This section presents the results of inclinometer measurements taken before and after piling, in order to identify the effect on lateral soil movement from ground consolidation assisted by PVD.

As mentioned before, although the inclinometer reading within the crest layer was effected by the piling rig, Figure 3.57 shows there was less lateral movement of soil under the embankment A than embankment B due to piling at 2D and 5D away from the face of the pile. In fact, at 2D away from the pile the average lateral movement of soil under embankment A was approximately 25% less, and at 5D away from the pile there was approximately 33% less lateral movement.

Figure 3.58 and Figure 3.59 shows that the lateral movement of soil after piling is mainly inwards, with one exception, and most of this displacement occurred in the layer of soft clay; the exception mentioned above was inclinometer 3, which was 0.7 m from a pile driven into ground without PVD. There was outward displacement between 17/08/2016 and 24/08/2016, which then became inwards.

Lateral soil displacement along the pile below 2.5 m is shown in Figure 3.60 and Figure 3.61. Data above 2.5 m is not included because they were disturbed by casing and mast on the piling rig during installation. There is a distinct difference in the pattern for inclinometers placed 2D and 5D from the pile; there are 4 sections of soil movement for inclinometers 1 and 3 which are 2D away from the pile, and with and without PVD respectively, but there are only 3 sections for inclinometers 2 and 4 which are 5D away from pile. These changes are likely be caused by a change of shape at the toe of the pile where a conical tip was
attached to a cylindrical body; this change of shape has more influence closer to the pile.

Figure 3.57 Comparison of lateral movement of soil while piling in foundations with and without PVD
Figure 3.58 Lateral movement of soil after piling with PVDs
Figure 3.59 Lateral movement of soil after piling without PVD
Figure 3.60 Lateral displacement of soil along the pile with PVDs: (a) inclinometer 1 (b) inclinometer 2
Figure 3.61 Lateral displacement of soil along the pile without PVD: (a) inclinometer 3 (b) inclinometer 4
The change of pore water pressure and lateral movement of soil 1 day after piling are plotted together in Figure 3.62 to Figure 3.73, and show that the trend of pore water dissipation and the lateral movement of soil generally agree with each other, especially at approximately 2D from the pile. At approximately 5D from the pile the soil had deformed at a higher rate than the rate of pore water dissipation within 10 days of installing the pile, but then the rate of pore water dissipation exceeded the lateral movement of soil. Due to the time limit on completing this study, the last set of inclinometer readings were taken on 3/11/2016, about 80 days after piling; Figure 3.54 to Figure 3.56 show that 90% of DoC had been reached at every piezometer location.

![Figure 3.62 Change of pore water pressure and lateral deformation approximately 2D from pile and 3 m deep with PVDs](Image)

Figure 3.62 Change of pore water pressure and lateral deformation approximately 2D from pile and 3 m deep with PVDs
Figure 3.63 Change of pore water pressure and lateral deformation approximately 2D from pile and 3 m deep without PVDs

Figure 3.64 Change of pore water pressure and lateral deformation approximately 5D from pile and 3 m deep with PVDs
Figure 3.65 Change of pore water pressure and lateral deformation approximately 5D from pile and 3 m deep without PVD

Figure 3.66 Change of pore water pressure and lateral deformation approximately 2D from pile and 5 m deep with PVDs
Figure 3.67 Change of pore water pressure and lateral deformation approximately 2D from pile and 5 m deep without PVD

Figure 3.68 Change of pore water pressure and lateral deformation approximately 5D from pile and 5 m deep with PVDs
Figure 3.69 Change of pore water pressure and lateral deformation approximately 5D from pile and 5 m deep without PVD

Figure 3.70 Change of pore water pressure and lateral deformation approximately 2D from pile and 8 m deep with PVDs
Figure 3.71 Change of pore water pressure and lateral deformation approximately 2D from pile and 8 m deep without PVD

Figure 3.72 Change of pore water pressure and lateral deformation approximately 5D from pile and 8 m deep with PVDs
3.4.6 Dragdown force and skin friction acting on pile face

Figure 3.74 shows the calculated dragdown force acting on pile A and B due to the consolidation of soil around the piles. The dragdown forces were calculated using the following equations:

\[ F_d = A \times \delta \times E_{steel} \]  

Equation 3-9

\( E_{steel} \) is the modulus of the steel
\( \delta \) is the measured strain
\( A \) is the cross-section area of the CHS pile
\( F_d \) is the downdrag force

On pile A where PVD were installed to facilitate consolidation, the downdrag force acting on the face of the pile is less than on pile B, the maximum downdrag force
recorded 142 days after piling was around 12 kN, which is approximately 45% of the maximum downdrag force on Pile B.

Figure 3.74 Downdrag force and skin friction on pile A and pile B calculated from strain gauge readings

3.4.7 Measured surface settlement

Unlike the pore water pressure data which only captured the consolidation process after the construction of second embankment lift, the surface settlement data was available 43 days after constructing the working platform and before the
construction of first embankment lift. Figure 3.75 shows the surface settlement readings from SP1 to SP4, these readings indicate that settlement developed more rapidly with PVDs than without PVD, but the ultimate settlement for both cases is still comparable. The Asaoka method was used to estimate ultimate settlement, as shown in Figure 3.76 to Figure 3.77, and the degree of consolidation based on settlement data are shown in Figure 3.78 and Figure 3.79. Again, the settlement data were separated into two parts: before and after piling. Before piling the DoC with PVDs reached 90%, while without PVD the DoC reached 60 to 70%. After piling, and within 130 days, the DoC with PVDs reached 90% while the DoC without PVD reached 70 to 80%. The Asaoka method was described in detail in the literature review section. The DoC from the settlement data confirmed the previous observation that PVD can improve the rate of consolidation due to piling. However, when used to facilitate dissipation of piling induced excess porewater pressure, the effectiveness of PVDs is not as significant as when used in rapid dissipation of excess porewater pressure due to vertical loading.
Figure 3.75 Surface settlement reading
Figure 3.76 Ultimate settlement before piling by the Asaoka method
Figure 3.77 Ultimate settlement after piling by the Asaoka method
Figure 3.78 Degree of consolidation from settlement before piling

Figure 3.79 Degree of consolidation from settlement after piling
To study the relationship between settlement and dragload, Figure 3.80 to Figure 3.82 are plotted below. Both piles did not settle during the monitoring period since no loads were applied to the piles after installation. Therefore, it is reasonable to assume that the relative movement between pile shaft and the surrounding soil can be represented by the soil settlement itself. It is impractical to measure the soil settlement immediately adjacent to the pile shaft due to construction restraints, i.e. The settlement plates installed within an area of 0.7 m radius from the centre of the pile would obstruct the piling. Due to the above-mentioned reasons, in Figure 3.80 the surface settlement approximately 1.5D from the pile versus time is plotted with the maximum dragload versus time. Figure 3.81 shows the difference in settlement readings between sp1 and sp3 and the difference in dragload on pile A and B versus time; note that the dragload develop as settlement increases. Figure 3.82 is the maximum dragload plotted against settlement roughly 1.5D from the pile, with a linear approximation between the dragload and settlement, but the dragload should be linked directly to the relative pile-soil movement at the pile surface. However, as the settlement and dragload develop, this linear approximation is unlikely to be maintained.
Figure 3.80 Surface settlement approximately 1.5D from the pile and maximum dragload on the pile after piling.

Figure 3.81 Difference in surface settlement approximately 2D from the pile and the dragload on piles with and without PVD.
3.4.8 Pile Dynamic Analyse (PDA) tests results

Two sets of PDAs were carried out after piling; the first set took place immediately after piling and the second set 3 hours after piling. Pile B was driven first, and therefore the PDA took place first and then the PDAs on Pile A took place one day later. As Figure 3.83 shows, the initial shaft capacity predicted in PDA on pile B was approximately 40 kN higher than pile A. This suggests that the soft clay strength is different at two locations. The three-hours-restrike PDA test predicted a 7 kN and 14 kN increase in shaft capacity on pile A and pile B, respectively. It is assumed that in the increase of shaft capacity is due to strength gained from consolidation process. The average maximum excess pore water pressure measured near pile A and pile B during piling was 33 and 62 kPa, respectively.
Three hours after pile installation, the average dissipated pore water pressure along pile A and B was 13 kPa and 21 kPa respectively. It can be seen that the DoC achieved three hours after piling near pile A (with PVDs) was higher than the DoC near pile B (without PVDs), however, the average dissipated pore water pressure near pile A was less, compared to pile B. Since the strength gain of soft clay is directly linked to pore water dissipated not DoC, the PDA test carried out on pile B predicted a higher increase in shaft capacity.

