INSTALLATION AND LOAD TESTING OF HELICAL PILES
IN A SENSITIVE FINE-GRAINED SOIL

by

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ABSTRACT

The purpose of this research was to determine how soil disturbance caused by the installation of helical piles in sensitive fine-grained soils affects the mobilization of axial pile capacity at different times after installation.

Six instrumented, full-scale helical piles were installed in lightly overconsolidated, highly sensitive, marine silt and clay at the Colebrook test site in South Surrey, B.C. Prior to pile installation, a detailed in-situ testing program was carried out using field vane shear tests and seismic piezocone penetration testing which included pore pressure dissipation tests.

The excess pore pressures within the soil surrounding the piles was monitored during and after pile installation by means of piezometers located at various depths and radial distances from the pile shaft, and using piezo-ports which were mounted on the pile shaft. The changes in pore pressure during pile installation were indicators of the soil deformations caused by pile installation. The observed pore pressure dissipation around the piles indicated that primary reconsolidation of the soil was complete after about 7 days.

After allowing a recovery period following installation, which varied between 19 hours, 7 days and 6 weeks, piles with two different helix plate spacings were loaded to failure under axial compressive loads. Strain gauges mounted on the pile shaft were monitored during load testing to determine the distribution of loading throughout the pile at the various load levels up to and including failure. Load-settlement curves were generated for different pile sections at different times after installation. The piezometers and piezo-ports were also monitored during load testing and the distribution of excess pore pressures was used as an indicator of the distribution of soil deformations caused by pile displacement.

The undrained shear strengths mobilized by the different sections of the piles were back-calculated from the measured loads using published formulations. An “index of soil destructuring” is proposed which relates the ratio of the mobilized undrained shear strength to the in-situ vertical effective stress at the start of load testing to the corresponding strength ratios of the soil in its intact state and in a completely destructured state. The index of soil destructuring is proposed as the basis for a proposed capacity prediction method that is based on the undrained strength ratio.
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1.0 INTRODUCTION

1.1 CONVENTIONAL PILE TYPES

The following pile types are most commonly used as structural support for foundations:

- timber piles (typically tapered cylindrical geometry),
- steel pipe piles (installed open-ended or closed-ended),
- steel H-piles (rolled HP structural sections),
- pre-cast reinforced concrete piles (cylindrical, square or octagonal cross-sections), and
- cast-in-place concrete piles (in pre-bored holes or inside a driven shell or casing).

These piles can be driven into place with a pile hammer, vibrated or jacked into place, or installed in a pre-bored hole. Vibro-installation of piles is typically limited to granular soils. Pre-boring is typically limited to stiff to very stiff fine-grained soils or unsaturated soils in which there is less chance of hole collapse. In soft saturated soils, it is unlikely that a borehole would stay open long enough to be filled with concrete. For this reason, piles installed in soft fine-grained soils are usually driven or jacked into place. This causes an outward displacement of soil away from the pile, the volume of which depends on the pile geometry. Timber piles, pre-cast concrete piles, and steel pipe piles driven closed-ended, are all classified as “displacement” piles, since they cause a large volume of soil displacement. Steel H-piles and open-ended pipe piles are usually classified as “low-displacement” piles. If, however, the bottom of such piles become plugged with soil, they will also cause a large volume of soil displacement.

Piles are typically designed to penetrate through layers of weak and/or compressible soils to reach a relatively competent bearing stratum, in which most of their capacity will be mobilized. This class of piles is typically referred to as “end-bearing piles”.

In some areas, however, the thickness of the weak soil strata may be too extensive to make it practical or economical to advance piles all the way to a competent bearing stratum. In these cases, it is usually preferable to design a raft-type foundation over a pile foundation. However,  

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1 A more detailed review of the various common pile types and their advantages and disadvantages is given by Bowles (1996).
raft foundations are not well-suited to some structures; e.g. tall structures subjected to significant overturning moments due to lateral loads. Also, the behaviour of the surficial soils (e.g. highly compressible organic silts and peats) can be such that the overall performance of the structure would be improved if the bearing capacity was mobilized within deeper soils, particularly if those soils are significantly less compressible. In these cases, it may be warranted to suspend a group of piles within the relatively weak soils. This class of piles is typically referred to as “friction piles”.

The capacity of friction piles depends almost entirely on the available shear strength of soil that has typically been severely disturbed by the pile installation process. The soil deformations that are induced by the pile installation process alter the total and effective stress states within the soil surrounding the pile and can significantly alter the micro-structure of the soil. Most natural clays are micro-structured and will exhibit some degradation in strength and stiffness when the natural micro-structure is disturbed (Burland, 1990; Leroueil & Vaughan, 1990). The degree of strength and stiffness degradation will vary from soil to soil and will depend on the intensity of the soil deformations caused by pile installation. Further changes in the stress state and soil fabric, and hence the strength and stiffness, can continue to occur with time after pile installation.

Consequently, the strength and the stiffness of the soil around the piles after installation can be significantly different from that which existed prior to pile installation. If reasonably accurate predictions of pile capacity are to be made, it is imperative that the effects of potential changes in the applicable soil parameters be considered, even if it is not yet possible to accurately quantify these changes.

1.2 **INTRODUCTION TO HELICAL PILES**

Helical piles, also called screw piles, consist of a series of helix-shaped steel plates that are attached to a slender steel shaft. The piles are installed by rotating the shaft using a hydraulic torque unit. As the shaft turns, the helices cut into the soil, and the downward pitch of each helical plate allows the helix plates to pull the rest of the pile into the ground. Extensions are added to the shaft as the helix plates are advanced down to the desired bearing stratum. In order to reach the desired depth:
• the torque unit must be able to apply sufficient torque to the pile shaft to overcome the friction along the surface of the shaft and the helix plates, and

• the soil must provide enough bearing resistance along the top of the helix plates to overcome the penetration resistance at the tip of the pile shaft.

These piles displace a relatively small volume of soil and can therefore be classified as “low-displacement” piles.

When axial loads are applied to the piles, each of the helix plates transfers compressive stress to the soil above or below the plate (for piles in uplift or compression, respectively) while frictional resistance is mobilized along the pile shaft above the group of helix plates.

The geometry of this type of pile allows soil resistance to be mobilized more or less equally in either compression or in uplift. Hence, this type of design and installation process has been used for decades for tensile soil anchors, or where a foundation is subjected to large overturning moments, such as for masts and towers. Screw piles were reportedly first used in 1838 for the foundations of Maplin Sands lighthouse in the Thames estuary (Narasimha Rao et al., 1991). More recently, large numbers of helical piles were used in the USSR between 1961 and 1964 for the foundations of communication masts up to 254 m high (Narasimha Rao et al., 1991). They were also tested and used in clayey and sandy soils as guy wire anchors for some of the support towers for two 870 km long high-voltage transmission lines extending from the northeastern to southwestern corners of British Columbia (Robinson & Taylor, 1969).

Since the torque units used to install the piles are relatively small, they can be mounted to the boom of a backhoe or other small excavator, or can be hung from a man-portable A-frame. Thus, the installation equipment that is required is limited, and the screw piles and installation equipment can be used in very tight spaces, if necessary, or used in areas with restricted overhead clearance. Also, there are virtually no ground vibrations and relatively little noise created by the installation of helical piles. Consequently, this type of pile is very well-suited and has been used frequently in recent years for foundation repairs, upgrades and retrofits. The lighter installation equipment can also be used in areas with weak and/or compressible subgrades with much less associated disturbance or protection requirements than for conventional pile driving equipment. In any of the above instances, the use of helical piles as pure compression
members can be more practical than conventional driven piles, which must be installed using large pile driving rigs and require a great deal of overhead clearance. Due to the economical use of material and minimal equipment mobilization requirements, this type of pile can provide a more economical alternative to conventional piles in a wide range of applications.

In the past, the design of these piles was typically experience-based, with the assistance of empirical relations between measured installation torque and approximate load-carrying capacity. More recently, a better understanding of the load transfer mechanism acquired from full-scale field testing and laboratory-based model pile research has allowed the development of prediction formulas which are based on soil shear strength parameters. From this research, it was determined that the spacing of the helix plates has a very important effect on the load transfer mechanism, and hence, on the capacity that can be mobilized within a given soil. A summary of the findings of this research will be given in Chapter 2.

When helical piles are used to support compression loads, a problem in the past has been the tendency for the slender shafts to buckle before the full capacity of the helix plates could be mobilized. This was found to be a problem particularly when the helix plates are anchored within a competent bearing stratum that is overlain by soft, weak soils that do not provide adequate lateral support for the slender shaft. To reduce the risk of this occurring, Vickars Developments Co. Ltd. devised and patented a method of “pulling down” a column of grout around the shaft extension sections (the method used to achieve this is described in more detail in Section 5.1). Once the grout cures, the bolted connections between successive pile sections become much more rigid and the composite column can mobilize much greater lateral support from the soil, thereby greatly increasing the buckling resistance of the pile shaft. The larger diameter composite column also provides a greater resistance to lateral pile loads. This adaptation of the more conventional un-grouted helical pile design has been patented by Vickars Development Co. under the trademark PULLDOWN™ Pile. Pile capacities in excess of 300 kN have been achieved using this type of design.

1.3 PURPOSE & OBJECTIVES OF RESEARCH

There are a number of areas in Canada, North America, and the world, where thick deposits of relatively sensitive fine-grained soils (clayey silts to clays) overlie competent bearing strata. Such deposits are particularly prevalent in coastal regions. In the Fraser River Lowland region
of southwestern British Columbia, there are extensive deposits of fine-grained alluvial sediments that are greater than 30 m thick in places. These clayey silts and silty clays were typically deposited in salt-water environments and have subsequently developed a moderate to high sensitivity due to fresh-water leaching. These soils present significant challenges for construction of civil infrastructure in these areas. To make things even more challenging, the surficial soils in these lowland areas are typically highly organic (peat and organic silts) and highly compressible. Consequently, piled foundations are often used to mobilize adequate load-carrying capacity and/or to reduce foundation settlements to tolerable levels. However, it is not always economically feasible to drive the piles all the way to the dense bearing stratum, or there are economic benefits to installing friction piles within the sensitive silty clays and clayey silts. In these cases, it is very important that the design engineer is aware of the changes in soil properties that occur as a result of disturbance effects during pile installation, and as a result of the subsequent reconsolidation and aging processes. These changes need to be considered carefully when selecting design parameters.

Because of the difficulties in accurately predicting the capacity of piles in such difficult soil conditions, most engineers prefer to use conventional driven piles, with which there has been the most experience, instead of helical piles. This is sometimes the case even when helical piles may be better suited or more economical for a given application. With the recent development of sound theoretical formulations to predict the capacity of helical piles, there is no reason why these piles could not be designed with a similar degree of confidence as for conventional piles. For both types of piles, the accuracy with which the capacity can be predicted will depend to a large degree on the choice of design soil parameters.

The purpose of this research project was:

- to obtain a better understanding of the response of sensitive fine-grained soils to the installation of helical piles, and

- to investigate the manner and effectiveness with which the different sections of the helical piles mobilize axial resistance within these soils, given the spatial variations in soil properties caused by pile installation and the subsequent changes in these properties with time.

The improved understanding of the soil disturbance effects and the implications of soil disturbance on the resistance mobilization properties of the piles forms the basis for a proposed
method of estimating the soil strengths mobilized by the different pile sections at different times after installation.

1.4 **Scope and Limitations of Study**

To accomplish the research objectives listed above, a detailed field testing program was carried out at a soft soil test site in South Surrey, British Columbia. In this study, a total of 6 instrumented, full-scale helical piles were installed in soft, sensitive, marine silt and clay. The piles were loaded to failure under axial compressive loads, after allowing a recovery period following installation of 19 hours, 7 days or 6 weeks. Piles with two different helix plate spacings were investigated in this study, because of previous research suggesting different load transfer mechanisms for the different spacings. The variations in excess pore pressures within the soil surrounding the piles during and after pile installation were monitored by means of piezometers located at various depths and radial distances from the pile shaft, and using piezo-ports mounted on the pile shaft. The changes in pore pressure during pile installation were indicators of the type and extent of soil deformations caused by pile installation. The dissipation of the excess pore pressures after pile installation was an indicator of the degree of primary consolidation which would have occurred prior to load testing. Strain gauges mounted on the pile shaft were monitored during load testing to determine the distribution of loading throughout the pile at the various load levels up to and including failure. This allowed the changes in the development of resistance along different sections of the piles at different times after installation to be studied in detail. The piezometers and piezo-ports were also monitored during load testing and the distribution of excess pore pressures generated during load testing was used as an indicator of the distribution of soil deformations caused by pile displacement.

A complete understanding of the pile and soil behaviour observed during this study would require a detailed laboratory study of the soil behaviour, which was beyond the scope of the current research project. For this study, site characterization and soil property definition was based on high-quality in-situ testing procedures, complemented by laboratory index testing.

The long-term settlement behaviour of helical piles under sustained loads has not been investigated within the scope of this study. The effects of creep can significantly affect the long-term performance of a piled foundation, and so creep can be a critical design issue in some soils and will need to be considered in future research.
1.5 Thesis Organization

Chapter 1 of this thesis has laid out the theoretical premise of and practical need for this research project, and has outlined the objectives that were set out for the study. The scope and limitations of the study have also been mentioned.

Chapter 2 provides background theory, based primarily on published information, which is important to, or provides the basis for, the interpretation of the results obtained during this study.

A description of the test site is provided in Chapter 3. This includes the location of the test site, the surficial geology in the area, a history of site development and a characterization of the general subsurface conditions at the test site.

The magnitudes and trends with depth of the key engineering parameters that were used in the interpretation of the observed pile behaviour are described in Chapter 4.

A description of the helical test piles and electronic instrumentation used in this study is provided in Chapter 5.

Chapter 6 presents the results of the pore pressure monitoring carried out during pile installation and during the subsequent recovery period before load testing. The implications of the observed pore pressure response are discussed and, wherever possible, the results are compared to similar measurements made with the piezocone penetrometer or to predictions from theoretical solutions.

Chapter 7 presents and discusses the results of axial compressive load tests and the observed trends with recovery time after pile installation. The changes in the observed pile behaviour with time are explained in terms of the degree and distribution of soil disturbance caused by pile installation and the inferred changes in the soil properties caused by installation disturbance, reconsolidation and aging.

In Chapter 8, a rational methodology is proposed for estimating the soil parameters required to predict the axial capacity of helical piles at different times after installation in fine-grained soils.

Recommendations for further research are given in Chapter 9.

The figures for each chapter are all located at the end of the chapter.
2.0 BACKGROUND THEORY

In this chapter, the results of an extensive literature review are presented with the aim of providing a framework for understanding the soil and pile behaviour observed in this study. This includes:

- A conceptual framework for understanding the response of fine-grained soils to pile installation (Section 2.1) and the changes in soil properties that occur during the reconsolidation process following pile installation (Section 2.2).

- The existing methods of predicting the capacity of helical piles/anchors installed in fine-grained soils (Section 2.3).

- A discussion of the common methods that exist to predict the reconsolidation of the soil with time after pile installation or piezocone penetration, and of the important factors influencing such predictions (Section 2.4).

2.1 SOIL RESPONSE TO PILE INSTALLATION

2.1.1 Soil Deformations

Flaate (1972) makes reference to observations by Skrede (1967) of downward bending of clay layering next to the surface of a driven timber pile. The downward bending of light and dark bentonite layers due to penetration of a flat-ended model pile was observed by Rourk (1961). Flaate also makes reference to observations by Skaven-Haug (1940) of fluid clay that was squeezed up to the ground surface when a pile was driven into quick clay. Similar observations of fluid clay being squeezed out to the ground surface around the shaft of piles driven into Mexico City clay were reported by Zeevaert (1950). Clearly, the process of pile installation causes severe deformation of the soil close to the pile.

The deep penetration of a conventional pile or a cone penetrometer into a saturated soil medium under undrained conditions causes the soil particles to displace in a manner which depends on the geometry of the penetrating body. The deformation of the soil during these displacements causes changes in the strain field around the body until some steady state is achieved, which generally occurs at large distances above the tip of the pile or probe. The distribution of shear
strains caused by undrained penetration of cylindrical objects has been modelled conceptually using Cylindrical Cavity Expansion (CCE) theory and using the Strain Path Method (SPM). Cavity expansion theory is described in Appendix A.1, while the Strain Path Method is described in Appendix A.2.

Cavity expansion theory provides a simple and rational theoretical model for predicting the distribution of shear strain in the soil over the large region deformed by an expanding cavity. The radial distribution of shear strain that is predicted using CCE theory, based on the logarithmic strain formulations described in Appendix A.1.1, is shown on Figure 2.1. The radial distribution of average shear strain around the shaft of a cone penetrometer (which is essentially a miniature pile with a conical tip), which was predicted by Levadoux & Baligh (1980) using the SPM, was almost identical to that predicted by CCE theory.

2.1.2 Total Stresses and Shear Stresses

The soil displacements caused by pile installation are accompanied by increases in the total stresses in all three principal stress directions: radial, circumferential and vertical. The distribution of these stresses around the pile depends on the strain distribution and on the stress-strain behaviour and pore pressure response of the soil.

Within an annulus of soil surrounding the pile, the shear stresses induced by the soil deformations will have been large enough to cause failure of the soil. The soil that has reached the failure state is said to be “plastic”, and the outer boundary of the region of failed soil is called the “plastic boundary”. The soil beyond this boundary is in a pre-failure state. The location of this boundary relative to the pile depends on the strain distribution around the pile and on the pre-failure stress-strain behaviour of the soil.

The elastic-plastic constitutive model is often used to simply represent the stress-strain behaviour of soils. This model assumes that the stress-strain response up to failure is linear and elastic and the post-failure strength remains constant at the peak strength of the material (i.e. perfectly plastic). However, the pre-failure stress-strain response of natural soils is typically non-linear, since stiffness tends to decrease with increasing strain. Also, during undrained shearing, soils will typically soften or harden once the failure state is reached, depending on the pore pressure behaviour at large strains (either contractive or dilative, respectively). The pore pressure
response is dependent on many factors such as stress history, micro-structure, mode of shear, etc. Soft fine-grained soils tend to be contractive during shearing so that positive pore pressures are generated under undrained conditions. These soils will experience a reduction in shear strength \( (s_u) \) with continued straining after the applied shear stress reaches the peak \( s_u \). An example of a non-linear, strain-softening (NL-SS) stress-strain curve is compared to that of an elastic-plastic (EL-PL) material on Figure 2.2.

The radial distributions of induced radial, circumferential and vertical total stresses \( (\Delta \sigma_r, \Delta \sigma_\theta, \Delta \sigma_z, \text{respectively}) \) and shear stress \( (\tau) \) around an expanded cylindrical cavity, which were generated from CCE theory (as described in Appendix A.1) using both the NL-SS and EL-PL stress-strain relations from Figure 2.2, are shown on Figure 2.3. The total stresses and shear stress have all been normalized by the undrained shear strength, \( s_u \). In this particular case, the change in mean total stress \( (\Delta \sigma_{\text{mean}}) \) is equivalent to \( \Delta \sigma_z \). It can be seen that the principal stresses calculated using the EL-PL and NL-SS solutions are very similar across the region where the NL-SS curve is at peak shear stress. However, in the regions where the elastic-plastic and NL-SS stress-strain curves diverge (within the large-strain region and in the pre-failure region), the major and minor principal stress distributions also diverge. At radial distances \( (r) \) around the edge of the plastic zone \( (r = r_p) \), the elastic-plastic model tends to give estimates of \( \Delta \sigma_{\text{mean}} \) that are less than that of the more realistic NL-SS model. At the wall of the cavity \( (r = R) \), the elastic-plastic model tends to give estimates of \( \Delta \sigma_r \) that are higher than that of the strain-softening material.

Based on the equation that was used to derive the EL-PL radial stress distribution shown on Figure 2.3 (Equation A15 in Appendix A), the total radial stress at the wall of the cavity will tend to increase with increasing undrained shear strength \( (s_u) \) and with increasing rigidity index \( (G/s_u) \). This is consistent with the observations by Lehane et al. (1994) that the total radial stress measured along the shaft of model piles during installation tends to increase with increasing overconsolidation ratio (OCR).

Figure 2.3 provides a conceptual framework for visualizing the stress distribution around a penetrating pile and for understanding how the distribution of stresses is influenced by the stress-strain response of the soil. However, cavity expansion theory does not necessarily provide an accurate prediction of actual stresses around penetrating cylindrical objects. Jardine et al. (1998)
compared the limit pressures calculated using CCE theory to those measured along the shaft of the Imperial College instrumented model pile. They found that the radial stresses predicted from CCE were 2.5 to 4 times greater than those measured along the shaft of the pile far above the tip in both soft, lightly overconsolidated Bothkennar clay and stiff, highly overconsolidated London clay.

During penetration of a pile or cone penetrometer, the soil is subjected to both vertical and radial displacements, which make the resulting distributions of stresses and strains much more complex than is assumed by cavity expansion theories. The difference between the entirely radial deformation pattern assumed by cavity expansion theories and the actual deformations caused by penetration of piles (eg. as observed by Rourk, 1961) are particularly evident close to the pile surface.

The Strain Path Method (Baligh, 1975; Levadoux & Baligh, 1980; and Baligh & Levadoux, 1980) is expected to provide a more realistic analysis of penetration effects close to the pile surface. The magnitudes of $\sigma_r$, $\sigma_\theta$, and $\sigma_z$ predicted by Baligh & Levadoux, 1980 (see Figure A-1 in Appendix A) along the surface of the cone probe during penetration are significantly higher close to the face of the cone than they are along the shaft of the probe above the cone. The sharp decrease in $\sigma_r$ behind the shoulder of the conical tip, which is predicted by Baligh & Levadoux (1980), is consistent with the trends of $\sigma_r$ measured by Jardine et al. (1998) using the Imperial College instrumented model pile. At some distance behind the cone, the stresses reduce to steady state values, which are predicted to be about 33% less than those predicted by CCE theory. Baligh & Levadoux (1980) also predict that $\sigma_r$ is the minor principal stress within this region, instead of being the major principal stress, as predicted by CCE theory.

### 2.1.3 Excess Pore Pressure

The generation of large positive excess pore pressures due to pile driving in normally consolidated to lightly overconsolidated fine-grained soils have been reported in the literature by a number of researchers, including: Bjerrum & Johannessen (1961), Lo & Stermac (1965), Orrje & Broms (1967), Koizumi & Ito (1967), Fellenius & Samson (1976), Bozozuk et al. (1978), Roy et al. (1981), Robertson et al. (1990) and Pestana et al. (2002). Baligh & Levadoux (1980) plotted the measured excess pore pressures from a number of studies at different sites, as shown
on Figure 2.4. Most of the data came from sites with plasticity index between 20 and 25, OCR between 1 and 2.5, and $s_u$ between 15 and 25 kPa. For these soils, they found that the excess pore pressures at the shaft of the piles was generally on the order of about twice the in-situ vertical effective stress, but decreased with increasing radial distance from the pile, generally becoming negligible at distances of about 20 to 30 pile radii.

The excess pore pressure, $\Delta u$, can be expressed as the sum of two components:

\[ \Delta u = \Delta u_{oct} + \Delta u_{shear} \]  

where:

- $\Delta u_{oct}$ accompanies a change in mean (or octahedral) total stress ($\Delta\sigma_{mean}$ or $\Delta\sigma_{oct}$), and
- $\Delta u_{shear}$ accompanies a change in the deviator stresses, and results from the tendency for the soil to change volume during shear.

In saturated soils, $\Delta u_{oct} = \Delta\sigma_{oct}$. The magnitude of $\Delta u_{shear}$ is strongly dependent on stress history. Normally consolidated clays are strongly contractant when sheared and, therefore, positive $\Delta u_{shear}$ are generated during shearing. If structural collapse occurs at failure, as is typically the case in highly sensitive clays, higher positive $\Delta u_{shear}$ is generated. Conversely, moderately to heavily over-consolidated clays are strongly dilatant when sheared, so negative $\Delta u_{shear}$ are generated. For clays that have been mechanically over-consolidated to an overconsolidation ratio (OCR) of about 2, $\Delta u_{shear}$ is typically negligible.

Closed-form solutions exist for the radial distribution of excess pore pressures around cylindrical and spherical cavities (Equations A18a and A18b in Appendix A). However, these solutions are not expected to be very reliable for predicting $\Delta u$ close to the surface of a penetrating pile. Also, the elastic-plastic constitutive model that is used to generate the closed-form solutions neglects $\Delta u_{shear}$, as is discussed in Appendix B.2.

Some researchers have suggested accounting for $\Delta u_{shear}$ by using pore pressure parameters, such as Skempton’s A or Henkel’s $\alpha$, determined from laboratory testing, with the shear strength of the soil. Since these parameters are usually determined from the measured pore pressures at
failure, they do not necessarily capture the large-strain $\Delta u_{\text{shear}}$ response, which is applicable in the region of soil near an expanded cavity or penetrating object.

Some researches have also attempted to predict $\Delta u_{\text{shear}}$ from effective stress paths inferred from the Cam Clay or Modified Cam Clay (MCC) models, which are based on critical state soil mechanics (e.g. Burns & Mayne, 1998, Cao et al., 2001). In both the cited cases, the final effective stress is determined from the undrained shear strength and the slope of the critical state line (M). However, the assumption that the effective stress path reaches the critical state line at the peak undrained shear strength is likely to be incorrect for strain-softening soils, since the critical state only occurs at large strains where the shear strength may have been significantly reduced below its peak value. Cao et al. (2001) acknowledge the difference between the peak and ultimate (large-strain) strengths but state that the MCC model cannot account for large-strain reductions in shear strength.

Levadoux & Baligh (1980) predicted the distribution of $\Delta u$ around a penetrating cone probe using the SPM and the total stress soil model MIT-T1 to calculate deviatoric stresses and shear-induced pore pressures. This model was calibrated using observed soil properties from laboratory testing of re-sedimented Boston blue clay (BBC) normally consolidated under $K_o$ conditions. The resulting distribution of $\Delta u$ (normalized by the initial vertical effective stress, $\sigma'_{vo}$) around the cone probe is included on Figure A-3 in Appendix A. Radial distributions are reproduced on Figure 2.5 for 3 different locations along the shaft: just above the shoulder of the cone, and at 6R and 10R above the cone tip. Beyond an inner radial boundary (located at $r_s$), $\Delta u$ is predicted to decrease approximately linearly with the logarithm of radial distance in a manner that is reasonably consistent regardless of distance behind the shoulder of the cone, rapidly trending toward zero at a relatively consistent outer radial boundary. This is similar to cavity expansion predictions. The reduction in $\Delta u$ with distance behind the shoulder of the cone, which occurs close to the surface of the probe, is due to the drop in total stresses behind the shoulder of the cone. This produces a flatter radial gradient of pore pressure within the inner annulus between R and $r_s$. This flat radial gradient, which extends to greater radial distances with greater distance behind the shoulder, explains the slow initial rate of pore pressure dissipation predicted by SPM-based consolidation solutions for locations along the shaft of the penetrometer.
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Whittle (1990, 1993) describes a more realistic effective stress soil model, MIT-E3, which avoids having to calculate shear-induced pore pressures separately. This is a more complex model that can account for non-linear pre-failure behaviour, post-peak strain-softening and strength anisotropy in both normally consolidated and over-consolidated soils.

2.2 Effects of Changes in Stress State after Installation on Pile Capacity

2.2.1 Changes in Radial Effective Stresses

Measurements of both total radial stresses and pore pressures during and after the installation of pile in fine-grained soils have been reported by Koizumi & Ito (1967), Azzouz & Morrison (1988), Coop & Wroth (1989), Lehane et al. (1994) and Jardine et al. (1998). Such measurements have shown that in normally consolidated to lightly overconsolidated fine-grained soils, the increases in total radial stress that occur during pile installation are accompanied by large positive pore pressures. The net result is that the radial effective stress along the surface of the pile at the end of installation ($\sigma'_r$) is reduced below the initial horizontal effective stress in the soil ($\sigma'_{ho}$).

After installation, the total radial stress decreases with time in a manner similar to the observed dissipation of excess pore pressure. However, at early times in the dissipation process, the drop in total stress occurs more quickly than the drop in pore pressure (Azzouz & Morrison, 1988; Jardine et al., 1998), such that a further reduction in $\sigma'_r$ is observed. Eventually, the rate of pore pressure dissipation becomes greater than the rate of total stress relaxation, and $\sigma'_r$ increases during the remainder of the dissipation period. The value of $\sigma'_r$ at the end of the dissipation period ($\sigma'_{rc}$) tends to be equal to $\sigma'_{ho}$ for normally consolidated soils but tends to be greater than $\sigma'_{ho}$ for overconsolidated soils by an amount that increases with increasing OCR of the soil.

Lehane et al. (1994) compiled the results of lateral stress measurements made during penetration of instrumented piles and model piles in clays at a number of test sites. They found that the degree of relaxation of the radial total stress during reconsolidation increased consistently with decreasing OCR and increasing sensitivity. They also found that there were consistent trends of $K_c = \sigma'_{rc}/\sigma'_{vo}$ decreasing with decreasing OCR and increasing sensitivity.
2.2.2 Increases in Pile Capacity with Time

Piles that are installed in normally to lightly overconsolidated fine-grained soils typically have low initial capacities. This is attributed to the low effective stresses next to the pile shaft at the end of pile installation, as discussed above. A subsequent increase in capacity with time is typically observed in such soils (e.g., Seed & Reese, 1957; Eide et al., 1961; Flaate, 1972; Konrad & Roy, 1987).

Seed & Reese (1957) and Konrad & Roy (1987) have shown that the increase in pile capacity is proportional to the dissipation of excess pore pressure following installation. Konrad & Roy (1987) derived a total radial stress relaxation curve ($\sigma_r(t)$ vs $t$) for a test pile by back-calculating the average radial effective stress acting on the surface of the pile from pile capacities measured at various times after pile installation. The back-calculated $\sigma'_r(t)$ values were added to the pore pressures measured on the pile shaft at the same times to obtain the total radial stresses, $\sigma_r(t)$. The resulting $\sigma_r(t)$ curve is similar to that measured directly by Azzouz & Morrison (1988) and by Koizumi & Ito (1967). This is strong evidence that the increase in effective stress along the pile shaft during the reconsolidation process controls the increase in soil shear strength and the resulting capacity of friction piles.

2.2.3 Undrained Shear Strength

The preceding discussion indicates that the increase in shear strength along the shaft of a friction pile is controlled by the radial effective stress that occurs during consolidation. It is very difficult to measure lateral soil stress, however, and so the $\sigma'_r$ acting along the pile surface is usually unknown. Consequently, the “total stress” approach to pile design, wherein the frictional resistance along the pile shaft is calculated using the undrained shear strength, $s_u$, is still the most common method, since the application of $s_u$ does not explicitly require a knowledge of the effective stress state of the soil.