Figure 3.83 PDA results
3.5 Summary

Three test embankments were constructed at NFTF, Ballina, NSW. One embankment (VP embankment) with 40 m diameter was built for study of vacuum surcharge and preloading with PVDs, the other two embankments (pile embankments) were built for study of interaction of Pile and PVDs in soft ground. Pore water and settlement data at early stage of VP embankment was used to validate the soil parameters obtained from previous and current site investigation results. From pore water monitoring date at VP embankment, it was concluded that there was a similarity between pore water measurement over time and accumulative rainfall minus evaporation over time. The accumulative rainfall minus evaporation data obtained from Australian Government, Bureau of meteorology web site was successfully used to calibrate pore water measurement against perception. It was noticed that at end of primary consolidation, the pore water pressure readings under VP embankment had a slight increasing trend. It was due to the soil settlement under secondary compression.

The pore water pressure readings under pile embankments indicated that the excess pore water pressure due to piling was much higher than preloading of 2 m high embankment. The excess pore water pressure during piling was mainly caused by cavity expansion and reduces as distance from pile shafts increases. Within the area where PVDs were installed to facilitate consolidation, the excess pore water pressure during piling was less than its counterpart with the area where no PVDs were installed. The inclusion of PVDs also accelerated the radial dissipation of pore water.
The inclinometer data suggests that improving the soft soil by preloading can reduce later soil deformation during pile installation. With the assistance for PVDs, a higher degree of consolidation was achieved, therefor, less lateral deformation was observed.

PVDs effectively accelerated pore water dissipation after embankment placement. Which reduced the post piling ground settlement, consequently reduced the negative skin friction after pile installation.
CHAPTER 4. SMALL SCALE LABORATORY MODEL TESTS

A series of model tests were carried out in the laboratory to investigate the generation and dissipation of pore water pressure due to piling, with and without PVD. The consolidation stress applied on the clay sample before piling was altered to study the impact of different pre-consolidation pressure and void ratios on the clay sample.

Details of the test apparatus, samples, procedure, and results are provided in this chapter.

4.1 Background of laboratory model test

Wood (2003) introduced the most commonly used modelling methods in geotechnical engineering. According to him, one type of modelling that geotechnical engineers and researchers rely on, in both cutting edge studies and state of practise projects, is physical modelling. Full scale field model testing and small-scale laboratory model testing both count as physical modelling. In Chapter 3, full scale field model testing was presented. Full scale model tests are widely used in projects, such as test and trial embankments, because they are the most reliable way to check the foundation design and to back analyse the soil parameters. In some case, full scale tests are not just optional, but a requirement by various codes. For example, pile load testing is a compulsory large-scale model test required by many standards to check the achievable geotechnical capacity of piles on site against the design values. The same philosophy is used in other geotechnical projects such as retaining structures (soil nail pull-out tests), shallow foundations (plate bearing tests), and trial compaction tests, etc.

Although the results from large scale field tests can simulate the foundation
behaviour in the most representative way, the spatial and time variation of parameters and factors cannot always be included in one set of full-scale field model test. To consider spatial and time variations, geotechnical practitioners often chose to increase the number of tests, such as pile load tests and soil nail pull-out tests. However, the relatively high cost to conduct the tests and long period of time to finish the tests in some case, trail embankment for example, mean that the tests are generally unrepeatable for each research or industrial project. Therefore, alternative approaches need to be taken, such as

- Design the test to consider worse case scenarios;
- Try to duplicate the model in small scale laboratory tests by controlling and altering the parameters/factors

Even though small-scale model testing cannot always replicate field conditions, especially with in-situ soil composition, and the stress history, it is much more cost effective and less time consuming therefore can be repeated under controlled conditions. Many researchers use model pile testing to predict the full-scale behaviour of piles in sand (Robinsky & Morrison, 1964, Lehane, et al., 1993, White & Bolton, 2004) and in clay. Table 3.2 is a brief summary of some of the work (Lo and Stermac, 1965, Clark & Meyerhof, 1972, Azzouz & Lutz, 1986, Coop & Wroth, 1989, Horvath, 1995), which were carried out in field and in laboratories; nevertheless, they followed the same methodology of controlling most of the variables while trying to vary only one target factor.

Other than model piles, PVDs are often studied in laboratory; Bergado et. al., (1991) studied the smear effect of vertical drains on soft Bangkok clay, while Indraratna and Redana (1998a), and Sharma and Xiao (2000) carried out
laboratory testing to determine the property of the smear zone. Indraratna et al., (2013) also studied the vacuum preloaded PVD assisted radial consolidation in the laboratory. All these studies have been successfully applied to full scale field condition to some extent.
Table 3.3 Brief summary of previous laboratory/field study on model piles in clay

<table>
<thead>
<tr>
<th>Soil type</th>
<th>Pile type</th>
<th>D (mm)</th>
<th>L (mm)</th>
<th>Data collected</th>
<th>Reference</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft to firm silty clay (Field)</td>
<td>Closed-end steel tube floating pile</td>
<td>90</td>
<td>13000</td>
<td>Excess pore water pressure during and short after the installation of pile</td>
<td>Lo and Stermac, 1965</td>
</tr>
<tr>
<td>Soft to firm silty clay (Laboratory)</td>
<td>Closed-end steel pile</td>
<td>76</td>
<td>762</td>
<td>Soil displacement near pile, Excess pore water pressure during pile driving, stress around and below pile</td>
<td>Clark &amp; Meyerhof, 1972</td>
</tr>
<tr>
<td>Soft to firm clay (Field)</td>
<td>Closed-end steel pile</td>
<td>39</td>
<td>-</td>
<td>Axial load, pore water pressure and horizontal earth pressure</td>
<td>Azzouz &amp; Lutz, 1986</td>
</tr>
<tr>
<td>Firm to stiff clay (Field)</td>
<td>Closed-end steel pile (IMP)</td>
<td>80</td>
<td>1135</td>
<td>Excess pore water pressure during pile driving, stress around pile</td>
<td>Coop &amp; Wroth, 1989</td>
</tr>
<tr>
<td>Soft pottery clay (Laboratory)</td>
<td>Closed-end steel pile</td>
<td>12.7</td>
<td>508</td>
<td>Load settlement relationship under different loading rate</td>
<td>Horvath, 1995</td>
</tr>
</tbody>
</table>
4.2 Apparatus and test procedure

A small-scale lab model test was designed to investigate the pore water pressure in soft clay during and after piling. To do this, Kaolin clay slurry was consolidated to represent natural soft clay deposits; the water content was controlled when mixing Kaolin clay with water in a large-scale electrical mixer. The capacity of the mixer is above 200 kg, however, to ensure an evenly distributed water content throughout the mix, maximum 120 kg of kaolin powder and water are allowed in each mix. The minimum time for mixing is 3 hours and then the mixed slurry was transferred into a large scale consolidometer cell for consolidation; different stresses were used during consolidation. After the degree of consolidation reaches 90%, an aluminium tube with a closed end was driven into the centre of the consolidated Kaolin clay by an electronical motor to simulate piling, and the pore water pressure was measured by piezometers at various distances from the tube during and after piling. A data acquiring system is used to record the pore water pressure measurement, the monitoring of pore water pressure continues until 90% of the excess pore water pressure at all piezometers locations are dissipated.

4.2.1 Test apparatus

The consolidation cell needs to serve several purposes in the tests:

- Acts as an oedometer to consolidate the slurry under the desired pressure.
- Has a suitable diameter and length to minimise the boundary effect in the model tests.
- Maintain the vertical stress applied during piling to simulate the real piling process when the embankment load is maintained
- Has the option to remove vertical stress during piling to simulate the case where the foundation soil is over-consolidated.

An existing larger scale consolidometer at UOW has been modified and used in this research. A schematic of the modified cell is given in Figure 4.1

![Figure 4.1 Test apparatus](image)

Figure 4.1 Test apparatus

The cell has an inside diameter 450 mm and is 950 mm high. There is a steel capping plate on top of the cell with a rubber ring to seal the cell. A piston with an air chamber is used to apply pressure to the capping plate and to the Kaolin clay sample. The piston is driven by compressed air and can apply up to 800 kPa of pressure. The piston has a 150 mm stroke that can be extended by adding
spacers between the piston and steel cap. The piston has an inner ring at the centre and the steel cap has a removable fitting which can be opened to allow the aluminium tube to pass through while stress is constantly applied to the sample as the tube is driven into the clay. The drainage conditions can be controlled by opening and closing the valve on the top and bottom drainage tubes. In this research, two-way drainage in a vertical direction is allowed.

4.2.2 Soil used in tests

Soils used in the tests include Kaolin clay and coarse grain sand. Sand is placed in layers above and below the layer of Kaolin clay and the sand is separated from the Kaolin layer with a geofabric filter which stops particles of sand entering the clay while allowing water to pass through.

4.2.2.1 Kaolin clay

In the model test, very soft to soft (12.5 < Cu <25 kPa) clay is needed to simulate foundation material in which the aluminium tube is going to be installed. Kaolin clay is used in model tests for the abovementioned purpose due to several reasons. First advantage of use Kaolin clay is that, compared to nature soil samples, with proper preparation, Kaolin clay samples are rather homogeneous, which makes the analyses of the test results more straightforward. Moreover, Kaolin clay has low sensitivity, and it has controllable, repeatable, and rather consistent engineering properties. It is also relatively easy to ensure the quality of kaolin samples due to its ease of saturation and good uniformity. Last but not least, it is cost effective and less time consuming to prepare Kaolin samples compared to prepare undisturbed or reconstituted field samples.
The specific gravity and Atterberg limits of the Kaolin clay used in this research were tested and are listed in Table 4.1.

Table 4.1 Specific gravity and Atterberg limits of Kaolin clay used in the test

<p>| | |</p>
<table>
<thead>
<tr>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Plastic Limit (%)</strong></td>
<td>27</td>
</tr>
<tr>
<td><strong>Liquid Limit (%)</strong></td>
<td>54</td>
</tr>
<tr>
<td><strong>Plasticity Index (%)</strong></td>
<td>27</td>
</tr>
</tbody>
</table>

To ensure the Kaolin slurry is thoroughly mixed, the water content is twice the liquid limit, which is about 110%. Immediately after mixing, the void ratio of Kaolin slurry was around 3 and was very difficult to handle due to its softness. Therefore, the slurry was placed into the consolidation cell and consolidated under 50 kPa pressure. After the void ratio decreased to about 2, samples are taken from the mix and oedometer tests were carried out on the samples. The typical results of oedometer tests on Kaolin and undisturbed field samples are shown in Figure 4.2. The compression index (Cc) and recompression index (Cr) of Kaolin clay from the test is approximately 0.9 and 0.1.
The permeability of the clay sample is a crucial factor in the vertical and radial consolidation process. Therefore, it is one of the key parameters need to be determined. Vertical permeability can be back calculated from the oedometer or triaxial tests. To do that, the coefficient of consolidation ($C_v$) is first calculated from the settlement vs time plot using either the log time method or the square root time method. When a complete settlement vs time curve is available, that is, when immediate settlement and the primary and secondary consolidation stages have been identified, the log time method is often used. However, when the data of the latter part of primary and secondary consolidation are not available, the square root time is often used. Although the square root time requires less data, at least 50% of primary consolidation must be finished and recorded to enable the square root time method to be used. The log time method is used to analyse
the consolidation test data in this research, with a typical analysis being shown in Figure 4.3.

![Figure 4.3 Log time method to obtain t₉₀](image)

Figure 4.3 Log time method to obtain t₉₀

Once t₉₀ is obtained, CV can be calculated from

\[ CV = \frac{t_{90} \times H_d^2}{t_{90}} \]

**Equation 4.1**

where \( t_{90} \) is a time factor when 90% consolidation has been reached, it is taken as 0.848.

\( H_d \) is the drainage length, which is equal to half of the initial sample site.

Once CV is calculated, the permeability (\( k \)) of the sample can be determined from well accepted relationship

\[ k = CV \times y_w \times m_V \]

**Equation 4.2**
where $\gamma_w$ is unit weight of water and $m_V$ is volume compressibility of the sample that is generally considered to be stress dependent.

Moreover, permeability is considered to be the void ratio dependant. Tavenas et al., (1983) proposed a relationship between the horizontal permeability and void ratio.

\[ e = e_0 + C_k \times \log \left( \frac{k_h}{k_{hi}} \right) \quad \text{Equation 4.2} \]

Where $e_0$ and $k_{hi}$ are the initial void ratio and initial horizontal permeability, respectively.

$C_k$ is the permeability index, which is generally considered as independent of the stress history, as investigated by Nagaraj et al., (1994) on over-consolidated clay. Al-Tabbaa and Wood (2015) published a series of lab tests on the permeability of Kaolin and found relationships between the permeability and void ratio and based on the relationship, the calculated permeability of Kaolin clay was between $2.5 \times 10^{-9}$ m/s to $7 \times 10^{-9}$ m/s for a void ratio ranging from 1.5 to 3; this is within the range reported in literature.

4.2.2.2 Sand

A layer of sand is placed above and below the layer of clay in the consolidation cell for the following reasons:

1. To make sure the stress applied by the piston is distributed evenly over the clay sample
2. To separate and prevent clay particles from entering and blocking the drainage outlets
3. To confine the clay layer during tube jacking and hence minimize the chance of soil escaping through the inner ring at the centre of the piston.

The sand used in the tests is poorly graded coarse sand; the particle size distribution test of the sand is shown in Figure 4.4. More than 90% of the sand particles are between 0.63 and 2 mm in size.

![Figure 4.4 PSD test results of sand sample used in the test](image)

4.2.3 Test procedures

The procedures used to test the pore water pressure generation and dissipation in soft clay due to tube jacking are described below.

4.2.3.1 Preparation of Slurry

Two bags of commercially available Kaolin powder (25 kg per bag) were mixed with 60 kg of water using an electronic mixer for at least 3 hours. To ensure a thorough mix, the Kaolin powder and water were added into the mixer 3 times,
and no more than 20 kg of powder and the corresponding amount of water were added each time. The finished Kaolin slurry has a moisture content of 110% to 120%; two mix of slurry, approximately 200 kg in total, were needed for one test.

4.2.3.2 Cell preparation

The main body of the consolidation cell consists of a Base plate and two half circular shells. The two shells sit in a slot on the base plate and are connected to each other with bolts and nuts on the sides. Silicone sealant was applied to the contact areas between the two half shells and between the shells and the base plate, and a layer of Vaseline was applied to the internal surface of the consolidation cell to minimise friction between the surface of the cell and the clay sample during consolidation.

4.2.3.3 Sample placement

When the cell was ready, sand was poured into the cell until it forms a 50 mm thick layer, which is then levelled with a brush and compacted with a steal rod which has a rubber plate attached to the head. Water was added into sand layer to achieve a moist content near 35%. A geofabric filter was then saturated and placed on top of the sand and Kaolin slurry is placed on top of the filter in 150mm thick layers, which are gently stirred and compacted with a steal rod to remove as many voids as possible, until a height of approximately 800 mm is reached. Another geofabric filter was saturated and laid on top of the slurry and a 50 mm thick layer of sand is placed on top, again with water added to achieve a 35% moisture content; the final step is to seal the top of the cell with the capping plate and place the piston on top.
4.2.3.4 Piezometer installation and pre-tube jacking consolidation

After installing the sample, piezometers were inserted into the slurry at designated position through holes already drilled into the cell. The piston was then pressurised with compressed air. There were two stages of consolidation before tube jacking; in stage one, 50 kPa pressure was applied, the piezometers were monitored, and the cell was drained from the top and the bottom. Once 90% of excess pore water pressure had been dissipated the piston was removed and a 150 mm thick spacer is placed between the cap and the piston. The second stage of consolidation begins with varying pressure, and depending on the pressure applied, more spacers were added as necessary. The second stage of consolidation also finishes when 90% of excess pore water had been dissipated.