However, it is very important to realize that $s_u$ is not a unique parameter, but depends on the mode of shear relative to the 3-dimensional effective stress state in the soil as well as to the pore pressure response of the soil to shearing. As a result of this anisotropy effect, the $s_u$ that is measured in different laboratory tests can vary by a factor of 2 or more (for compression vs extension tests). For the most common laboratory strength tests, triaxial compression (TC) and
extension (TE) and direct simple shear (DSS), the measured magnitudes of $s_u$ are typically in the following order: $(s_u)_{TC} > (s_u)_{DSS} > (s_u)_{TE}$.

The $s_u$ that is measured in in-situ tests may be completely different from all of the above strengths, or may be some average of some or all of the above strengths. Thus, careful consideration must be given to the selection of an appropriate $s_u$ to use for pile design.

The undrained shear strength measured in fine-grained soils is also known to be strain rate-dependent. In general, $s_u$ increases logarithmically with increasing strain rate (eg. Kulhawy & Mayne, 1990), although this increase has been found to be greater in soils with higher plasticity (Bjerrum, 1973; Azzouz et al. 1983).

The magnitude of $s_u$ is directly proportional to the magnitude of the mean effective stress during consolidation. However, since the horizontal effective stresses are not usually known with any confidence in field situations, it is common practice to normalize $s_u$ by the vertical effective stress ($\sigma_v'$). The resulting $s_u/\sigma_v'$ ratio is called the undrained strength ratio. For a given clay that is normally consolidated (NC), $(s_u/\sigma_v')_{NC}$ is a constant. For the same clay in an overconsolidated (OC) state, $s_u/\sigma_v'$ increases according to the overconsolidation ratio (OCR) of the soil, according to the following expression (Ladd et al., 1977):

$$
(s_u/\sigma_v')_{OC} = (s_u/\sigma_v')_{NC} \cdot OCR^m
$$

Typical values of the exponent, $m$, range between 0.8 and 1.0.

2.2.4 Natural Development of Soil Micro-Structure

Most natural clayey soils have a distinctive micro-structure which affects the stress-strain behaviour of the soil. This micro-structure is the product of a complex series of geologic processes, which includes:

- the depositional environment and post-depositional environment during primary consolidation (salt water or fresh water),
- secondary consolidation (aging),
- fresh-water leaching of salt ions,
• thixotropic hardening (as described by Mitchell, 1976),
• cementation (due to deposition of carbonates, metal oxides, organic matter, etc.), etc.

Soil that has been subjected to aging, thixotropic hardening and/or cementation will have a greater strength and stiffness in its intact state than the same soil that has not been subjected to such processes or has had such effects removed due to a break-down of the micro-structure. The effects of consolidation and aging processes on the development of shear strength were described by Leroueil et al. (1979) and by Leroueil and Vaughan (1990). Since these concepts are critical to the understanding of the behaviour of the test piles in this study, they will be illustrated here using the simplified one-dimensional void ratio – effective stress relations shown on Figure 2.6.

Consider a clay deposited in a marine (salt-water) environment. As the soil consolidates under the increasing overburden weight, the vertical effective stress, \( \sigma'_v \), increases and the void ratio, \( e \), decreases along a characteristic line which Burland (1990) has called the “Sedimentation Compression Line” (SCL). This soil is said to be “normally consolidated”, since the consolidation pressure \( (\sigma'_v) \) is the largest effective stress that the young soil has been subjected to. While the soil remains normally consolidated, the vertical yield stress, \( \sigma'_{vy} \) (which is commonly referred to as the preconsolidation pressure, or maximum past pressure, and denoted by \( \sigma'_p \)), remains equivalent to \( \sigma'_v \), and \( \sigma'_{vy} \) increases along the SCL as the void ratio decreases during primary consolidation. Once the deposition of overburden is complete, and any excess pore pressure has dissipated, the \( e-\sigma'_v \) state of the soil will have reached point I on Figure 2.6, at which point the vertical effective stress will be \( \sigma'_{vo} \), and the vertical yield stress will be \( (\sigma'_{vy})_t \).

Under constant effective stress, fine-grained soils continued to experience a decrease in volume due to secondary consolidation (indicated as \( \Delta e_s(t) \) on Figure 2.6). It has been observed that \( \sigma'_{vy} \) continues to increase during this aging period, while the vertical effective stress remains constant at \( \sigma'_{vo} \). To explain this observed increase in yield stress with time, Bjerrum (1967) proposed a conceptual model in which \( \sigma'_{vy} \) continues to follow the SCL as the void ratio decreases with time. The resulting increase in the yield stress ratio, YSR = \( \sigma'_{vy}/\sigma'_{vo} \) (which is more commonly referred to as the overconsolidation ratio, denoted by OCR), has the same net effect as if the soil had been preconsolidated to \( \sigma'_{vy} \) and then unloaded. Since the soil has not actually been subjected to any historical unloading, this form of aging-induced overconsolidation is sometimes
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referred to as “quasi-preconsolidation”. After sufficient time has passed, the yield stress will reach point A on the SCL on Figure 2.6, and \( \text{YSR}_A = (\sigma'_{vy})_A/\sigma'_{vo} \). Between points I and A, the peak undrained shear strength, \( s_u \), of the soil tends to increase as \( \sigma'_{vy} \) increases, according to the \( s_u/\sigma'_{vy} \) ratio of the soil, which remains essentially constant during primary consolidation and aging.

Once the rate of change in void ratio becomes slow enough, thixotropic bonding of the soil particles can occur. This thixotropic hardening causes an additional increase in \( \sigma'_{vy} \) that is independent of void ratio, thereby causing \( \sigma'_{vy} \) to displace to the right of the SCL, which serves to further increase the YSR of the soil. This process is illustrated on Figure 2.6 by the dashed segment between point A and point P. It should be noted that cementation at the particle contacts will have a similar effect on \( \sigma'_{vy} \) as thixotropic hardening. Between points A and P, \( s_u \) continues to increase as \( \sigma'_{vy} \) increases, although the \( s_u/\sigma'_{vy} \) ratio of the soil may be altered (Leroueil et al. 1979).

Thus, for a natural soil with an in-situ \( e_o \) and \( \sigma'_{vo} \) (point O on Figure 2.6), the yield stress would be \( (\sigma'_{vy})_P \) and the peak undrained shear strength would be \( (s_u)_0 \). A typical stress-strain curve for such a structured soil during undrained shearing is shown in the inset of Figure 2.6. The pre-failure shearing response tends to be relatively stiff and failure tends to be brittle, with a dramatic post-peak drop in strength, due to the shear-induced destruction of the inter-particle bonds and the resulting collapse of the soil structure.

### 2.2.5 Destruction of Micro-Structure

If a sample of this soil could be recovered and prepared without disrupting the natural micro-structure, and was tested in a one-dimensional consolidation test, the shape of the resulting compression curve would be similar to that shown on Figure 2.6. As the applied vertical effective stress is increased above \( \sigma'_{vo} \), the micro-structure will remain intact and the soil will behave as an overconsolidated material as long as the yield stress of the soil is not exceeded. Once the applied vertical effective stress reaches \( (\sigma'_{vy})_P \), any additional strain will cause the inter-particle bonds to be broken and the soil structure to collapse. This breakdown of the natural micro-structure is referred to as “destructuring”. In the drained consolidation test, the structural collapse is accompanied by an abrupt and significant reduction in void ratio, as shown...
on Figure 2.6. If the chemistry of the pore water was unchanged since deposition, the drained
destructuring would eventually bring the e-σ’v relation back onto the SCL. If, however, the salt
content had decreased due to fresh-water leaching, the slope of the compression line may be less
than that of the original SCL due to the reduction in salt content, as was observed by Locat &
Lefebvre (1982). Thus, the e-logσ’v will drop steeply from σ’vy, pass below the original SCL and
approach the “destructured compression line” (DCL) of the soil at the lower salt content. Such
e-logσ’v curves are typical of sensitive clays.

If the soil is destructured through undrained shearing, as would occur when a pile is installed in a
fine-grained soil, no reduction in void ratio is possible during the destructuring event. Instead,
the structural collapse generates large positive pore pressures that cause a reduction in effective
stress. Thus, the e-σ’v state of a soil that has been destructured by large undrained shear
deformations (typically referred to as remoulding) will tend to move from point O to a point D
on the DCL, as shown on Figure 2.6. The yield stress of the soil will also be reduced from (σ’vy)p
to some value between (σ’vy)A and (σ’vy)D which depends on the degree of destructuring. A loss
of soil memory within soil adjacent to driven piles has been reported by Roy & Lemieux (1986)
and Hunt et al. (1998, 2002). If the soil is completely remoulded, the “memory” of the soil will
be completely erased and the destructured soil will be normally consolidated (σ’vy = σ’, at point
D). The peak undrained shear strength of the soil after primary consolidation is also greatly
reduced due to the destruction of the inter-particle bonds and the pore pressures generated by the
collapse of the soil structure.

2.2.6 Recovery of Undrained Shear Strength During Reconsolidation

As the effective stress increases due to the dissipation of excess pore pressure and/or increases in
total stress, the void ratio decreases along the DCL. Such reductions in void ratio and moisture
content have been observed within soil adjacent to driven piles by many researchers: eg.
Cummings et al. (1950), Seed & Reese (1957), Flaate (1972), Roy & Lemieux (1986) and Hunt
et al. (1998, 2002).

For a completely destructured soil that reconsolidates from point D to point C on Figure 2.6, the
void ratio decreases to e_c and the vertical yield stress increases to (σ’vy)_C = σ’vc. This causes a
recovery in the undrained shear strength to (s_u)_C according to the s_u/σ’vy ratio of the destructured
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soil. This behaviour has been described by Leroueil et al. (1979) and Leroueil and Vaughan (1990) and evidence of this occurring adjacent to driven piles was given by Rutledge (1950) and Seed & Reese (1957).

From the conceptual example shown on Figure 2.6, \((s_u)_C\) at the end of the pore pressure dissipation period will be significantly less than the intact peak \((s_u)_o\) of the soil prior to pile installation. However, the difference between \((s_u)_C\) and \((s_u)_o\) will depend on the initial YSR of the soil, on the reduction in YSR and effective stress caused by pile installation, on the increase in effective stress during reconsolidation, and on the compression index of the destructured soil. Thus, \((s_u)_C\) could be less than or equal to \((s_u)_o\). It is unlikely that \((s_u)_C\) would be greater than \((s_u)_o\) under a given mode of shear and at a given strain rate, unless the effective stress in the soil adjacent to the pile is increased above the previous yield stress of the soil, or the soil acquires additional cementation due to chemical or organic processes. The above discussion explains the apparent contradictions between the comparisons of \(s_u\) measured before and after pile installation which are given in the literature: eg. Skempton (1950), Seed & Reese (1957), Orrje & Broms (1967), Flaate (1972), Roy & Lemieux (1986), Hunt et al. (2002).

The typical stress-strain behaviour of the reconsolidated destructured soil during undrained shearing is compared to that of the structured soil (at the same effective confining stress) in the inset of Figure 2.6. The destructured soil typically has a reduced stiffness, strength, and rigidity, and is less brittle (eg. Leroueil et al., 1979; Roy & Lemieux, 1986; Hunt et al., 2002).

The processes of aging and thixotropy/cementation may lead to further increases in the strength and stiffness of the reconsolidated soil with time after the end of the pore pressure dissipation period, as described above. This will depend on many factors, including the response of the pile and soil to loading caused by construction activities.

As a result of all these effects, the value of \(s_u\) that is measured during conventional in-situ and laboratory tests on the soil prior to pile installation is rarely the same as that which is mobilized by a pile at failure. This has frequently been observed to be the case for conventional piles, such that Meyerhof (1976) proposed correcting \(s_u\) by an empirical factor, \(\alpha\), when calculating the side friction on a pile shaft, \(f_s = \alpha \cdot s_u\), where \(\alpha \leq 1\) and generally decreases with increasing overconsolidation.

20
2.2.7 Summary and Conclusions

The key points from the above discussions are summarized below:

- The soil displacements caused by pile installation induce large deformations and shear strains within an annulus of soil surrounding the pile. These deformations cause large positive excess pore pressures to be generated in normally consolidated to lightly overconsolidated saturated fine-grained soils, and can cause a significant break-down of the micro-structure of the soil.

- Immediately after pile installation, the undrained shear strength of the soil next to the pile has been significantly reduced below the intact strength of the material, and the initial shaft resistance of the pile is typically very low. This is due in part to the high positive pore pressures and reduced effective stresses within the soil next to the pile. The degree of strength loss in the soil will depend on the sensitivity of the soil and on the induced strain levels relative to the threshold strain required to cause structural breakdown. Therefore, the distribution of shear strength around a pile after installation will depend on the distribution of strain caused during installation.

- As the excess pore pressures dissipate and the effective stresses increase, the soil next to the pile consolidates to a lower void ratio than in its natural state. The undrained strength increases according to the $s_u/\sigma'$ ratio of the soil in its partly to completely destructured state.

- Upon completion of the pore pressure dissipation process, the undrained strength of the soil next to the pile will have reached a value which depends on the magnitude of the principal effective stresses around the pile, and on the $s_u/\sigma'$ ratio of the partly to completely destructured soil. Any degree of structural breakdown will lead to a reduction in the $s_u/\sigma'$ ratio below the $s_u/\sigma'_{\text{vo}}$ of the intact soil before pile installation. The vertical effective stress after pore pressure equalisation will be equal to $\sigma'_{\text{vo}}$, while $\sigma'_{\text{rc}}$ will be equal to or greater than $\sigma'_{\text{ho}}$, depending on the YSR and sensitivity of the soil and the location relative to the pile tip. Thus, the undrained shear strength of the disturbed, reconsolidated soil may be higher than, equal to, or lower than, its natural intact strength, depending on the properties of the soil and location relative to the pile.
• The strength may continue to increase with time after completion of dissipation due to aging processes.

2.3 CAPACITY PREDICTION METHODS FOR HELICAL PILES/ANCHORS

A literature review suggests that, as early as the 1940s, pioneering work on the bearing capacity of helical piles was being carried out. The testing included model and full-scale load tests, and the results and theories from this testing were published by Wilson (1950). This work included an investigation of the relation between installation torque and bearing capacity.

Using the results of field tests on helical anchors, which were being used as guy-wire anchors for transmission towers within extensive clayey deposits, the pull-out resistance was empirically related to installation torque (Robinson & Taylor, 1969). This relation was not found to be very consistent, but monitoring during anchor installation was used to determine whether or not a given anchor would need to be proof tested.

More recently, a number of researchers have developed rational analytical methods of calculating helical pile capacity in both compression and uplift based on soil shear strengths. These methods are similar to those developed for under-reamed cast-in-place piles, since the load transfer mechanism of the helix plates on helical piles/anchors is similar to that of under-reams on cast-in-place piles. The behaviour of helical piles in compression and tension has been observed to be very similar (Narasimha Rao, Prasad and Shetty, 1991). Therefore, the analysis methods are essentially interchangeable, provided that the differences in gravitational influences are taken into account.

Two distinct methods have been proposed: the “cylindrical shear method” and the “individual plate bearing method”. These methods are described in the following sub-sections, along with the empirical torque-based capacity prediction method. All 3 methods were reviewed by Hoyt & Clemence (1989). They also carried out a statistical analysis of the prediction accuracy of these methods based on the results of 91 helical anchor pull-out load tests carried out on a number of different multi-helix anchor geometries at 24 different sites with sandy, silty and clayey soils.