4.2.3.5 Tube jacking

When the second stage of consolidation is complete, the fitting on the cap was removed and a 52 mm diameter by 1.5 m long aluminium tube with one end closed was jacked into the cell with an electronic motor through the centre of the piston and the hole at the centre of the cap. The rate of jacking can be approximately controlled by the output power of the motor, but in this research, how the rate of jacking affected the pore water in the Kaolin clay was not investigated, so the rate of jacking was fixed at 4 - 5 mm/second.

4.2.4 Test program

Four tests were carried out with different consolidation stresses before tube jacking; three tests were designed to investigate the influence of pre-
consolidation on pore water pressure during tube jacking, and one test was
designed to examine how the vertical drain affected the dissipation of pore water
pressure due to tube jacking. Table 4.2 is summary of the test program. In test 4,
10 mm diameter vertical drains were placed inside the cell before Kaolin clay.
The vertical drains used had a plastic core wrapped around with geofabric. The
cross section of the vertical drain was semi-circular with a radius of approximately
5 mm. The vertical drain was attached to a metal stick (<2 mm diameter) with zip
ties before placed inside the cell. The metal sticks were inserted into the sand
layer at the bottom of the cell and supported with steel wires on top. The sticks
were used to fix the vertical drains in place and prevent the vertical drains from
deforming during slurry filling and were withdrawn after slurry filling. The layers
of material, location of the piezometers, and placement of the vertical drain are
shown in Figure 4.5.

Table 4.2 Small scale lab model test program

<table>
<thead>
<tr>
<th>Test ID</th>
<th>Stage 1 consolidation pressure</th>
<th>Stage 2 consolidation pressure</th>
<th>With PVDs</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>50 kPa</td>
<td>100 kPa</td>
<td>No</td>
</tr>
<tr>
<td>2</td>
<td>50 kPa</td>
<td>150 kPa</td>
<td>No</td>
</tr>
<tr>
<td>3</td>
<td>50 kPa</td>
<td>200 kPa</td>
<td>No</td>
</tr>
<tr>
<td>4</td>
<td>50 kPa</td>
<td>200 kPa</td>
<td>Yes</td>
</tr>
</tbody>
</table>
4.2.5 Test results

After the pore water pressure due to tube jacking had dissipated, samples were extracted close to the piezometer for further tests. A Pocket Penetrometer was
used to determine the undrained shear strength of the samples. The water content of the samples was tested so that the void ratio of the samples could be calculated.

The undrained shear strength and void ratio are listed in Table 4.3, together with the shear modulus of one sample used in test 1, as determined from the triaxial test. The $G/Su$ ratio for all the test samples in this research was assumed to be constant.

Table 4.3 Properties of kaolin sample after pre-jacking consolidation

<table>
<thead>
<tr>
<th>Pre-jacking consolidation stress (kPa)</th>
<th>$Su$ (kPa)</th>
<th>Void ratio</th>
<th>Shear modulus (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>100</td>
<td>17</td>
<td>1.4</td>
<td>N/A</td>
</tr>
<tr>
<td>150</td>
<td>25</td>
<td>1.2</td>
<td>N/A</td>
</tr>
<tr>
<td>200</td>
<td>31</td>
<td>1.08</td>
<td>940</td>
</tr>
</tbody>
</table>

4.2.5.1 Maximum pore water pressure generated due to tube jacking

In Figure 4.7 the void ratio after pre-jacking consolidation is plotted against pre-jacking consolidation stress in a logarithmic scale; the figure shows that an approximately linear relationship can be established and the void ratio under 1 kPa of pre-jacking consolidation stress can be extrapolated as $e_1$, which is 3.3 in this case.
The author concluded that the maximum pore water pressure due to tube jacking was influenced by the void ratio of the clay sample before the tube was installed; Figure 4.7 was generated to examine this relationship. The maximum pore water pressure generated was normalised by the undrained shear strength of the soil sample, and the void ratio of the soil sample before the tube was installed was normalised by the void ratio being extrapolated under 1 kPa of consolidation stress. The distance between the piezometer and the centre of the tube was normalised by the radius of the tube, and therefore:

1. Less pore water pressure was generated further away from the tube
2. Less pore water pressure was generated in the sample with a lower void ratio
Figure 4.7 indicates that the pore water pressure generated due to tube jacking was affected by the void ratio of the soil. Randolph et al., (1997) proposed that the pore water pressure generated by piling is a function of $G/S_u$ and $r/r_0$ only. However, based on field and laboratory tests, the author concludes that the pore water pressure due to piling was also affected by void ratio of the clay, as well as the pore water coefficient $A$ and $G/S_u$ and $r/r_0$. The following empirical equation is proposed by the author:

$$\frac{u}{S_u} = 2 \times \ln \left( \frac{G}{S_u} \times \left( \frac{r_0}{r} \right)^{1.2} \times \frac{A_f}{2} \right) \times \alpha_u$$

Equation 4.30

Where $u$ is the maximum pore water pressure generated in soil due to piling.
Su is the undrained shear strength of the soil

G is the shear modulus of the soil

\( r_0 \) is the radius of the pile

\( r \) is the radial distance from the centre of the pile to a point in the soil

and

\[
\alpha_u = 1.15 \times ln\left(\frac{e_n}{e_1}\right) + 1.35
\]

Equation 4.30

\( e_n \) is the void ratio of the clay before piling

\( e_1 \) is the void ratio of clay under 1 kPa consolidation stress

The pore water pressure due to tube jacking is plotted with the theoretical predictions in Figure 4.8, because the proposed equation fits the lab test results better.
4.2.5.2 Dissipation of pore water pressure after tube jacking

Another important aspect of the pore water response is its dissipation after piling. In the laboratory model test, the dissipation of pore water pressure was monitored by piezometers until less than 5% of excess pore water pressure can be measured. Figure 4.9 to Figure 4.12 show the dissipation of pore water pressure in 4 tests where a time factor $T_r$, similar to $T_v$ in 1D consolidation theory, was used as the x-axis.

$$T_r = \ln \left( \frac{C_h \times t}{r_0^2} \right)$$  \hspace{1cm} \text{Equation 4.30}

Where $C_h$ is the horizontal coefficient of consolidation of the soil sample

t is the time elapsed since consolidation

$r_0$ is the radius of the pile
Figure 4.9 Pore water pressure after the tube jacking in test 1

Figure 4.10 Pore water pressure after the tube jacking in test 2
Figure 4.11 Pore water pressure after the tube jacking in test 3

Figure 4.12 Pore water pressure after the tube jacking in test 4
All the pore water dissipation curves from 4 tests are similar in that there is an initial stage where \( u \) decreases gently as \( T_r \) increases, and then there is a much steeper slope, and as the end of consolidation approaches the slope flattens out again. Figure 4.13 shows the degree of consolidation from piezometer readings in tests 1 to 3. Only piezometer 1 (2d from the centre of the tube) and piezometer 2 (4d from the centre of the tube) are included. Since the pore water pressure in piezometer 3 (8d from the centre of the tube) was low, the calculated DoC oscillates heavily. Figure 4.13 also illustrates that there are three phases of pore water dissipation. the first phase spanning between -8 to -2 in terms of \( T_r \), ended when approximately 10 % of DoC was reached; the second phase spans between -2 to 1 in terms of \( T_r \), with DoC ranging roughly from 10% to 90%, and the third phase commenced after reaching 90% of DoC and finished at 100% DoC with a \( T_r \) spanning between 1 to 3. Figure 4.14 shows a comparison of DoCs with and without vertical drains. There is no obvious difference between the two tests before reaching 60% DoC, but with the vertical drain, the pore water dissipated faster after reaching 60% DoC. There was a 5% to 10% higher DoC in test 4 (with a vertical drain) compared to test 3 (without a vertical drain) after 60% of excess pore water pressure had dissipated in both tests, until 90% of excess pore water pressure had dissipated in test 4.
Figure 4.13 Degree of consolidation versus time factor
4.3 Summary
A series of laboratory model tests were carried out to study the generated pore water pressure due to pile installation. The soft soil sample was modelled by pre-consolidated Kaolin slurry; the pile was modelled by an aluminium tube with closed end; and the pile installation process was modelled by jacking the aluminium tube into the soft soil sample with an electronic motor. Before tube was jacked into the soft soil sample, the Kaolin slurry was pre-consolidated with an air-pressure driven piston. The pre-consolidation pressure was varied in each test (100 kPa, 150 kPa and 200 kPa). The pore water pressure was monitored during and after tube jacking. The pore water pressure results show that the generated excess pore water pressure during tube jacking has a relationship with pre-consolidation pressure. With increased pre-consolidation pressure, excess pore water pressure generated during tube jacking decreased. Further study proposed
a method to calculated excess pore water pressure at a distance from pile surface based on void ratio of soil with which a pile is installed.

The impact of vertical drain on radial dissipation of pore water pressure was also studied by place 6 PVDs around the tube with a 190 mm distance from the centre of the tube. The pore water pressure monitoring results after tube jacking suggested that PVDs can facilitate dissipation of excess pore water pressure. Under the same pre-consolidation pressure, the DoC reached in the test that included PVDs is approximately 10% - 20% higher than the test that did not include PVD.
CHAPTER 5. NUMERICAL MODELLING

The use of numerical analyses is increasing in contemporary geotechnical research and projects. For research, numerical modelling can confirm hypotheses at relatively low costs and carry out parametric studies in short time frames, whereas physical models are far more expansive and time consuming; numerical modelling is also the most widely used method in case studies. For geotechnical engineering projects, numerical analyses can be used to validate methodology, parameters and construction sequences adopted in design, and serve as a tool for geotechnical-structure-interaction design projects which are too complicated for analytical solutions. The complexity of numerical models can be adjusted to cater different needs. While numerical results from a simple and straightforward model may be used as a reference check for design, comprehensive numerical analysis with advanced constitutive models can form the basis of designs, as well as the main source of confidence for engineers for projects with high complexity and significance.

5.1 Background of numerical analyses in this study

The numerical analyses applied in this research are via the commercial software package “PLAXIS”, one of the most widely accepted software used in geotechnical engineering projects and researches. It was initially developed by Technical University of Delft in Netherlands, but since the 1980s, numerous researchers have improved and upgraded the package. The PLAXIS version AE released in 2014 was used in this study.

Numerical analyses are used for the following tasks in this research:

i. To back calculate the soil parameters from field/lab test results
ii. To verify the proposed consolidation model
iii. To verify the proposed pile dragdown force model
iv. To perform parametric studies on dragdown forces

In PLAXIS AE 2D, there are two basic modelling options: Plane strain and axisymmetric, axisymmetric models are used in this research because they simulate the problem of concern better. The above-mentioned tasks are discussed in sections 5.2 - 5.4 in the same order listed above.

5.2 Soil parameters used in numerical modelling
The critical step in designing a foundation system or predicting its behaviour is to determine the soil parameters used in the analyses. Soil parameters are generally obtained by one or a combination of the following methods:

a. Local experience
b. Empirical correlations
c. In-situ tests
d. Lab tests on disturbed/undisturbed samples
e. Back analyses on field trails/lab models

Of these methods, a and b can only give a rough estimation of the parameters; c and d are more reliable, however, the accuracy of the parameters obtained cannot be guaranteed due to spatial variations, disturbance in the samples, and differences in the stress conditions tested for and under real project loads. Therefore, after the design parameters are obtained, it is recommended that the parameters should always be calibrated or adjusted based on back analyses. In most cases, back analyses include both hand calculation based on analytical solution and numerical modelling. In this research an initial estimation of
parameters is obtained from in-situ and lab tests, numerical models are then built with these parameters. The results of numerical analyses are compared with measurements taken on site and parameters are adjusted accordingly. Due to time and budget constraints, only limited pore water pressure data were collected during the laboratory model testing. Therefore, the numerical model was validated by field data before parametric studies are carried out.

The soil parameters determined from in situ/lab tests for field model test were presented in Section 3, together with the back analyses of measurements from the vacuum preloading embankment. It was noted that there were some discrepancies between measured and predicted data. Therefore, the parameters are adjusted accordingly shown in Table 5.1. The modelling results with updated parameters fit the measured data better. The validation process is described below in section 5.3.1.

5.3 Numerical modelling of consolidation under vertical stress
As mentioned before, consolidation around a pile due to vertical stress and installation are two distinct processes, so they are discussed separately in this section. The parameters used in the PLAXIS model are listed in Table 5.1. The Mohr-Coulomb model was used for fill, and for the transition layer and sand layer. The density of the fill was determined with the sand replacement method in accordance with AS 1289.5.3.1. The layer of soft clay was divided into 3 sub layers to capture the distinct compression and permeability characters identified by various tests carried out for this research and reported in the literatures.

Due to the geometry of the problem, a 2D axisymmetric model was used to represent the problem.
5.3.1 Modelling method used

There are two modelling methods available in PLAXIS 2D: Plane strain and axisymmetric. Both models are simplifications of a real 3D problem. The plane strain model assumes that there is no strain along the axis perpendicular to the modelling page and is suitable for problems with uniform cross section geometries. The axisymmetric model assumes that there are no strain along circumferential directions and is suitable for problems with uniform radial cross section geometries (Brinkgreve and Broere, 2015). Due to the arrangement of the field tests, the axisymmetric model is used to model the piled embankment. Finest option was used during mesh generation. The generated mesh for different models are shown in Figure 5.1 and Figure 5.2.

Figure 5.1 Mesh generation for embankment with PVDs
5.3.2 Soil element type used

In PLAXIS 2D, there are two types of elements available for numerical modelling: 6-Node element and 15-Node Element, as shown in Figure 5.3.

A 6-Node element has six nodes and 3 stress point while a 15-Node element has fifteen nodes and 12 stress points. At each Node, deformation and stress are
calculated and stored, they can be inspected in the output program. At each stress point, only stresses are calculated and stored, hence only stresses can be inspected in the output program.

The interpolation functions are used to interpolate values inside an element based on known values in the (Brinkgreve et al., 2011). The 6-Node elements provide a second order interpolation while as the 15-Node elements provide a forth order interpolation. The 15-Node element is a very accurate element, as suggested by Nagegaal et al. (1974,) Sloan (1981) and Sloan and Randolph (1982). Brinkgreve and Broere (2015) pointed out that the 15-Node elements are particularly recommended to be used in axisymmetric analysis. The 15-Node elements are used in this study.

5.3.3 Material model used

The soft Ballina clay was modelled with Soft Soil Creep Model in PLAXIS and frictional materials are model with MC model in PLAXIS. Piles are modelled with linear elastic model with equivalent EA as the CHS piles used in the field test. The ground water table has been assumed to be at the ground surface.

For Soft Soil Creep Model, Modified deformation indices are used.

\[
Modified\swelling\indexs = \kappa = \frac{C_s}{2.3 \times (1 + e)} \quad \text{Equation 5-1}
\]

\[
Modified\swelling\indexs = \lambda = \frac{C_c}{2.3 \times (1 + e)} \quad \text{Equation 5-2}
\]

\[
Modified\swelling\indexs = \mu = \frac{C_a}{2.3 \times (1 + e)} \quad \text{Equation 5-3}
\]

Where

\(C_s\) is Swelling index
$C_c$ is Compression index

$C_a$ is Secondary compression index

$\kappa$ and $\lambda$ are Cam-Clay parameters

e is Void ratio

To consider the interaction between the pile and surrounding soil, an interface unit was added between the pile shaft and soil matrix. With the interface, the relative movement between pile and soil was captured and fed into the negative skin friction calculation performed by PLAXIS.