Both the cylindrical shear and individual plate bearing methods require an estimate of the shear strength of the soil to calculate the ultimate resistance mobilized by the helical piles/anchors.
Wherever these methods have been applied to fine-grained soils in the literature, the undrained shear strength, $s_u$, of the soil has been used for this purpose.

### 2.3.1 Cylindrical Shear Method

The cylindrical shear method was proposed by Mooney, Adamczak and Clemence (1985). In this method, a continuous cylindrical failure surface is assumed between the top and bottom helices, as illustrated on Figure 2.7 (the helical pile is shown in uplift; however, the load transfer mechanisms in compression are identical but opposite in direction). The total pile capacity is the sum of:

- $Q_{cyl}$ - the frictional resistance mobilized along the cylindrical failure surface,

- $Q_{end}$ - the bearing resistance above the top plate (pile in uplift) or below the bottom plate and base of pile shaft (pile in compression), and

- $Q_{shaft}$ - the frictional resistance along the pile shaft.

For relatively rapid loading in fine-grained soils, total stress parameters are typically used in design, such that the resistances described above can be calculated according to the following formulations:

\[
Q_{cyl} = A_{cyl} \cdot s_u \quad (2.3)
\]

\[
Q_{end} = (A_{hx} + A_{tip}) \cdot N_c \cdot s_u \quad \text{(in compression)} \quad (2.4a)
\]

or

\[
Q_2 = A_{hx} \cdot N_{cu} \cdot s_u \quad \text{(in uplift)} \quad (2.4b)
\]

\[
Q_{shaft} = A_{shaft} \cdot \alpha \cdot s_u \quad (2.5)
\]

where: $s_u$ is the applicable undrained shear strength of the soil,

$A_{cyl}$ is the area of the cylindrical shear surface, which tends to follow the outer edge of successive helices,

$A_{hx}$ is the net bearing area of the helix plate (not including the cross-sectional area of the pile shaft),

$A_{tip}$ is the cross-sectional area of the pile shaft,
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A\textsubscript{shaft} is the surface area of the pile shaft; for grouted shafts on PULLDOWN Piles, this will be the surface area of the grout column (in this case, the additional resistance mobilized by the bottom grout disc in bearing, Q\textsubscript{4}, can also be included),

\( \alpha \) is the undrained strength correction factor proposed by Meyerhof (1976), which is commonly referred to as the shaft adhesion factor; \( \alpha \leq 1 \) and tends to decrease with increasing overconsolidation of the soil,

\( N_c \) is the standard bearing capacity factor for use in undrained analyses, and is typically estimated to be about 9. For anchors in uplift, the \( N_{cu} \) value is about 9 for helix plate depths greater than about 2-2.5 plate diameters but decreases at shallower depths (Narasimha Rao, Prasad and Veeresh, 1993).

Extensive research carried out on model piles (helix diameter kept constant on each pile) at the Indian Institute of Technology (IIT) in Madras, India, has revealed that the spacing of the helix plates has an important influence on the failure mechanism (Narasimha Rao, Prasad, Shetty and Joshi, 1989). This testing was carried out in tanks that were filled with very soft, completely remoulded clay of known moisture content and shear strength. In their research program, \( s_u \) was measured using the vane shear test. At a helix plate spacing-to-diameter ratio (S/D) of 1.5 (shown on the left side of the photograph on Figure 2.8), nearly continuous failure surfaces were observed from the soil that was recovered on the plates after the anchors were pulled out of the soil. At an S/D ratio of 2.3 (shown in the middle of Figure 2.8), the failure surface was incomplete. At an S/D ratio of 4.6 (shown on the right side of Figure 2.8), some soil was retained on the top of the bottom helix plate, but the failure was certainly not cylindrical.

For helix spacings of 1.5 diameters or less, the total capacity of the model anchors was found to be very consistent. For piles tested in both uplift and compression, excellent agreement was found between measured capacities and predictions made using the cylindrical shear method (mean over-prediction of 5%, with standard deviation of 5%, based on 16 test results from Narasimha Rao, Prasad & Shetty, 1991).

However, at S/D ratios greater than 1.5 (equal lengths between top and bottom helices), the measured capacities of the anchors decreased and the over-prediction error associated with using the cylindrical shear method increased with increasing spacing. These trends are shown on Figure 2.9, which was taken from Narasimha Rao & Prasad (1993). To correct for the error
associated with applying the cylindrical shear method to piles with helices spaced at S/D ratios greater than 1.5 to 2, empirical spacing correction factors, \( S_F \), were proposed by Narasimha Rao & Prasad (1993), which are a function of the S/D ratio. However, all of the model piles tested in the IIT research program had relatively low helix plate-to-shaft diameter ratios, \( D/d_{sh} \), of between 2 and 3. Therefore, the applicability of their \( S_F \) relations to piles with larger \( D/d_{sh} \) ratios has not been verified.

The model anchors and piles tested in the IIT research program all had uniform helix plate sizes. However, helical piles with plate diameters increasing from the pile tip upwards are also very common. For these pile types, Hoyt & Clemence (1989) propose using a tapered cylindrical surface, as shown on Figure 2.10. In undrained analyses, the difference between a tapered and right cylindrical failure surface is not likely to yield significantly different capacity predictions, even though the applicable undrained shear strength may differ between the two different shaped failure surfaces. However, in a drained analysis where the mobilized shear strength depends on the lateral effective stress, the resistance of a tapered pile, such as on Figure 2.10, should be higher in compression than in tension due to differences in lateral earth pressure generated during displacement.

### 2.3.2 Individual Plate Bearing Method

If the helix plates are spaced at greater than about 2 diameters, the capacity can be calculated by assuming that an individual bearing failure occurs above (pile in uplift) or below (pile in compression) each plate, as illustrated on Figure 2.11 (for the uplift case). In undrained analyses, the total pile capacity is calculated as follows:

\[
Q_{ult} = \sum (A_{thx} \cdot N_c \{or \ N_{cu}\}) \cdot s_u + Q_{shaft}
\]  

(2.6)

where all of the parameters were defined in Section 2.3.1. The length of shaft that mobilizes resistance will include the shaft segments above the helices and may include portions of the shaft between the helices, depending on the spacing of the plates. Narasimha Rao, Prasad and Veeresh (1993) determined that a shaft length of 1.4 to 2.3 helix diameters above the top helix plate is not effective in resisting uplift since this section of the shaft is contained within the bearing zone of the top plate. An examination of the data from their study indicates that shaft adhesion between the helices is being mobilized for \( S/D \geq 2 \), but a length corresponding to 1 to 2 helix diameters
(average of 1.7D) is not effective, likely due to the bearing zone of the plates. Thus, for helices spaced at S/D > 2, shaft adhesion between the helices should be considered over inter-helix lengths in excess of about 1.5 to 2 helix diameters. However, for helical piles with large helix plate-to-shaft diameter ratios, the contribution of inter-helix shaft resistance may be negligible, particularly for S/D ≤ 3.

Narasimha Rao, Prasad and Veeresha (1993) compared a large number of measured capacities to the values predicted using the individual bearing method with known shear strengths, and found that the error was generally between about 0 and 12% (under-prediction), with a maximum under-prediction of 22%.

### 2.3.3 Torque-Capacity Relations

Hoyt & Clemence (1989) recommended the following relation between installation torque, T, and uplift capacity, \( Q_u \):

\[
Q_u = K_t \cdot T \tag{2.7}
\]

where \( K_t \) is an empirical correlation factor. The authors quote the following \( K_t \) factors, depending on the pile shaft size:

- square and cylindrical shafts less than 89 mm diameter: \( K_t = 10 \text{ lbs/ft-lb or } 33 \text{ N/N-m} \)
- 89 mm diameter cylindrical shafts: \( K_t = 7 \text{ lbs/ft-lb or } 23 \text{ N/N-m} \)
- 219 mm diameter cylindrical shafts: \( K_t = 3 \text{ lbs/ft-lb or } 10 \text{ N/N-m} \)

These proposed factors were given by Hoyt & Clemence (1989) with no dependence on soil type, consistency/density or helix plate configuration.

From the IIT research program, both installation torque and uplift capacity measurements were published by Narasimha Rao, Prasad, Shetty and Joshi (1989) for anchors with different helix plate spacings and different shaft sizes tested in clays with different consistencies. The torque – capacity comparisons are summarized on Figure 2.12. From this figure, the following observations can be made about the torque – capacity relations for soft, unstructured fine-grained soils:
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• For a given anchor/pile geometry, the mobilized capacity increases with torque in a linear manner as the soil strength/stiffness increases. This is the basis of the empirical prediction method given in Equation 2.7, and the slope of the capacity-torque relations shown on Figure 2.12 (i.e. Q/T) is equivalent to the $K_t$ factor from Equation 2.7.

• For a given number of helices in a given soil consistency, the ratio of capacity to torque was found to decrease with increasing pile shaft size (the ratio of the helix to shaft diameters was kept roughly the same). This is consistent with the increasing $K_t$ values proposed by Hoyt & Clemence for increasing shaft sizes, and is due to the fact that the larger shaft area greatly increases the frictional resistance of the pile during installation but does little to increase the pile capacity.

• The $K_t$ factor was also found to increase with increasing number of helix plates (decreasing S/D), which is clearly illustrated on Figure 2.13. The relative increase in $K_t$ with increasing number of plates is greater for the piles with the smaller shaft diameter. The range of $K_t$ values proposed by Hoyt & Clemence does not account for this observed effect.

Clearly, the helical pile/anchor geometry has an important influence on the torque-capacity relation. The degree of this influence may vary depending on the pore pressure generation behaviour, and possibly on the disturbance-induced strain-softening tendencies, of the soil in which the piles are installed. Therefore, selection of an appropriate $K_t$ factor to predict the capacity of helical piles with varying geometries in different soil conditions would appear to involve a great deal of uncertainty and potential error. However, for a given pile geometry at a given site, it is reasonable to expect that reliable torque-capacity correlations could be derived from a limited number of field load tests which could then be used as a valuable quality control mechanism.

2.3.4 Conclusions

If the shear strength of the soil at the time of pile loading is known with confidence, it appears that the capacity of helical piles can be predicted with a good degree of accuracy using either the cylindrical shear method for $S/D \leq 1.5$ or the individual plate bearing method for $S/D > 2$. It does not appear that the torque-capacity factors proposed by Hoyt & Clemence (1989) should be used for anything other than rough estimates of capacity, unless they can be verified on a site-
specific and pile-specific basis. If site-specific/pile-specific K values could be determined from actual load-test results, torque measurements could serve as a valuable quality control mechanism.

Despite the apparent accuracy of the capacity prediction methods for helical piles/anchors installed in soil with a known strength, Hoyt & Clemence (1989) showed that a great deal of variation in accuracy is observed when the methods are applied to actual field cases without the use of good engineering judgement in the selection of representative soil parameters. Based on the discussions in Sections 2.1 and 2.2, it can be appreciated that the task of selecting an appropriate soil strength is very difficult due to the changes in the soil behaviour that occur as a result of pile installation. Thus, the pile capacity prediction equations given in this section should not be applied without a proper understanding of the effects of pile installation and reconsolidation on the soil behaviour. The purpose of this research study was to develop such an understanding and to use this understanding as a guide for selecting realistic strengths to use in the capacity prediction methods.

2.4 Predicting Pore Pressure Dissipation Around Piles and Piezocones

Due to the large volume of soil through which the excess pore pressures are generated during undrained penetration, and the low permeability of fine-grained soils, the dissipation of the excess pore pressures generated by pile installation can occur slowly over long periods of time. Thus, an estimate of the duration of the dissipation period is important to determine if it is necessary to delay loading of the piles, particularly in the case of large diameter piles, or in groups of closely spaced piles. The rate of excess pore pressure dissipation at different sites can vary by orders of magnitude, mainly due (although not entirely) to the large range of permeabilities encountered within different soils. Methods of estimating the length of time required for pore pressure dissipation are discussed below.

The development of the piezocone probe (which is essentially a small-scale instrumented pile) has made it possible to directly measure installation pore pressures that are typical of those generated during pile driving. The agreement between pore pressures generated around the piezocone and around full-scale piles will depend on:
1) the relative strain rates, which depends on the ratio of the penetration rate to the diameter of the pile or probe (Konrad & Roy, 1987); and

2) the location of the measuring point relative to the tip, since the excess pore pressures generated around the tip are significantly higher than around the shaft at large distances above the tip, particularly for over-consolidated soils (discussed in more detail in Chapter 4).

The dissipation of excess pore pressures generated during piezocone penetration at a particular site can also be monitored during breaks in penetration. However, it is not usually economical to allow dissipation tests to continue much beyond about 50%, due to the slow rate of dissipation in most fine-grained soils.

Also, the standard pore pressure measuring point on piezocones is located very close to the tip (immediately behind the shoulder of the cone). Experience has shown that installation pore pressures are higher and dissipation is faster at this location than at locations further up the shaft of the probe, which are more representative of the conditions around the shaft of long piles. Thus, despite its potential to be used directly to predict the dissipation around full-scale piles, piezocone dissipation testing is usually aimed at obtaining an estimate of the coefficient of consolidation, $c_h$. This parameter is then required as input to solve a theoretical dissipation solution that is appropriate for the pile shaft.

Examples of some of the existing solutions for the dissipation of excess pore pressures around driven piles, piezocone probes, and impervious cylindrical and spherical cavities in general are given below (and described further in Appendix B):

- Torstensson (1977) – 1-D consolidation around impervious cylindrical and spherical cavities; suggested for dissipation around piezocone probe (non-standard geometry),
- Randolph & Wroth (1979) – 1-D consolidation around impervious cylindrical cavity; suggested for dissipation around shaft of driven piles,
- Baligh & Levadoux (1980), Levadoux & Baligh (1986) – 2-D consolidation around piezocone probes (for both 18° and 60° tips); solutions for different locations along probes,
- Houlsby & Teh (1988), Teh & Houlsby (1991) – 2-D consolidation around piezocone probes (standard 60° cones); solutions for different locations along probe.
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Most dissipation solutions are based on linear, uncoupled Terzaghi-Rendulic consolidation theory, which is governed by the following assumptions:

1) There is no coupling between total stress and pore pressure changes during consolidation. However, the field observations by Azzouz & Morrison (1988) and Jardine et al. (1998) prove that this is not true. Coupled Biot theory is more realistic, but makes the analyses much more complex.

2) The permeability - compressibility relation remains linear throughout the consolidation process (i.e. the coefficient of consolidation, $c$, remains constant). This is generally true for soils that are stressed within the recompression range (i.e. $\sigma' < \sigma'_y$). However, most soils experience significant changes in $c$ if the yield stress is exceeded and consolidation proceeds within the normally consolidated range.

Levadoux & Baligh (1986) used two-dimensional finite element solutions for the consolidation of a linear, elastic, isotropic soil skeleton around a cylindrical probe with an 18° conical tip to compare dissipation predictions based on both uncoupled and coupled theories. For such soils, they found that coupling has little effect on the dissipation behaviour along the shaft of the probe, except at early times during the consolidation process, but predictions close to the tip were found to be significantly different. Their analysis does not consider initial undrained total stress redistribution effects.

Both Levadoux & Baligh (1986) and Houlsby & Teh (1988) have determined that consolidation around the cone penetrometer is dominated by the horizontal coefficient of consolidation, $c_h$, provided that $c_h \geq c_v$ (vertical coefficient of consolidation), as is usually the case in fine-grained deposits.