5.3.4 Boundary condition and mesh size

Automatically generated deformation boundary condition was adopted which has total fixity at the bottom of the model and normal fixity at either side of the model. The upper boundary of the model is free to deform. The interface between the pile and soil was set to impermeable, so as the bottom and left boundary of the model. The finest mesh size option was used to generate the mesh.

5.3.5 Numerical modelling of consolidation under vertical stress without PVD

The first step in numerical analysis is to validate the model by comparing the monitoring results from embankment without PVD to results predicted by the prediction so that the parameters and geometry used in the numerical model can be verified. The comparison between calculated DoC based on measured and predicted excess pore water pressure are shown in Figure 5.4 to Figure 5.8. The smooth continues curves were from PLAXIS prediction. The readings from Piezometer 4
at a depth of 8 m are not shown because the DoC value is unrealistic (as low as 
−2.5). At depths of 3 m and 5 m the measured data matches the PLAXIS results 
reasonably well, especially at a depth of 3 m, but a depth of 8 m the data is rather 
discrete, even though the average value is comparable to the prediction. Although 
there were limited settlement data, Figure 5.9 and Figure 5.10 show that 
predicted settlement matches the measured data very well, and therefore the 
numerical model with selected parameters was validated by the field tests.
Table 5.1 Parameters used in field test PLAXIS analyses

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Depth (m)</th>
<th>E (kPa)</th>
<th>N</th>
<th>γ</th>
<th>e0</th>
<th>C'</th>
<th>φ'</th>
<th>k_v (m/s)</th>
<th>k_h (m/s)</th>
<th>C_c</th>
<th>Cr</th>
<th>OCR</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>N/A</td>
<td>8000</td>
<td>0.3</td>
<td>18</td>
<td>0.5</td>
<td>1</td>
<td>35</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Crust</td>
<td>0.0 - 1.5</td>
<td>N/A</td>
<td>N/A</td>
<td>16</td>
<td>1.8</td>
<td>5</td>
<td>30</td>
<td>1.3E-4</td>
<td>1.8E-4</td>
<td>0.14</td>
<td>0.035</td>
<td>7</td>
</tr>
<tr>
<td>Soft clay 1</td>
<td>1.5 – 4.0</td>
<td>N/A</td>
<td>N/A</td>
<td>15</td>
<td>2.9</td>
<td>1</td>
<td>20</td>
<td>1.7E-3</td>
<td>3.4E-3</td>
<td>1.2</td>
<td>0.16</td>
<td>1.7</td>
</tr>
<tr>
<td>Soft clay 2</td>
<td>4.0 – 7.0</td>
<td>N/A</td>
<td>N/A</td>
<td>15</td>
<td>2.9</td>
<td>1</td>
<td>20</td>
<td>2.3E-4</td>
<td>4.5E-4</td>
<td>1.3</td>
<td>0.16</td>
<td>1.7</td>
</tr>
<tr>
<td>Soft clay 3</td>
<td>7.0 -10.0</td>
<td>N/A</td>
<td>N/A</td>
<td>15</td>
<td>2.9</td>
<td>1</td>
<td>20</td>
<td>1.7E-4</td>
<td>3.4E-4</td>
<td>1.4</td>
<td>0.16</td>
<td>1.7</td>
</tr>
<tr>
<td>Transition</td>
<td>10.0-15.0</td>
<td>4000</td>
<td>0.3</td>
<td>16</td>
<td>1.5</td>
<td>5</td>
<td>28</td>
<td>0.8E-3</td>
<td>1.6E-3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Sand layer</td>
<td>15.0-20.0</td>
<td>6000</td>
<td>0.3</td>
<td>18</td>
<td>0.5</td>
<td>1</td>
<td>35</td>
<td>4E-3</td>
<td>4E-3</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>
Figure 5.4 DoC calculated from measured data and PLAXIS at P3, at a depth of 3m

Figure 5.5 DoC calculated from measured data and PLAXIS at P4, at a depth of 3m
Figure 5.6 DoC calculated from measured data and PLAXIS at P3, at a depth of 5m

Figure 5.7 DoC calculated from measured data and PLAXIS at P4, at a depth of 5m
Figure 5.8 DoC calculated from measured data and PLAXIS at P3, at depth of 8m

Figure 5.9 Settlement from monitoring and PLAXIS analyses at SP3
5.3.6 Numerical modelling of consolidation under vertical stress with PVDs

Once the numerical model for consolidation under vertical load without PVD was validated with monitoring data, a conversion is needed to simulate the vertical drains in the axisymmetric condition. The concept of simulating PVDs around a single pile under surcharge load in an axisymmetric model was developed by Indraratna et al., (2008). In the field PVDs were installed in a triangular pattern, and therefore, as shown in Figure 5.11, the unit cell of the problem, including a pile with a diameter a, is sitting at the centre, surrounded by 6 vertical drains. The distance between any vertical drain to the centre of the pile or to any adjacent drain is S. A pile and the outer boundary of the equivalent drain wall are considered to be impermeable, so once individual drains are converted to a continuous drain wall, an axisymmetric model can be built.
$a$ is the diameter of the pile, and $s$ is the spacing between the centre of the pile and PVD.

The rate of volume change in a vertical direction can be expressed as

$$\frac{\partial V}{\partial t} = \pi \frac{\partial \varepsilon}{\partial t} (r^2 - a^2) dz$$

Equation 5-4

The rate of water flow out of the soil mass can be expressed as

$$\frac{\partial Q}{\partial t} = k_h \frac{\partial u}{\partial r} \frac{2\pi r dz}{\gamma_w}$$

Equation 5-5

By assuming that the rate of volume change is equal to the rate of water flow out of the soil mass, then
\[ u = \frac{r_w}{2k_h} \frac{\partial \varepsilon}{\partial t} \left( \frac{r^2}{2} - a^2 \ln r \right) + c \]  

Equation 5-6

Apply the boundary condition \( u = 0 \) when \( r = s \), we have

\[ u = \frac{r_w}{2k_h} \frac{\partial \varepsilon}{\partial t} \left( \frac{r^2 - s^2}{2} - a^2 \ln \left( \frac{r}{s} \right) \right) \]  

Equation 5-7

The average pore water pressure can be expressed as

\[ u'_{ave} \pi (s^2 - a^2) l = \int_0^l \int_a^s 2\pi r u dr dz \]  

Equation 5-8

Substituting equation 4.14 into equation 4.15 gives

\[ u'_{ave} \pi (s^2 - a^2) l = \int_0^l \int_a^s 2\pi r \frac{r_w}{2k_h} \frac{\partial \varepsilon}{\partial t} \left( \frac{r^2 - s^2}{2} - a^2 \ln \left( \frac{r}{s} \right) \right) r dr dz \]  

Equation 5-9

Integrate the left side of the above equation and let \( \alpha = \frac{s}{a} \) and \( \beta = \frac{a}{a} \), leads to

\[ u'_{ave} = \frac{y_w}{k_h} \frac{\partial \varepsilon}{\partial t} \frac{1}{\beta} \frac{a^2 \mu'}{d_e} \]  

Equation 5-10

where \( \mu' = \frac{a^4 + 4a^2 \beta^2 + 4\beta^4 \ln \left( \frac{\beta}{2 \sqrt{1 + \beta^2}} \right)}{a^2 - \beta^2} \), \( \alpha = 0.952 \) so \( \beta = \frac{0.952 a}{s} \)

Also,

\[ \frac{\partial \varepsilon}{\partial t} = -m_v \frac{\partial u_{ave}}{\partial t} \]  

Equation 5-11

By combining the above two equation, we have

\[ u'_{ave} = - \frac{y_w}{k_h} m_v \frac{\partial u_{ave}}{\partial t} \frac{1}{\beta} \frac{a^2 \mu'}{d_e} \]  

Equation 5-12

Rearranging the above equation and integrating it with the initial condition \( u_{ave} = u_{max} \) at \( t = 0 \), gives
\[
\frac{u'_{ave}}{u_0} = e\left(\frac{-\beta U}{\mu} \right)
\]

Equation 5-13

Hence
\[
U' = 1 - \frac{u'_{ave}}{u_0} = 1 - e\left(\frac{-\beta U}{\mu} \right)
\]

Equation 5-14

Let \( U = 1 - e\left(\frac{-\beta U}{\mu} \right) = U' = 1 - e\left(\frac{-\beta U}{\mu} \right) \), hence
\[
\frac{k_{h,ring}}{k_h} = \frac{\mu'}{\mu} = \frac{\alpha^4 + 4\alpha^2 \beta^2 + 4\beta^4 \ln\left(\frac{\beta}{2.117\alpha}\right)}{\alpha^2 - \beta^2}
\]

Equation 5-15

Similarly, if smear effect is considered
\[
\frac{k_{h,ring}}{k_h} = [1 - \theta(1 - \theta)]
\]

\[
\frac{\alpha^4 + 4\alpha^2 \beta^2 + 4\beta^4 \ln\left(\frac{\beta}{2.117\alpha}\right)}{\alpha^2 - \beta^2}
\]

Equation 5-16

Where \( \theta = \frac{k_{hs}}{k_h} \)

\[ \theta = \frac{A_s}{A} \]

Based on Equation 5-15 the equivalent horizontal permeability can be calculated, as listed in Table 5.2. To model the behaviour of an embankment under vertical stress with PVDs, \( K_{h,ring} \) is used to replace initial \( k_h \) after installing vertical drains; vertical drains are modelled as a continuous drain wall.
Table 5.2 Equivalent horizontal permeability \( k_{h,\text{ring}} \)

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>( K_h )</th>
<th>( K_{h,\text{ring}} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soft clay 1</td>
<td>3.4E-3</td>
<td>0.6E-3</td>
</tr>
<tr>
<td>Soft clay 2</td>
<td>4.5E-4</td>
<td>0.8E-4</td>
</tr>
<tr>
<td>Soft clay 3</td>
<td>3.4E-4</td>
<td>0.6E-4</td>
</tr>
</tbody>
</table>

A comparison between the measured and predicted behaviour of an embankment with PVDs are shown in Figure 5.12 to Figure 5.19; note that apart from Piezometer 2 at a depth of 8 m, the predicted results fit the monitoring data very well.

Figure 5.12 DoC calculated from measured data and PLAXIS at P1, at depth of 3m
Figure 5.13 DoC calculated from measured data and PLAXIS at P2, at depth of 3m.

Figure 5.14 DoC calculated from measured data and PLAXIS at P1, at depth of 5m.
Figure 5.15 DoC calculated from measured data and PLAXIS at P2, at depth of 5m

Figure 5.16 DoC calculated from measured data and PLAXIS at P1, at depth of 8m
Figure 5.17 DoC calculated from measured data and PLAXIS at P2, at depth of 8m

Figure 5.18 Settlement from monitoring and PLAXIS analyses at SP1
5.4 Modelling the drag down force acting on the pile

One of the main purposes of this study is to determine how the consolidation of soft soil affects the downdrag force acting on a pile. In Chapter 3, the results from a full-scale field test supported the assumption that less downdrag force on a pile can be expected where the foundation is improved by pre-consolidation. However, the optimum time of consolidation which produces the most economical outputs was not studied on site, since a full-scale parametric study is impractical. Therefore, numerical modelling is used in parametric study instead.

5.4.1 Validation of numerical model

The numerical models used in section 5.3.5 and 5.3.6 are also used to model the downdrag on a pile. Several consolidation stages were added at the end of previous models by installing a plate element to represent the piles. The time
interval for each stage of consolidation is set to match the monitoring schedule for the field test, and since the values predicted by the model generally agree with the measured data, the model was also used to simulate the effect of drag down. The comparison between the measured and predicted drag down forces is shown in Figure 5.21.

![Figure 5.20 Drag down force from modelling with field Cα](image)
The predicted dragdown force on a pile installed at an embankment improved by PVD is much higher than the measured value, but the predicted values in an embankment without PVD matches the measured data very well. This inconsistency between the predicted and measured downdrag force on a pile installed in soft soil under a surcharge preloading with PVDs is caused by secondary consolidation. After two weeks of consolidation with PVDs, part of the soft clay had reached the end of primary consolidation, and secondary consolidation had already commenced. As stated above, the $C_\alpha$ value adopted by the numerical model and back calculated from nearby VP embankment was between 0.04 to 0.06, but the clay under the VP embankment had not altered much, however, the clay surrounding the driven pile had changed enormously due to installation. As mentioned, the $G/Su$ value of soft clay at the test facility, according to Pineda et al. (2016), ranged from 73 to 121, with an approximate average value of 100. Based on cavity expansion theory, the radius of plastic zone for a 0.4 m diameter pile is 2 m, so it is assumed that soft clay within a 2.2 m radius from the centre of the pile is heavily deformed and over stressed. Again, cavity expansion theory suggests that the radial stress induced by pile driving within plastic zone is from 1.3 to 5 $Su$, so piling and the follow up dissipation of excess pore water pressure over consolidates the soil, causing an average over consolidation ratio of around 1.4. It is well known that $C_\alpha$ may be reduced by increasing the OCR. Ladd and Mesri proposed two separate methods to determine $C_\alpha'/C_\alpha$, in which for 1.4 OCR, $C_\alpha'/C_\alpha$ is approximately 0.1. Hence, values of $C_\alpha'$ ranging from 0.004 to 0.006 is updated in the numerical analysis, and the results are shown in Figure 5.21. Note that with updated $C_\alpha'$ value, the
predicted drag down force matches the measured field test value well and suggests that the numerical model used in analyses is valid and can be adapted in the parametric study.

Figure 5.21 Measured and predicted drag down force with updated $\alpha$ value
5.4.2 Parametric study

To investigate how efficient adopting PVD facilitated consolidation is as a measure to reduce downdrag a numerical parametric study was carried out with 3 sets of simulations. The first set of simulations had the validated model as a starting point, so only the time for consolidation was varied. In the second set of simulations the length of the pile changed to 22 m, the layer of soft clayed changed according, but other aspects of the model were unaltered. In the third set of simulations the pile increased in length to 32m, with the corresponding thickness of soft clay. The cases considered are summarised below in Table 5.3.

Table 5.3 Cases for parametric study

<table>
<thead>
<tr>
<th>Soft clay thickness</th>
<th>Case ID</th>
<th>With/without PVD</th>
<th>Consolidation time before piling</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>Without</td>
<td>2 weeks</td>
</tr>
<tr>
<td>Set 1: 8 m</td>
<td>2</td>
<td>Without</td>
<td>2 Months</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>Without</td>
<td>1 year</td>
</tr>
<tr>
<td>Set 2:20 m</td>
<td>4</td>
<td>Without</td>
<td>2 years</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>Without</td>
<td>4 year</td>
</tr>
<tr>
<td>Set 3: 30 m</td>
<td>6</td>
<td>With</td>
<td>2 weeks</td>
</tr>
<tr>
<td></td>
<td>7</td>
<td>With</td>
<td>2 months</td>
</tr>
<tr>
<td></td>
<td>8</td>
<td>With</td>
<td>1 year</td>
</tr>
</tbody>
</table>

For each set of study, 8 sub cases are included. 5 of the sub cases are without PVD and 3 remaining cases are with PVDs. These sub cases are set out to check
the impact on downdrag force by various pre-consolidation time. The time considered for post piling is up to 20 years and the development of drag down force over time is plotted in Figure 5.22 to Figure 5.33; the drag down force along the pile at selected times is also presented.