In regards to the question of the applicability of linear analyses, Baligh & Levadoux (1980) suggest that the reconsolidation of normally consolidated and lightly overconsolidated (OCR < 4) soils around cone probes occurs within the recompression range, at least at early stages in the dissipation process. Their conclusions are based on the assumption that the yield stress of the soil does not change during undrained penetration, so that the reduction in effective stress increases the apparent overconsolidation. However, a reduction in yield stress has been observed from samples of reconsolidated soil next to driven piles (noted in Section 2.2.5) due to
destructuring effects. Thus, the relation of the effective stress level at various stages during the reconsolidation process to the yield stress of the partly to completely destructured soil is unclear based on the available data. Given that the radial effective stress levels at the end of consolidation have been observed to be generally higher than under the initial $K_0$ conditions, it would seem reasonable to expect that the yield stress could be exceeded at later stages in the dissipation process. The large reductions in the coefficient of consolidation that occur as the yield stress is exceeded suggest that linear analyses may underpredict the total time for full dissipation.

Baligh & Levadoux (1980) conducted a parametric study on the effects of assuming different initial distributions of excess pore pressure on predicted dissipation curves. The predictions were made using closed form and numerical (linear uncoupled) solutions for the one-dimensional (radial) consolidation around impervious cylindrical and spherical cavities. They found that the dissipation solutions do not depend on the actual magnitudes of $\Delta u$, but are sensitive to the shape of the normalized pore pressure distribution, which defines the gradient of $\Delta u$ at any point within the soil mass. Among other factors, the authors considered the effects of varying the radial extent, $r_p$, of the initial excess pore pressures, $\Delta u_i$, and of varying the spatial distribution of the initial excess pore pressures within the region between the wall of the cavity (with radius, $R$) and $r_p$. Their findings were as follows:

1) **Spatial distribution between $R$ and $r_p$**: Distributions of $\Delta u(r)$, varying both linearly with radial distance (linear distribution) and linearly with the logarithm of radial distance (logarithmic distribution) were fitted to the measured pore pressure data from various tests sites (mentioned above) such that $r_p$ was held constant at 20R. The two distributions were similar for $5 < r/R \leq 20$, but $\Delta u(r)_i$ was significantly lower for the linear distribution than for the logarithmic distribution for $1 \leq r/R \leq 5$, and at $r = R$, $\Delta u(R)_i$ from the logarithmic distribution was approximately double that from the linear distribution. As a result, the dissipation predicted for the logarithmic distribution is significantly faster than for the linear distribution (by a factor of 5 at 50% dissipation), particularly at early times in the dissipation process. However, the total times for complete dissipation are very similar.

2) **Extent of excess pore pressures**: For an increase in $r_p/R$ from 10 to 20, 50% dissipation was found to take 4 times longer, assuming a linear distribution of $\Delta u(r)_i$, and 2.5 times longer,
assuming a logarithmic distribution. The times for complete dissipation were also significantly longer for greater values of $r_p/R$.

The above findings illustrate the importance of using a reasonably accurate estimate of the initial distribution of excess pore pressures around a pile or piezocone if the dissipation times are to be estimated with even a rough degree of accuracy. Much research effort has been put into developing methods of predicting the distribution of excess pore pressure around cone probes. However, the accuracy of such methods in predicting actual pore pressures measured along the surface of piezocones in intact, non-idealized soils has been limited at best. As a result, the majority of the error associated with using simplified consolidation solutions is usually a result of limitations in the applicability of the initial distribution on which the consolidation solution is based.

The dissipation solutions proposed by Torstensson (1977) and Randolph & Wroth (1979) are based on initial radial distributions (one-dimensional) of excess pore pressure around the cone probe or pile that are estimated from cavity expansion theory, which is described in Appendix A.1. The dissipation solutions by Lebadoux & Baligh (1986) and Houlsby & Teh (1988) are based on initial two-dimensional (vertical and radial) distributions of excess pore pressure around the cone probe that are estimated from the strain path method (SPM), which is described in Appendix A.2.
Figure 2.1  Radial Distribution of Shear Strain Predicted Around Cylindrical Cavity Expanded from Zero Initial Radius under Undrained Conditions

Shear Strain: \( \gamma = \varepsilon_\theta - \varepsilon_r \)

Radial Distance from Center of Cavity \((r/R)\)

R = cavity radius
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Linear-Elastic, Perfectly Plastic (EL-PL): $G/s_u = 350$

Non-Linear, Strain-Softening (NL-SS)

$\gamma_f = 0.3\%$ (EL-PL)

$\gamma_f = 2.0\%$ (NL-SS)

$G_{\text{NL-SS}} = G_{\text{EL-PL}}$ at $\tau/s_u = 0.63$

$G_{\text{max}}/s_u = 875$ (NL-SS)

$\tau/s_u = 0.25$ at $\gamma = 100\%$

Figure 2.2: Comparison Between Idealized Elastic-Plastic Stress-Strain Model and Non-Linear/Strain-Softening Stress-Strain Behaviour
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Figure 2.3: Radial Distribution of Shear Stress and Total Principal Stresses from Cylindrical Cavity Expansion Theory
Figure 2.4: Measured Excess Pore Pressures at Different Clay Sites due to Installation of Conventional Piles (from Lebadoux & Baligh (1980))
Radial Distributions based on contours from Fig. 2.15

Radial Distance from Cone Centerline - $r/R$

Figure 2.5: Radial Distribution of Excess Pore Pressure at Different Locations Along Shaft of Cone Probe in Normally Consolidated Boston Blue Clay from SPM Predictions (after Baligh & Levadoux, 1980)
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intact structure  \[(s_u)_o\]

Shear Stress - \(\tau\)

\[(s_u)_{C}\]

Sedimentation

Compression Line (SCL)

Destructured Compression Line (DCL)

Reduced salt content due to leaching

\((\sigma'_{vy})_D, \sigma'_{v}\)

\(\Delta e_s(t)\)

UnDest\(^*\)

\(\sigma'_{vo}(\sigma'_{vy})_h\)

\(\sigma'_{vy}A\)

\(\sigma'_{vy}P\)

DrDest\(^*\)

Reconsolidation of Destructured Soil

\(e_0\)

\(e_c\)

Void Ratio – e

Effective Stress – \(\log \sigma'_{v}\)

Shear Strain – \(\gamma\)

destructuring due to \(\gamma\)

Destructured, reconsolidated

Drained 1-D Compression from Intact State

*Note:
UnDest = Undrained Destructuring
DrDest = Drained Destructuring

Figure 2.6: Development of Micro-Structure and Behaviour of Structured and Destructured Soils in One-Dimensionnal Consolidation and Undrained Shear
Figure 2.7: Cylindrical Shear Method (after Narasimha Rao et al., 1993)

Figure 2.8: Model Piles Pulled Out of Very Soft Clay (after Narasimha Rao et al., 1989)

Figure 2.9: Pile Capacity vs. Helix Spacing for Different Soil Strengths (after Narasimha Rao & Prasad, 1993)
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Figure 2.10: Assumed Cylindrical Shear Surface for Tapered Helix Groups (after Hoyt & Clemence, 1989)

Figure 2.11: Individual Plate Bearing Method (after Hoyt & Clemence, 1989)
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Figure 2.12: Capacity vs. Torque for Piles with Different Numbers of Plates and Different Shaft Diameters (after Narasimha Rao et al., 1989)

Figure 2.13: Capacity/Torque Factor vs. No. of Helix Plates (after Narasimha Rao et al. 1989)
3.0 TEST SITE DESCRIPTION

3.1 SITE LOCATION AND GENERAL SURFICIAL GEOLOGY

The test site is located under the Highway 99A overpass over Colebrook Road and the adjacent BC Railway (BCR) line, in South Surrey, B.C., as shown on Figure 3.1. This site is located in the northwest corner of the Serpentine River Lowland, 2.5 km east of the sea at Mud Bay. The ground surface in the vicinity of the overpass is very flat, lying at or slightly below sea level, but slopes upward at a moderate grade beyond the edge of the lowlands just north of the overpass.

The subsoils in the western region of the Serpentine River Lowland are Salish Sediments, which are post-glacial deposits of the Quaternary period that were laid down between 10,000 and 5,000 years ago, and include both terrestrial and marine sediments (Armstrong, 1984). Since these sediments were deposited after glaciation and there is no evidence of unloading by erosion, it is unlikely that they have been subjected to any past pressures in excess of the existing overburden pressure. It is possible, however, that the upper soils have been subjected to a light overconsolidation due to increased effective stresses resulting from sea levels lower than the present day level and/or from lower groundwater levels.

3.2 CONSTRUCTION OF COLEBROOK ROAD OVERPASS

The Colebrook Road overpass was constructed between 1971 and 1974 and consists of a 384 m long bridge structure supported by 17 piers with concrete abutments founded on approach embankments. Each pier consists of 4 concrete columns which are each supported by a row of 3 precast reinforced concrete piles (300 mm wide hexagonal cross-section). Prior to construction of the approach embankments in 1971, vertical sand drains (0.4 m diameter) were installed at 4.3 m spacing below the embankment fill to accelerate consolidation within the soft compressible sediments. The sand drains extend to the edges of the embankment footprint.

The test site is located between the 2nd and 3rd piers from the south end of the bridge and is centered under the bridge. The bridge piles, which were driven into the dense sand and gravel below the soft sediments, extend to an average depth that varies from 38 m in the north row of the 2nd pier to 43 m in the south row of the 3rd pier. The distance between these two rows of piles is approximately 19 m. This was considered to be large enough that two rows of test piles
on 3 m centers) could be installed between the piers at a sufficient distance beyond the zone of potential soil disturbance due to the installation of the bridge piles.

The north toe of the south embankment, which is up to approximately 6 m high, is located about 40 m from the centerline of the test site. Therefore, the approach embankment is not expected to have influenced the stress history of the sediments below the test site. Similarly, the road embankments, which are located to the east and to the west of the bridge, are offset at least 15 m from the edges of the test site, and so are not expected to have influenced the soils within the 10 m depth that is of interest in this study.

3.3 SOURCES OF SUBSURFACE INFORMATION

3.3.1 Previous Subsurface Investigations

The Ministry of Transportation and Highways (MoTH) carried out a subsurface investigation along the alignment of the bridge and its approaches prior to construction of the overpass. This investigation included a number of boreholes advanced using a diamond drill, as well as dynamic cone penetration test (DCPT) probeings to determine the depth and density of the competent bearing stratum below the soft sediments. Shelby tube samples of the soft soil were obtained from the boreholes and field vane shear strength testing (FVST) was carried out at some depths. Laboratory testing on the Shelby tube samples included index testing, laboratory vane testing, unconsolidated undrained (UU) triaxial strength testing, and consolidated triaxial compression testing to obtain friction angles and consolidation parameters.

The results of the MoTH investigations, the construction of the bridge and approach embankments and the observed settlements below the embankments, are summarized in a paper published by Crawford & deBoer (1987). A sample MoTH test hole log (TH#2) for a boring located just north of the BC Railway line is included in Appendix C.

Additional in-situ and laboratory testing was carried out between 1988 and 1989 by the University of British Columbia in an area near the pier closest to the south approach embankment. This testing included:
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- two FVST borings, where tests were carried out at 1 m intervals of depth using both a Nilcon vane boring apparatus, which includes a slip coupling to measure rod friction, and a Geonor vane borer that has an outer casing to eliminate friction on the torque rod,
- an electronic piezo-cone penetration test (CPTU) sounding with pore pressure dissipation tests at 6 different depths,
- a flat dilatometer test (DMT) sounding with pore pressure dissipation testing at 2 different depths,
- laboratory consolidation testing using a fixed-ring oedometer.

The purpose of this previous UBC investigation was to compare settlement predictions made using the results of in-situ and laboratory tests to the observed consolidation below the approach embankments at the Colebrook overpass. The results of this investigation were published by Crawford & Campanella (1991).

3.3.2 Investigation During Present Study

For the present study, a detailed in-situ site characterization program was carried out at the pile test site (between the 2nd and 3rd piers) prior to installing the test piles. The locations of the various test holes are indicated on the site plan on Figure 3.2. The program included:

- shallow solid stem auger holes at two different locations (AH-1 and AH-2) in order to visually inspect and characterize the highly variable surficial soils within 2.5 m of ground surface; and deeper auger holes (AH-3b/3c) to obtain continuous piston tube samples between 1.8 and 8.6 m depth, which were logged for visual evidence of layering and to identify organic or stony inclusions or marine fossils,
- laboratory index testing on samples to determine natural moisture content, Atterberg limits, grain-size distribution, organic content, and salt content,
- FVST borings at two locations (VH-1 and VH-2) using the Nilcon vane boring apparatus, with peak and remoulded strength tests carried out at 1.0 m intervals of depth (VH-1 and VH-2 tests offset by 0.4 m elevation),
- installation of a nested pair of push-in electric piezometers, with filter depths at 3.6 and 8.1 m, to measure background piezometric conditions including vertical seepage gradients,
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- CPTU soundings at 5 different locations, where pore pressure measurements were obtained at the standard position behind the shoulder of the cone (U2, at CPT-2,5,6,7), behind the friction sleeve (U3, at CPT-5&6), and midway up the cone face (U1, at CPT-1),

- CPTU pore pressure dissipation testing (at CPT-5 and CPT-6) with simultaneous U2 and U3 measurements at 7 different depths, including the 5 different depths at which pile piezometer filters were located, and

- shear wave travel time measurements during 3 different cone soundings (CPT-5,6,7) using the UBC cone which includes a seismic module located behind the cone (SCPT).

A description of the FVST, CPTU and SCPT apparatus and procedures is provided in Appendices D & E. A summary of the various types of cone tests carried out during this study is provided in Table 3.1.

Table 3.1
Summary of Cone Penetration Tests Carried Out in This Study

<table>
<thead>
<tr>
<th>Cone Test No.</th>
<th>Cone Penetrometer ID</th>
<th>Tip Capacity (MPa)</th>
<th>Measurements during Tests</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT-1</td>
<td>ConeTec (10 cm²)</td>
<td>24.5</td>
<td>( q_c, f_s, u_1 )</td>
</tr>
<tr>
<td>CPT-2</td>
<td>ConeTec (15 cm²)</td>
<td>4</td>
<td>( q_c, f_s, u_2 )</td>
</tr>
<tr>
<td>CPT-5</td>
<td>UBC-10 (10 cm²)</td>
<td>98</td>
<td>( q_c, f_s, u_2, u_3, u(t)_2, u(t)_3, \text{seismic, temp} )</td>
</tr>
<tr>
<td>CPT-6</td>
<td>UBC-10 (10 cm²)</td>
<td>98</td>
<td>( q_c, f_s, u_2, u_3, u(t)_2, u(t)_3, \text{seismic, temp} )</td>
</tr>
<tr>
<td>CPT-7</td>
<td>UBC-10 (10 cm²)</td>
<td>98</td>
<td>( q_c, f_s, u_2, \text{seismic} )</td>
</tr>
</tbody>
</table>

Legend:  
\( q_c \) = tip resistance  
\( f_s \) = sleeve friction  
\( u_{1,2,3} \) = penetration pore pressures measured on face of cone, behind shoulder and above friction sleeve, respectively  
\( u(t) \) = pore pressure dissipation test  
\text{seismic: shear wave travel time measurements}  
\text{temp: internal temperature measurements}

A summary of the general subsurface conditions at the test site is given in Section 3.4 below, while more detailed interpretations of the in-situ test results from this investigation are provided in Chapter 4.
3.4 SUMMARY OF SUBSURFACE CONDITIONS

3.4.1 Stratigraphy

The ground surface elevation at the pile test site lies below sea level, varying between -1.1 and -1.3 m. The near-surface soil stratigraphy is shown on Figure 3.3. The test site is covered with a 0.5 to 0.7 m thickness of fill material overlying a 0.2 to 0.3 m thickness of firm to stiff peat which formed the original ground surface. The peat is underlain by a layer of firm clayey silt interbedded with seams of fine sand to sandy silt which extends to about 2 m depth, and which likely originated as deltaic fluvial sediments. Between about –3.2 and –4.1 m elevation, the soil is predominantly soft silty clay with significant inclusions of grasses and other plant stalks. A salt content of 20 g/L was measured from a sample of these sediments, which indicates that they were deposited in a salt-water environment, likely within the inter-tidal zone between the river delta and Boundary Bay.

These surficial soils are underlain by an extensive deposit of soft clayey silt to silty clay, containing occasional shells and shell fragments. This deposit of marine sediments extends to a depth of at least 35 m below the pile test site, based on the stratigraphic cross-section compiled by MoTH and the tip depths of the bridge piles on either side of the site. The focus of this study was on the upper portion of this deposit, between –4 and –10 m elevation, in which the test piles mobilized their capacity. The upper 0.6 m thickness of this deposit (between –4.1 and –4.7 m elevation) is somewhat stiffer than the underlying sediments. This suggests that there may have been some light desiccation within this zone, possibly as a result of lower groundwater levels in the past. The marine silt and clay appears to have a massive macro-structure down to at least –10 m elevation (the limit of sampling during this study), with no visual indications of any distinct layering. Laboratory testing during this study indicates that the amorphous organic content of the silt and clay deposit down to –10 m elevation is fairly uniform at about 1% of the dry soil weight (Dolan, 2001).

3.4.2 Index Properties

The moisture contents and Atterberg limits of the clays and silts obtained from the piston tube samples between –3 and –10 m elevation are also plotted on Figure 3.3. Below the crust (below –4.7 m), the average natural moisture content \( w_n \) of the marine clayey silt is 42%+-3% and the
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average liquid limit \( (w_{LL}) \) is 40%+/−4%. The plasticity index \( (I_p) \) is plotted on Figure 3.4a. The average \( I_p \) is 13.5%+/−4.5%, and the plasticity is observed to be at a minimum within the crust at about −4.5 m elevation and at −7.3 m elevation. Zones of higher plasticity soil \( (I_p \) of 18 to 21 percent) are observed below −8 m elevation.

The moisture content of the clayey silt is generally above its liquid limit, with liquidity indices \( (I_L) \) typically between 1 and 2, as shown on Figure 3.4b. Both of the Atterberg limit determinations below −9 m elevation were carried out on samples with moisture contents corresponding to localized minimums, and therefore the apparent trend of \( I_L \) less than 1 below −8 m elevation may be misleading.

From the measured plasticity indices and liquid limits, the resulting positions of the samples relative to the A-Line on a standard plasticity chart are shown with depth on Figure 3.4c. Most of the samples tested plotted below the A-Line, indicating that the deposit is comprised mainly of clayey silt. This is consistent with clay-size fractions of between 25% and 30%, which were measured in hydrometer tests carried out on two of the low-plasticity samples (from −5.85 m and −8.6 m elevations), as shown on Figure 3.5.

The index property measurements obtained during this investigation are in general agreement with the average results from the MoTH testing of samples obtained throughout the marine silt and clay deposit along the overpass alignment, as published in Crawford & deBoer (1987): \( w_n \approx 45\% \), \( w_{LL} \approx 36\% \) and \( I_p \approx 11\% \) (and \( I_L \approx 1.8 \) can be inferred).

Assuming a specific gravity \( (G_s) \) of 2.75, the natural moisture contents determined in this study indicate that, below the crust (i.e. below −4.6 m), the in-situ void ratio \( (e_o) \) is about 1.16+/−0.09, and the unit weight \( (\gamma) \) is about 17.8+/−0.3 kN/m\(^3\). Assumed unit weights were assigned to each of the soil types encountered in the upper 2 m and an average \( \gamma \) of 17 kN/m\(^3\) was estimated from the layer thickness measurements at each of the solid-stem auger holes.

### 3.4.3 Groundwater Conditions and Salt Content

Piezometers located within the upper 10 m of the silt and clay deposit indicate that the water table is typically around −2 m elevation and that there is an upward hydraulic gradient which is typically between 5% and 10%. This upward gradient is likely due to groundwater recharge.
from the upland area just north of the site, which seeps up from the more permeable sand and gravel underlying the marine silt and clay. During the dry summer months, the water table was observed to drop to as low as –2.7 m elevation with a corresponding increase in upward gradient to as high as 18%.

Salt contents of 14 to 15 g/L and 11 to 12 g/L were measured from samples above and below -6.8 m, respectively, as shown on Figure 3.4b. These salt contents are less than half the typical depositional salt content of marine clays, which indicates that some leaching has occurred, likely as a result of the upward flow of groundwater at the site. This suggests that the silt and clay at this site has a metastable microstructure resulting from the salt-water depositional environment and the subsequent leaching of salt ions from the soil fabric, which would explain the high liquidity indices that were measured.

3.4.4 Field Vane Shear Strengths and Sensitivity

Vane borings VH-1 and VH-2 were carried out during this study on opposite sides of the test site, as indicated on Figure 3.2. At each location, tests were carried out at 1 m intervals, but the two sets of tests were offset by about 0.4 m elevation so that the results from the two borings could be superimposed to obtain more detailed vane strength profiles for the site, which are shown on Figure 3.6.

The peak undrained shear strength of the intact silt and clay measured with the field vane, \( (s_u)_{FV} \), at VH-1/VH-2 is plotted with depth on Figure 3.6a, along with the strengths reported by Crawford & Campanella (1991) from vane tests at a nearby location using both the Nilcon and Geonor vane boring systems. It appears that \( (s_u)_{FV} \) measured at VH-1/VH-2 is generally in good agreement with values reported by Crawford & Campanella (1991) within the marine clayey silt below –4.1 m elevation, generally ranging from about 15 to 30 kPa in the upper 10 m.

The remoulded shear strengths, \( (s_u)_{rem} \), measured during this study using the Nilcon vane apparatus are also plotted on Figure 3.6a along with \( (s_u)_{rem} \) measured by Crawford & Campanella (1991) at the nearby test site using the Geonor vane apparatus, which has a cased torque rod. It can be seen that the measured \( (s_u)_{rem} \) from this study is in very good agreement with that determined by Crawford & Campanella, ranging from 2 to 0.7 kPa within the marine clayey silt below the crust.
Due to these very low remoulded strengths, the sensitivity, $S_t = \frac{(s_u)_{peak}}{(s_u)_{rem}}$, determined from the field vane measurements is very high. Profiles of sensitivity determined from the VH-1/VH-2 Nilcon vane measurements and from the Crawford & Campanella (1991) Geonor vane measurements are shown on Figure 3.6b. The sensitivity appears to increase approximately linearly with depth from a minimum of 6 to about 40 at –12 m elevation. The observed scatter in the $S_t$ values may be due in part to small errors in the measured remoulded strengths, which have a large impact on the calculated values of $S_t$ since $(s_u)_{rem}$ is so small. Even higher sensitivity, in the range of 50 to 75, was measured by Crawford & Campanella between –12 and –17 m.

### 3.4.5 Stress History

The undrained shear strength of normally consolidated soils tends to be stress-level dependent, and therefore, the ratio of $s_u$ to the effective overburden pressure, $\sigma_v^*$, is commonly used as an indicator of stress history (Schmertmann, 1978). The $s_u/\sigma_v^*$ ratio determined from the VH-1/VH-2 shear strength profile is plotted on Figure 3.6c. The $s_u/\sigma_v^*$ ratio is significantly higher within the crust at about –4.4 m elevation than below the crust, where the $s_u/\sigma_v^*$ ratio ranges from 0.5 to 0.3 and generally decreases with depth. Kokan (1998) suggests that an appropriate strength ratio for normally consolidated plastic soils in the Lower Fraser Valley is 0.21 when $I_p$ is between 15 and 30. The ratio of the in-situ value of $s_u/\sigma_v^*$, shown on Figure 3.6c, to $(s_u/\sigma_v^*)_{NC} = 0.21$ suggests that the clayey silt below the crust is lightly overconsolidated, while the crust is moderately overconsolidated.

In addition to the light overconsolidation which may have resulted from possible drops in sea level and/or groundwater levels (particularly at shallow depths), it is quite likely that the marine clays and silts have undergone quasi-preconsolidation due to aging and possibly due to thixotropic hardening. The effects of aging and thixotropic hardening on the yield stress of the soil were described qualitatively in Section 2.2.4.

The above interpretation is inconsistent with the preconsolidation pressures, $\sigma_p^*$, reported by Crawford & Campanella (1991), which were obtained from one-dimensional consolidation tests based on Casagrande constructions on void ratio – effective stress (e-log$\sigma$’) curves and/or on cumulative work curves. The e-log$\sigma$’ curves published in this paper and in the subsequent paper by Crawford et al. (1994) all exhibit very gradual transitions from recompression to virgin
compression which is uncharacteristic of highly sensitive soils and is likely a result of sample disturbance. Therefore, the published values of $\sigma'_p$, which were close to and sometimes less than the in-situ vertical effective stress, $\sigma'_v$, at the sampling location, are probably not representative of the true vertical yield stress of the soil.

A more detailed examination of the $s_u/\sigma'_v$ ratio and of the overconsolidation ratio of the soil is given in Section 4.2.

### 3.4.6 Cone Penetration Profiles

Profiles of corrected tip resistance, $q_T$, sleeve friction, $f_s$, and excess penetration pore pressure, $\Delta u$, measured at the U2 filter position behind the shoulder of the cone, are plotted on Figure 3.7.

Between –4.1 m and about –4.6 m elevation, there is a distinctive peak in $q_T$ and $f_s$ which corresponds to the upper crust of the marine deposit. The tip resistance is relatively uniform between –4.6 and –7 m elevation, as was observed from the vane strength profile. The clayey silt below –7 m is noticeably stiffer than between –4.6 and –7 m, and has greater variability down to about –10 m elevation. This is consistent with the greater variability in the measured moisture contents between –6.4 and –10 m elevation.

There is a second peak in the measured sleeve friction between –7.2 and –7.65 m elevation, which may also be a result of light desiccation corresponding to the top of the lower, stiffer layer of soil. The larger frictional resistance in this zone may be indicative of higher lateral stress and/or higher large-strain shearing resistance, possibly as a result of stronger or more resilient inter-particle bonding due to desiccation.

The excess pore pressure measured behind the shoulder of the cone increases with depth. The pore pressure response of the soil to cone penetration will be discussed in more detail in Section 4.4.

### 3.4.7 Engineering Parameters from Previous Testing

A drained friction angle, $\phi'$, of 35° was reported in Crawford & deBoer (1987) based on triaxial compression tests carried out on samples from 7 m depth from a borehole located north of Colebrook Road and the BC Rail line.
The values of the vertical coefficient of consolidation, $c_v$, published in Crawford & Campanella (1991), were determined from constant rate of strain consolidation tests at stresses in the normally consolidated range, and vary between $0.3 \times 10^{-3}$ and $2.8 \times 10^{-3} \text{ cm}^2/\text{s}$. However, these values cannot be used directly to compare with values obtained from CPTU dissipation tests where the effective stresses during reconsolidation are in the overconsolidated range. For testing carried out at $-12.7 \text{ m}$ elevation, values of $c_v$ decreased from $8 \times 10^{-3} \text{ cm}^2/\text{s}$ in the overconsolidated range (at a void ratio 0.025 less than at the start of the test) to a minimum of $2 \times 10^{-3} \text{ cm}^2/\text{s}$ in the normally consolidated range. Furthermore, if sample disturbance resulted in an additional reduction in void ratio, the value of $(c_v)_{OC}$ reported above would likely be less than the true in-situ value. A value of the horizontal coefficient of consolidation, $c_h$, of $20 \times 10^{-3} \text{ cm}^2/\text{s}$ was determined by Crawford & Campanella (1991) from a CPTU dissipation test at 10 m depth.

Crawford & deBoer (1987) reported values of the compression index, $C_c$, from one-dimensional consolidation tests on samples of intact soil from the upper 16 m that ranged from 0.25 to 0.46. However, these samples appear to have been affected to some extent by sample disturbance. Crawford et al. (1994) attempted to correct the values of $C_c$ for the in-situ conditions using the Schmertmann (1955) method and assuming that $\sigma'_p$ is equal to $\sigma'_vo$ under the road embankment where the samples were obtained, and obtained corrected $C_c$ values that ranged from 0.39 to 0.74. The ratio of the expansion index, $C_e$, obtained from curves published by Crawford et al. (1994), to the corrected value of $C_c$ at elevations of approximately -3.5 m and -12.7 m are 0.03 and 0.06, respectively.

A coefficient of secondary compression, $C_\alpha$, of 0.015 was back-calculated from 12 years of observed settlement under the south embankment of the Colebrook overpass (Crawford & deBoer, 1987). A $C_\alpha$ of 0.014 was also reported based on laboratory tests with radial drainage.
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Figure 3.1: Location of Test Site
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Figure 3.2: Test Hole and Test Pile Location Plan
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Figure 3.3: Soil Stratigraphy at Test Site

Ground Surface at approx. −1.3 m elev.

FILL

PEAT

CLAYEY SILT with Sandy Silt Seams

Organic SILTY CLAY

Marine CLAYEY SILT to SILTY CLAY

0% 10% 20% 30% 40% 50% 60% 70% 80%
Water Content

Elevation (m)

Moisture Content
(Auger Samples)

Moisture Content (Piston Samples)

Atterberg Limits
Figure 3.4: Variation of Index Properties with Depth
Figure 3.5: Grain Size Distributions from Hydrometer Tests (after Dolan, 2001)
Figure 3.6: Variation of Field Vane Shear Strength Test Results with Depth
Figure 3.7: Example of Cone Penetration Test Results (CPT-7)
4.0 EVALUATION OF SITE-SPECIFIC SOIL PARAMETERS

The results of a comprehensive program of field vane shear strength testing (FVST) and piezocone penetration testing (CPTU) are discussed in this chapter. Some of the cone penetration tests included shear wave travel time measurements (seismic cone penetration tests - SCPT) and pore pressure dissipation tests. The FVST and CPTU equipment and test procedures are described in Appendices D & E, respectively. Methods of interpreting the torque applied to the vane and the resulting rotation of the vane from measurements taken at the top of the vane rods are described in Appendix D.3. The resulting torque and rotation measurements during the FVST’s are also summarized in Appendix D.4. The sources of potential error in calculating corrected tip resistance ($q_T$), sleeve friction ($f_s$) and pore pressure ($u$) from the CPTU field measurements are discussed in Appendix E.3.1.

Using the data from the FVST and CPTU (and SCPT) tests, values for the following key engineering parameters could be estimated, for use in the interpretation of the observed pile behaviour:

- undrained shear strength - $s_u$,
- undrained strength ratio - $s_u/\sigma'_v$,
- overconsolidation ratio - OCR,
- shear stiffness - $G$,
- rigidity index - $I_r = G/s_u$,
- excess pore pressure generation parameters - $\Delta u/\sigma'_v$ and $\Delta u/(q_T - \sigma_v)$, and
- excess pore pressure dissipation characteristics.

The magnitudes and trends with depth of each of the above soil characteristics will be discussed in the remainder of this chapter.