Figure 5.22 Drag down force on pile in 8m thick soft clay with no ground improvement
Figure 5.23 Drag down force on pile in 8 m thick soft clay with surcharge preloading for 2 months
Figure 5.24 Drag down force on pile in 8 m thick soft clay with surcharge preloading for 2 years
Figure 5.25 Drag down force on pile in 8 m thick soft clay with PVDs and surcharge preloading for 2 weeks
Figure 5.26 Drag down force on pile in 20 m thick soft clay with no ground improvement
Figure 5.27 Drag down force on pile in 20 m thick soft clay with surcharge preloading for 2 months
Figure 5.28 Drag down force on pile in 20 m thick soft clay with surcharge preloading for 2 years
Figure 5.29 Drag down force on pile in 20 m thick soft clay with PVDs and surcharge preloading for 2 weeks
Figure 5.30 Drag down force on pile in 30 m thick soft clay with no ground improvement
Figure 5.31 Drag down force on pile in 30 m thick soft clay with surcharge preloading for 1 year
Figure 5.32 Drag down force on pile in 30 m thick soft clay with surcharge preloading for 2 years
In every case, the downdrag force increased over time, with or without ground improvement. The downdrag forces were consistently less with ground improvement compared to the cases without ground improvement. As expected, by increasing the length of a pile and the thickness of soft soil, a higher drag down force is predicted. As shown in Figure 5.34, 30m, 20m and 8 m long piles...
experienced 350, 150, and 50 kN of drag down force after 4 years post piling respectively, but with ground improvement (surcharge preloading), the maximum downdrag force is reduced. If ground is improved by surcharge preloading with PVDs, the maximum downdrag force is reached after 1-year post piling and are below 100 kN.

![Figure 5.34 Maximum drag down force predicted by numerical modelling](image)

The degree to which reducing the downdrag force by preloading can be examined by comparing the maximum downdrag force in different cases. An index “Drag Down Reduction (DDR)” was defined as the maximum downdrag force with ground improvement over the maximum downdrag force with no ground improvement. A plot of DDR vs time allowed for consolidation prior to pile installation, while the efficiency of reducing downdrag force by preloading is obtained at the slope of the curve.
Figure 5.35 DDR and DoC for piling in 8 m thick soft clay

Figure 5.36 DDR and DoC for piling in 20 m thick soft clay
Figure 5.37 DDR and DoC for piling in 30 m thick soft clay

Figure 5.35 to Figure 5.37 shows that the efficiency of reducing the downdrag force by pre-consolidation without PVD decreases as the thickness of soft soil increases. For 8 m thick soft clay, about 0.7 DDR can be achieved if the ground is allowed to consolidate for one year, so the efficiency is 0.7 DDR/year. Between one and two years of consolidation increases the efficiency to about 0.2 DDR/year, but if the consolidation time remains unchanged, the maximum DDR decreases below 0.2 and 0.1 for 20 m and 30 m thick clay, so the resulting efficiencies are 0.1 and 0.05 DDR/year. These results indicate that pre-consolidation with surcharge only method is ineffective for reduce the downdrag on piles installed in thick compressive soil, but at least 70% of downdrag force can be removed if the ground is allowed to consolidate with PVDs installed for only 8 weeks; this will deliver an efficiency of 4.2 DDR/year, regardless of the thickness of the compressible layer. If the ground is allowed to consolidate for more than 8 weeks with PVDs installed, the DDR can reach 0.8 to 0.9, but the
efficiency is much lower at approximately 0.12 DDR/year. It is therefore much more efficient to reduce downdrag by pre-consolidate the ground with PVDs installed for no more than 8 months.

The plots for developing the downdrag force vs ground surface settlement after piling is shown in Figure 5.38 and Figure 5.39 where all three sets of drag down force initially developed rapidly as settlement increased. The difference between the three sets is due to the lengths of embedded piles and the rate of drag down forces at different settlements.

![Figure 5.38 Drag down force vs settlement at ground surface after piling without PVD](image)

There is a bi-linear relationship between the drag down force and soil settlement in all cases without PVD, such that the longer the pile embedded in soft clay, the higher the maximum drag down force and the larger the soil settlement at which the rate of drag down force diminishes. Moreover, after 20 mm of settlement,
there are clear boundaries between each two sets of data, so for the cases with PVDs, there is no obvious bi-linear relationship, or the clear boundaries mentioned above.

Figure 5.39 Drag down force vs settlement at ground surface after piling with PVDs

5.5 Summary
Numerical parametric study was carried out to investigate the influence on negative skin friction from various factors, i.e. inclusion of PVDs, thickness of the soft soil and pre-piling consolidation duration. Finite element software package PLAXIS was adopted to perform the numerical parametric study. A 2-D axis-symmetrical model was created based on site investigation results and a technique which converts 3-D consolidation with PVDs to 2-D axis-symmetrical consolidation with equivalent horizontal permeability of soil was developed. The model was validated with the monitoring data from the field test and then used in parametrical study.
The results of parametrical study indicated that pre-consolidated the ground before piling can reduce the negative skin friction. Inclusion of PVDs significantly increases the effectiveness of pre-consolidation. To quantify the efficiency of PVDs assisted pre-consolidation, an index “DDR” was defined as the maximum downdrag force with ground improvement over the maximum downdrag force with no ground improvement. It was concluded that the efficiency of PVDs assisted pre-consolidation increase as vertical drainage length increases, as well as pre-consolidation time decreases.
CHAPTER 6. SUMMARY OF RESEARCH OUTCOMES AND RECOMMENDATIONS FOR FUTURE STUDY

The purpose of this research is to investigate the potential benefits of the combined use of PVD and driven piles as a ground improvement technique. Several aspects of interaction between a pile foundation and soft clay were considered and the outcomes of this research are summarised below; recommendations for future study are also given.

6.1 Reducing in excess pore water pressure and lateral deformation
Excess pore water pressure and lateral soil displacement during pile driving can potentially cause problems because they can have an adverse impact on sensitive structures and cause localised instability. Some attempts have been made to attach vertical drains on piles before installation, but this is not always effective in terms of reducing excess pore water pressure; and in terms of lateral soil displacement, there is no record of any reduction in lateral soil movement. This research has confirmed that pre-consolidating soft clay reduces excess pore water pressure and later soil deformation during piling.

6.2 Reducing the drag down and increasing the shaft capacity after piling
Drag down forces acting on piles can be troublesome, especially for piles installed in thick compressive soil. Literature shows that after 20 years of installation, a pile embedded in 30 metres of marine sediment can be subjected to a drag down force which exceeds the structural capacity of the pile. Current research has proved that improving the compressible layer by surcharge preloading can control
the down drag, and PVD can facilitate consolidation and reduce potential drag down within a relatively short period of time.

6.3 **Recommendations for future study**

In terms of full-scale model testing, it is always better to have more field data, but due to the time frame and budget constraints, data were collected at limited locations. Although these data are considered to be representative, they are not enough to study the interaction between piles and PVD improved ground comprehensively. Again, due to time and budget constraints, the surcharge used in the field test was never removed, so the soft clay layer on site had not been vertically over consolidated by preloading. Although the test results show a significant reduction in excess pore water pressure, lateral soil displacement and the drag down force, if the surcharge is removed then further reductions are expected. A future study could investigate the influence that different OCRs of soft soil would have on piling, and since a single pile was modelled in this research, the group effect of multiple piles was not considered, and since piles are always used as groups of different patterns, future research should study a group of piles installed in soft soil improved by PVDs over a relatively large surface area.

A semi-empirical model of pore water pressure generated by pile driving is given. It fits the field and lab test results better than previous models. All existing models only consider the same pore water pressure along the length of the pile while the proposed model is able to distinguish between pore water generated along a pile in different layers of soil. However, it cannot provide continuous portrait of piling
induced pore water pressure along the depth. This is one direction that could be taken in future studies.

The consolidation of soil surrounding a pile after installation is not yet fully understood. The field test indicated that excess pore water pressure generated by pile driving dissipates faster than excess pore water pressure generated due to surcharge preloading. It is considered here that the radial consolidation of soil after piling imposes an OCR on the soil which has not been considered elsewhere. A comprehensive understanding of the consolidation of soil after piling that considers the radial and vertical drainage paths, reduces the total radial stress during consolidation, and also considers the various soil properties along the depth is another area of research for future studies.

PLAXIS was used in this thesis for parametric studies, and the Soft Soil Creep model is used to simulate the compressive clay layer encountered on site. Although interaction between a pile and soft clay was modelled successfully, there is no existing model in the software package with the capacity to accurately model the pore water pressure generated by pile driving, so more work is needed to establish a model which can fulfil these purposes.
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