4.1 UNDRAINED SHEAR STRENGTH

It is widely accepted that the undrained shear strength of a soil is not a unique parameter but depends on a number of factors, which include shear mode and strain rate, in addition to stress level and stress history. The reference $s_u$ used in this study was measured by the field vane (i.e. $(s_u)_{FV}$) and therefore corresponds to the strength along a vertically oriented cylindrical plane with shearing in the circumferential direction. This mode of shearing is expected to be reasonably
applicable to the cylindrical shearing inferred along the surface of the grout column of the helical test piles, and along the outside of the closely spaced helix plates, although the direction of shearing in these cases is vertical rather than circumferential. In bearing-type failures, such as are inferred to occur below widely spaced helix plates, the mobilized $s_u$ will probably be some average of $s_u$ mobilized in triaxial compression (TC), $s_u$ mobilized in direct simple shear (DSS) and possibly, $s_u$ mobilized in triaxial extension (TE). Lefebvre et al. (1988) found that $s_u$ obtained from the arithmetic average of the TC, DSS and TE stress-strain curves was very similar to that measured by the field vane in Champlain Sea clay. Thus, $(s_u)_{FV}$ should be reasonably applicable to $s_u$ mobilized by the helix plates in bearing-type failures, although $(s_u)_{FV}$ may be slightly lower if the actual contribution of TE-type shearing is small.

The method of calculating $s_u$ from the torque measurements taken during the FVST is described in Appendix D.3.1. For the purposes of this study, it was desired to have detailed, continuous profiles of undrained shear strength throughout the thickness of soil influenced by the test piles. Therefore, the near-continuous cone measurements were correlated empirically to the values of $(s_u)_{FV}$ determined from the FVST measurements obtained during this study, using depth-specific cone-strength correlation factors ($N_{KT}$ & $N_{\Delta u}$), as described in Appendix E.3.2.

The continuous strength profiles derived from the CPTU measurements are presented on Figure 4.1, along with the measured $(s_u)_{FV}$ from the vane tests carried out during this study (VH-1/VH-2) and $(s_u)_{FV}$ reported by Crawford & Campanella (1991). The CPTU-generated strength profiles are in good agreement with the measured $(s_u)_{FV}$ values. The $(s_u)_{FV}$ measured in this study is very consistent at about 20 kPa above −7.5 m elevation. The only exceptions to this appear to be within the crust between −4.1 and −4.6 m, where $s_u$ is higher, and immediately below the crust where $s_u$ is slightly lower. Below −7.5 m elevation, $s_u$ appears to be more variable with measured $(s_u)_{FV}$ ranging between 20 and 30 kPa and averaging about 25 kPa.

4.2 Undrained Strength Ratio & Overconsolidation Ratio

A knowledge of the overconsolidation ratio, OCR (where OCR is herein defined as the ratio of vertical yield stress, $\sigma'_{vy}$, to in-situ vertical stress, $\sigma'_{vo}$), throughout the depth of interest in this study was important to allow interpretation of the observed pore pressure response of the soil.
The ratio of the in-situ value of $s_u/\sigma'_{vo}$ to the corresponding ratio in a normally consolidated state, $(s_u/\sigma'_{vo})_{NC}$, is an indicator of the extent of overconsolidation of a clay. The average of the CPT-interpreted profiles of $s_u$ from each of CPT-1,2,7 are shown normalized by the in-situ vertical effective stress ($\sigma'_{vo}$) on Figure 4.2. Below the crust, the $s_u/\sigma'_{vo}$ ratio generally decreases with depth, ranging from 0.5 to 0.35. Kokan (1998) suggests that an appropriate strength ratio for normally consolidated plastic soils in the Lower Fraser Valley is 0.21 when $I_p$ is between 15 and 30. Experimental data from Ladd (1991), which is summarized by Kokan (1998), suggest that $(s_u/\sigma'_{vo})_{NC}$ varies little with $I_p$ between 10 and 20, and a value of 0.20 to 0.21 would seem appropriate. Below −13 m elevation, $s_u/\sigma'_{vo}$ interpreted from CPT-7 and vane tests carried out by Crawford & Campanella (1991) decreases from about 0.3 to a minimum of 0.21 below −20 m elevation. This supports the assumption of a normally consolidated $s_u/\sigma'_{vo}$ of 0.21.

Estimates of OCR were made using the CPTU data with 5 different methods, which are described in Appendix E-3.3. Two of these methods are based on relations between OCR and $s_u/\sigma'_{vo}$, two methods are based on relations between OCR and $q_T$-u (cone tip resistance and penetration pore pressure), and one method is based on the difference between u measurements on the cone face and behind the cone shoulder. In an attempt to filter out unreasonable values calculated using the various relations, the maximum and minimum values were ignored and the OCR at each elevation was expressed as a range of the 3 median values for each CPT location. The resulting median bands determined for each of CPT-1,2,7 are presented on Figure 4.3. If it is assumed that the variation of OCR with depth is relatively uniform from one side of the test site to the other, the best estimate of OCR should be provided by the overlapping areas of the ranges from all 3 CPT locations.

Within the upper crust, the soil appears to be moderately overconsolidated (OCR between 4 and 7). Below the crust, the soil is lightly over-consolidated within the depths influenced by the piles. OCR decreases with depth from about 3 at −4.8 m elevation, to about 2 at −7 m elevation. Between −7 and −7.5 m, the OCR appears to increase back up to about 3 before decreasing with depth again to about 1.5 between −11 and −12 m elevation. These values of OCR are inferred to represent the ratio of the vertical effective yield stress, $\sigma'_{vy}$, to the in-situ vertical effective stress, $\sigma'_{vo}$. It should be recognized that $\sigma'_{vy}$ may have been influenced by aging effects (as discussed
in Section 2.2.4) and therefore may be greater than the actual maximum past vertical effective stress applied to the soil.

### 4.3 Shear Stiffness

In lieu of a complete stress-strain curve, simple elastic-plastic total stress models are often used to predict excess pore pressure generation around cone probes and piles. The two parameters that are necessary to describe the elastic-plastic total stress behaviour are the undrained shear strength and an equivalent elastic shear stiffness, $G$. The difficulties in obtaining reasonable estimates of $G$ for a given soil are discussed in Appendix B.2. Due to the high sensitivity and low shear strength of the Colebrook silts and clays, no attempts were made to obtain stress-strain curves from laboratory testing due to expected difficulties with disturbance-induced stiffness reductions.

The small-strain shear stiffness, $G_{\text{max}}$ (or $G_0$), can be readily estimated from in-situ shear wave velocity measurements using the seismic cone, as is described in Appendix E.3.5 and E.3.6. Profiles of $G_{\text{max}}$ and $G_{\text{max}}/s_u$ are presented in Section 4.3.1 below. In order to obtain estimates of $G$ within the larger-strain region up to failure, suitable $G(\gamma)/G_{\text{max}}$ ratios had to be determined. The selection of appropriate $G/G_{\text{max}}$ ratios for various depth ranges to obtain representative profiles of $G/s_u$ with depth is discussed in Section 4.3.2.

#### 4.3.1 Small-Strain Shear Stiffness

The profiles of $G_{\text{max}}$ determined from $V_s$ are presented on Figure 4.4a for each of CPT-5,6,7. The average values of $G_{\text{max}}$ over the depth interval across which $V_s$ was calculated are shown on Figure 4.4a. In both CPT-5&7, shear wave travel times were measured at the standard 1.0 m intervals of depth, while in CPT-6, shear wave travel times were measured across 0.5 m intervals to obtain a more detailed profile. From the profiles on Figure 4.4a, it is apparent that $G_{\text{max}}$ increases approximately linearly with depth from about 10 MPa at –4 m to about 25 MPa at –9 m elevation, which suggests a strong dependence on the in-situ stress level. This dependence is examined in more detail in Appendix E.3.6.
Profiles of $G_{\text{max}}$ normalized by $s_u$ are presented on Figure 4.4b. Within the marine clayey silt below $-4.1$ m elevation, $G_{\text{max}}/s_u$ generally ranges from 700 to 1000, with an average of 860, and increases slightly with depth.

### 4.3.2 Rigidity Index – $G/s_u$

D’Appolonia et al. (1971) suggest that $E_u/s_u$ (where $E_u$ is the undrained Young’s modulus) typically ranges between 1000 and 1500 for low-plasticity inorganic clays of moderate to high sensitivity, and therefore $G/s_u$ should range from 330 to 500 ($G = E_u/3$). These stiffness ratios were based on values of $E_u$ back-calculated from published records of elastic settlements from load tests and full-scale foundations, along with values of $s_u$ measured using the field vane and from triaxial tests.

The rigidity index can be calculated from the $G_{\text{max}}/s_u$ ratio if a suitable shear modulus reduction factor, $G(\gamma)/G_{\text{max}}$, can be estimated, since:

$$G/s_u = G_{\text{max}}/s_u \cdot G(\gamma)/G_{\text{max}}$$  \hspace{1cm} (4.1)

Modulus reduction curves, $G(\gamma)/G_{\text{max}}$ vs $\gamma$, obtained from various laboratory testing programs on fine-grained soils were compiled by Sun et al. (1988). The $G(\gamma)/G_{\text{max}}$ vs $\gamma$ relations were found to be strongly dependent on plasticity index, $I_p$. The composite curves published by Sun et al. (1988) for a range of $I_p$ between 10% and 20% are reproduced on Figure 4.5. In order to use such curves, an estimate of the in-situ shear strain level is required.

From the observed radial distribution of excess pore pressure around the helical piles during installation (which will be described in Section 6.2.3), the initial penetration of the pile shaft was inferred to have caused failure of the Colebrook clayey silt out to a radial distance of about 18 or 19 shaft radii. Thus, based on the distribution of shear strain predicted by cylindrical cavity expansion, the shear strain at failure, $\gamma_f$, for an idealized bilinear elastic-plastic soil would be about 0.3%.

The equivalent $G$ that corresponds to $s_u/G = \gamma_f = 0.3\%$ for an elastic-plastic soil was assumed to be equivalent to the secant shear stiffness at 50% of failure (i.e. $G(\gamma_{50})_{\text{sec}}$ at $\tau = 0.5s_u$) for the true pre-failure stress-strain behaviour of the Colebrook clayey silt. Therefore, $\gamma_{50} \approx 0.15\%$ was
assumed for the true non-linear pre-failure stress-strain curve. At this strain level, $G(\gamma)/G_{\text{max}}$ from Sun et al. (1988) on Figure 4.5 ranges from 0.25 to 0.4 with an average of about 0.3. If a $G/G_{\text{max}}$ of 0.4 is applied to the average value of $G_{\text{max}}/s_u = 860$ from Figure 4.4b, an average $G/s_u$ of about 330 is obtained, which is similar to that inferred from the observed pore pressures around the test piles.

As an independent check that such a magnitude of $G/s_u$ is reasonable for this site, theoretical solutions of the cone factor $N_{KT}$ in terms of the rigidity index, $I_r = G/s_u$, given by Houlsby & Teh (1988) were considered. The expression derived by Houlsby & Teh is provided in Appendix E.3.7. The advantage to back-calculating $G/s_u$ in this way is that $G/s_u$ influences the generation of pore pressure in the soil around a penetrating cylinder in the same way as it influences the generation of a limit pressure on the face of the cone (as measured by $q_T$). However, since $N_{KT}$ is a function of the natural logarithm of $I_r$, the back-calculated values of $G/s_u$ are very sensitive to any errors in the calculated values of $N_{KT}$. The values of $G/s_u$ back-calculated from the $N_{KT}$ factors, which were calculated from the measured vane shear strengths and $q_T - \sigma_{vo}$ from the cone, averaged 380 at CPT-2 (average $N_{KT}$ of 12.6) and 300 at CPT-7 (average $N_{KT}$ of 11.0) between -4 and -11 m elevation. The values are in reasonable agreement with the $G/s_u$ magnitudes estimated above. Of these two values, the $G/s_u$ from CPT-2 may be more reliable since the measured $q_c$ values correspond to around 10% of the full-scale output for the cone used for CPT-2, compared to less than 0.5% of full-scale for the cone used for CPT-7.

Given the strong dependency of the $G/G_{\text{max}}$ ratio on the shear strain level, the $G/G_{\text{max}}$ ratio may vary with depth such that the shape of the $G/s_u$ profile is not necessarily consistent with the shape of the $G_{\text{max}}/s_u$ profile. In order to get an idea of what the trend of $G/s_u$ with depth should be, $G/s_u$ profiles generated using $G/G_{\text{max}}$ ratios of 0.4±0.1 for different characteristic depth ranges were compared to the profile of normalized vane modulus with depth. The vane modulus, $M_v$, is the slope of the pre-failure torque-rotation curve obtained during the vane shear test (the determination of $M_v$ from the vane data obtained in this study is described in Appendix D.4), and is considered to be analogous to the shear stiffness, $G$. When $M_v$ is normalized by the maximum torque applied to the vane during the test, $T_{\text{max}}$ (which is a function of $s_u$), the resulting ratio of $M_v/T_{\text{max}}$ should be analogous to the rigidity index. Therefore, the trend of $G/s_u$ with depth should be similar to the observed trend of $M_v/T_{\text{max}}$ with depth. The $M_v/T_{\text{max}}$ profile, which is shown on Figure D-6c in Appendix D, is more or less constant with depth, with a mean $M_v/T_{\text{max}}$ of 1.39
and a standard deviation of 0.21. Based on a probable range of average $G/s_u$ magnitudes of 325 to 375 between $-4.1$ m and $-11$ m elevation, the following correlation between $G/s_u$ and $M_v/T_{max}$ is tentatively suggested for the Colebrook clayey silt:

$$G/s_u \approx 250 M_v/T_{max} \quad (4.2)$$

Using this relation, the $G/s_u$ profiles for each of CPT-5,6,7 generated using Equation 4.1 are plotted on Figure 4.6 along with the $M_v/T_{max}$ ratios. In order to obtain the best fit of the $G/s_u$ profiles to the $M_v/T_{max}$ profile, the following $G/G_{max}$ ratios were used in Equation 4.1:

- above $-7$ m: $G/G_{max} = 0.45$,
- between $-7$ and $-8$ m: $G/G_{max} = 0.5$,
- below $-8$ m: $G/G_{max} = 0.35$.

Using these $G/G_{max}$ ratios, the mean $G/s_u$ from the profiles for CPT-5&6 between $-4$ and $-11$ m was found to be about 350, with a standard deviation of about 45 to 50. This average is consistent with the estimates discussed previously and the resulting trend with depth is consistent with the expected variations in $G/s_u$ with plasticity and OCR described in Appendix B.2; i.e. lower $I_r$ at higher OCR and at higher plasticity.

### 4.4 Excess Pore Pressure Generation during Cone Penetration

#### 4.4.1 Profiles of Penetration Pore Pressure with Depth

Profiles of excess pore pressure, $\Delta u = u - u_o$, measured at each of the U1, U2 and U3 filter positions during cone penetration are included on Figure E-2a,b,c, respectively, in Appendix E. The excess pore pressures were observed to increase with depth. Therefore, in order to compare the excess pore pressure response at different depths, $\Delta u$ was normalized by the vertical effective stress, $\sigma'_{vo}$. Representative profiles of the normalized penetration pore pressure, $\Delta u/\sigma'_{vo}$, measured at the U1, U2 and U3 locations on the cone are shown on Figure 4.7.

At the U2 and U3 locations along the shaft of the penetrometer, significant drops in $\Delta u$ were measured within the upper crust (between $-4.1$ and $-4.6$ m) where the OCR is known to be significantly higher than in the rest of the soil profile below the crust. This is probably due to the generation of strongly negative $\Delta u_{shear}$ within this moderately overconsolidated zone. The slight
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trend of increasing $\Delta u_i/\sigma_\text{vo}'$ with depth below $-5$ m elevation at the U3 location along the shaft may also be due to increasing $\Delta u_\text{shear}$ as OCR continues to decrease slightly with depth.

At the U1 location, larger and more variable pore pressures are measured within the stiffer zone of soil below $-7.2$ m elevation, as compared to the softer zone of soil between $-4.6$ and $-7.2$ m. In fact, the peaks and valleys in $\Delta u_{U1}$ (Figure E-2a in Appendix E) closely resemble those in the $q_c$ profile (Figure E-1a in Appendix E) at CPT-1. This close agreement is a result of the dominant influence of the mean normal stress increase around the cone on both $q_c$ and $\Delta u$ measured on the face of the cone.

4.4.2 $B_q$ Pore Pressure Parameter

The dependence of $\Delta u$ on the stress increase caused by the penetration of the cone is conventionally assessed using the pore pressure parameter, $B_q = \Delta u/(q_T - \sigma_\text{vo})$. $B_q$ profiles from U1 pore pressure measurements at CPT-1 and from U2 measurements at CPT-2&7 are shown on Figure 4.8.

The variation of $B_q$ with depth is indicative of variations in $\Delta u_\text{shear}$ since the other component of pore pressure, $\Delta u_\text{oct}$, which is generated by the increase in mean normal stress, should be directly proportional to the net tip resistance, $q_T - \sigma_\text{vo}$. Thus, $B_q$ will tend to increase with increasing positive $\Delta u_\text{shear}$ and will tend to decrease with increasing negative $\Delta u_\text{shear}$. Since shear-induced pore pressures tend to become increasingly negative with increasing OCR, there is a strong correlation between $B_q$ and OCR, as indicated on Figure E-13 in Appendix E. For example, the drop in $\Delta u$ at the U2 location within the moderately overconsolidated upper crust, which was observed on Figure 4.7, also appears as a substantial drop in $B_q$, which confirms that strongly negative $\Delta u_\text{shear}$ is being generated along the cone shaft through this zone.

4.4.3 Vertical Distribution of Penetration Pore Pressure along Cone Probe

From the $\Delta u_i/\sigma_\text{vo}'$ profiles shown on Figure 4.7, it can be seen that $\Delta u_{U1} > \Delta u_{U2} > \Delta u_{U3}$, as is typically observed with CPTU pore pressure measurements, due to the reduction in mean total stress moving from the face of the cone up the shaft away from the cone. The penetration pore pressures, $\Delta u_i/\sigma_\text{vo}'$, measured simultaneously at the U2 and U3 locations at several depths, along with $\Delta u_i/\sigma_\text{vo}'$ measured at the U1 location at the corresponding depths in CPT-1, are shown on
Figure 4.9 plotted against their vertical position along the CPTU probe. According to Baligh & Levadoux (1986), the distribution of pore pressure along the face of a 60° cone is relatively uniform, and so it is reasonable to assume that the pore pressure measured at U1 will be about the same as that immediately below the shoulder of the cone. This approximation was used to construct the vertical distributions, shown as dashed curves on Figure 4.9, from the measured data. Clearly, the vertical gradient of pore pressure between the shoulder of the cone and the U2 filter position is very large, despite the low OCR of the Colebrook clayey silt below –5 m. By comparison, the vertical gradient between the U2 and U3 filter is very small.

The shape of the distributions in the lightly overconsolidated soil at the Colebrook site were found to be similar to the distribution based on normally consolidated (NC) Boston Blue Clay (BBC) predicted by Baligh & Levadoux (1986). However, the measured U2 pressures are less than the prediction and the U1-U2 and U1-U3 pore pressure differences are greater than the prediction in almost all cases. The U1 and U3 pore pressures from –9.7 m elevation, where the OCR is estimated to be less than 2, are in excellent agreement with the distribution predicted by Baligh & Levadoux.

4.5 Dissipation Test Results

4.5.1 Dissipation Curves - General

The changes in excess pore pressure, \( \Delta u/\sigma'_{vo} \), at the U2 and U3 filter locations during CPTU dissipation tests carried out during this study are presented as logarithm and square root time plots on Figures 4.10 and 4.11, respectively.

A consistent increase in \( \Delta u \) from the penetration values to some peak value is observed during the early stages of the dissipation process at both the U2 and U3 filter locations. This is commonly observed at the U2 location in overconsolidated soils due to the large vertical gradient in pore pressure between the face of the cone, where large positive \( \Delta u \) is generated, and the U2 filter just behind the shoulder of the cone, where \( \Delta u \) is small or negative. However, in normally to lightly overconsolidated clays, where the difference between \( \Delta u \) generated on the face of the cone and behind the shoulder does not tend to be as large, \( \Delta u(t) \) is typically predicted to decrease monotonically from the moment that penetration is stopped. A common explanation for an initial increase in pore pressure along the shaft that is given by researchers is poor saturation of
the pore pressure cell, which results in time lags to register the full pore pressure in the soil. If poor saturation was the cause of the observed behaviour, it would be expected that the penetration pore pressures measured on different days with different instruments would be significantly different. This was not the case, and the initial redistribution behaviour was found to be very consistent. This behaviour has also been observed at different sites with similar soft fine-grained soils in the Mud Bay area. Thus, it appears more likely that such behaviour is a product of the actual distribution of stresses and pore pressures around the cone, which will be discussed further in Section 4.6.2 below.

At the U2 filter location, an initial slight decrease in $\Delta u$ can be observed on Figure 4.11a which occurs within the first 1 to 2 seconds after stopping penetration and may be a result of the drop in tip load which occurs when pushing is stopped.

The magnitude of $\Delta u_{\text{peak}}/\sigma'_{vo}$ at the U2 location is relatively consistent at 2.3 to 2.4 for all of the tests from –6.0 to –8.8 m elevation, and occurs at times between 30 and 45 seconds after stopping penetration. For the tests at the U3 location, $\Delta u_{\text{peak}}/\sigma'_{vo}$ is also relatively consistent at between 1.8 and 2.0 for all test depths. Greater times to peak are observed at the U3 location, between 55 and 90 seconds for the tests carried out between 4.75 m and 8.5 m depth. For the tests carried out at –4.6 m elevation, at the base of the moderately overconsolidated crust, $\Delta u_i$ is significantly lower than at the locations in the more lightly overconsolidated soil and a greater amount of time is required to reach $\Delta u_{\text{peak}}$. It is interesting to note, however, that values of $\Delta u(t \geq t_{\text{peak}})/\sigma'_{vo}$ at -4.6 m elevation are similar to the values at the other depths at equivalent times. In fact, the dissipation portion of the curves at all test depths has the same shape and all fall within a fairly tight band.

A significant reduction in the dissipation rate was observed between 11 and 19 minutes into the test at –4.6 m elevation in CPT-6 during a period when the rods were not clamped. This apparent reduction in dissipation rate, which was not observed at the U3 location, is probably due to pore pressure generation around the cone resulting from soil creep under the weight of the cone rods when not clamped. The dissipation curve measured at the U2 location at –9.7 m elevation does not follow the trend of the other curves after the peak at 25 seconds, which occurs at a significantly lower magnitude of $\Delta u/\sigma'_{vo}$, until about 5 minutes into the test. The reason for this apparent deviation in behaviour is not clear.
4.5.2 Initial Pore Pressure Redistribution Behaviour

The measured pore pressures at both the U2 and U3 locations during the initial redistribution period in the dissipation test at –6.0 m elevation are presented on Figure 4.12 for comparison with the predicted radial distribution of $\Delta u(r)/\sigma'_vo$ from Levadoux & Baligh (1980, 1986). This comparison appears relevant since it will be seen in Section 6.2.3.2 that the predicted radial distribution for cone penetration in NC BBC is in reasonable agreement with the initial distribution observed around the test piles at radial distances greater than about 5 shaft radii. Also, the magnitude of $\Delta u_i/\sigma'_vo$ that is predicted at the U3 location is in good agreement with the average values observed during CPTU testing in the Colebrook clayey silt below –5 m elevation. It should be noted that the agreement between the pore pressure magnitudes measured in the highly sensitive, lightly overconsolidated Colebrook clayey silts and those predicted for resedimented, normally consolidated BBC is somewhat fortuitous. The mean stress-induced component of pore pressure, $\Delta u_{oc}/\sigma'_vo$, is expected to be higher in the Colebrook clayey silt than in NC BBC since $s_u/\sigma'_vo$ is higher (an average of 0.4 below the upper crust at Colebrook compared to 0.26 (Levadoux & Baligh, 1980) for NC BBC). However, this is may be partially offset by lower $\Delta u_{shear}$ below the upper crust at the Colebrook site, as compared to $\Delta u_{shear}/\sigma'_vo$ of up to 0.4 (Levadoux & Baligh, 1980) predicted near the cone surface in NC BBC.

As was discussed previously in Section 2.1.3, the radial distribution opposite the U3 position predicted by the Strain Path Method suggests that there is a very small radial gradient at radial distances less than about 5 shaft radii (i.e. $r < 5R$). Beyond about 6 shaft radii, $\Delta u(r)/\sigma'_vo$ opposite the U3 location is predicted to decrease with the logarithm of radial distance, at a rate that is similar to that predicted opposite the U2 location for $r > 2R$.

During the redistribution period, $\Delta u$ increases with time at both U2 and U3, as shown by the data points at $r/R = 1$ and data labels on Figure 4.12. By the end of the redistribution period, the magnitude of $\Delta u(t=t_{peak})/\sigma'_vo$ has become significantly higher than the predicted and measured $\Delta u/\sigma'_vo$, particularly at the U3 location. During the same period of time, minimal decreases in the pore pressures within the outer annulus of soil ($r > 2R$ around U2 and $r > 6R$ around U3) are predicted by Levadoux & Baligh (1986).
The inferred radial distributions opposite the U2 and U3 filters at -6.0 m elevation at the end of
the redistribution period (i.e. when net dissipation begins) are shown by the dashed lines on
Figure 4.12. The radial distributions are essentially logarithmic from the shaft of the cone probe
through to the outer annulus of soil (where the distribution was predicted to be logarithmic to
begin with). Thus, it appears likely that the upward redistribution of pore pressure along the
shaft of the cone probe continues until the radial gradient near the shaft increases sufficiently to
induce radial dissipation.

This redistribution effect should be expected at the U2 location since the vertical gradient of $\Delta u_i$
between the shoulder of the cone and the U2 position (as was shown on Figure 4.9) is
significantly larger than the radial gradient that is predicted within the soil opposite the U2
location. However, based on the vertical distributions shown on Figure 4.9, it appears that the
vertical gradient below the U3 position is not that large. And yet, an even larger increase in $\Delta u$
was observed at the U3 position immediately after stopping penetration. Therefore, based on the
large magnitude and relatively fast rate of increase in $\Delta u$ at U3 immediately after stopping
penetration, it is believed that the observed increases in $\Delta u(t)$ at U2 and U3 at early times during
the dissipation process are due to an upward redistribution of total stress. Such pressure
redistribution is believed to occur within the very soft, highly disturbed clayey soil close to the
surface of the cone shaft as a result of the distribution of total stresses induced around the cone
during steady penetration.

When a clay with high liquidity (i.e. with a moisture content above its liquid limit) and high
sensitivity is remoulded under undrained conditions, it has very low shear strength and so
behaves like a viscous fluid. Therefore, the remoulded soil will readily deform in response to
even small pressure gradients, unless movements are contained by stronger material. The zone
of mostly remoulded soil around the cone shaft will extend further in the vertical direction above
the piezo filters than in the radial direction, where the shear strength of the disturbed soil will
tend to increase with increasing distance from the surface of the probe. Therefore, the
impedance to deformation away from the zone of highest pressure will be lower in the vertical
direction than in the horizontal direction. Such deformations will tend to occur until a new state
of equilibrium is established. At this point, the stress within the zone of higher pressure will
have decreased, while the stress within the zone of lower pressure will have increased. This
deformation probably occurs under essentially undrained conditions and so would occur much more quickly than the flow of pore water through a stationary soil skeleton.

4.5.3 Importance of Radial Pore Pressure Distributions on Dissipation

Since it is usually not practical to carry out CPTU dissipation tests to completion in fine-grained soils, due to the long periods of time required, the available dissipation data is normalized and compared to a complete dissipation solution. Dissipation solutions are typically given in terms of $\Delta u(T)/\Delta u_o$, where $T$ is a normalized time factor, and $\Delta u_o$ is the excess pore pressure at time zero during the dissipation process. $\Delta u_o$ is usually taken as the penetration pore pressure, $\Delta u_i$, when $\Delta u(T)$ decreases monotonically for $T>0$. In cases where $\Delta u$ increases initially before exhibiting monotonic decay, it is difficult to compare observed dissipation curves with conventional dissipation solutions that do not consider initial pore pressure redistribution effects.

For dissipation curves that exhibit an initial increase in $\Delta u$, Sully et al. (1999) have suggested normalizing the curves using either of two different methods:

- the “Root Time” correction method, where $\Delta u(T)$ is normalized by a corrected $\Delta u_o$ which is determined by extrapolating back the linear segment of a root time dissipation plot to zero time, or
- the “Log Time” correction method, where $\Delta u(T)$ is normalized by a corrected $\Delta u_o = \Delta u_{peak}$ and zero time is taken as the time at which $\Delta u = \Delta u_{peak}$.

Both methods will produce a dissipation curve that is monotonically decreasing and can therefore be compared directly to theoretical dissipation curves. Based on the redistribution process that is inferred to cause the initial increase in $\Delta u$ along the cone shaft, the “log time” correction method is considered to be a more fundamentally sound means of correcting the dissipation data. Nevertheless, both methods were considered in this study.

Levadoux and Baligh (1980) found that dissipation solutions do not depend on the actual magnitudes of $\Delta u$, but are sensitive to the shape of the normalized distribution of $\Delta u(r)/\Delta u(R)_o$, where $\Delta u(R)_o$ is the excess pore pressure at the shaft of the cone probe at the start of dissipation. The effects of differently shaped radial pore pressure distributions on the predicted dissipation curves were discussed in Section 2.4. Upon stopping penetration, the initial $\Delta u(r)/\Delta u(R)$ distribution around the cone may exactly match the assumed distribution used in the dissipation
solutions. But the initial pore pressure increase near the cone shaft will tend to change the shape of $\Delta u(r)$, as was implied on Figure 4.12. Even after normalizing the radial distribution by a larger value of $\Delta u(R)_o$, as is done inherently when the “root time” or “log time” correction methods are applied to the dissipation curves, this new shape may be significantly different from that which was assumed for the theoretical dissipation solution. As a result, the actual dissipation behaviour may be significantly different from the predicted behaviour, which can make comparisons between the observed and predicted dissipation curves difficult or misleading.

In order to evaluate the applicability of the dissipation solutions by Levadoux & Baligh (1980, 1986) and Teh & Houlsby (1991) to the observed dissipation at the U2 and U3 locations in the Colebrook clayey silt, the theoretical $\Delta u(r)/\Delta u(R)_o$ distributions are compared on Figure 4.13 to those which are inferred for the Colebrook soil. The Levadoux & Baligh distributions are based on the stress-strain and pore pressure generation behaviour of NC BBC, while the Teh & Houlsby distributions are based on an $I_r = 300$, which is close to that estimated for the Colebrook soil. The $\Delta u(r)/\Delta u(R)_o$ distributions predicted by cylindrical cavity expansion theory (for $I_r = 350$) are also shown on Figure 4.13 for comparison. The distributions opposite the U2 and U3 filter positions are shown separately on Figures 4.13a and 4.13b, respectively. The Colebrook distributions correspond to the inferred $\Delta u(r)$ at the end of the initial redistribution period during the dissipation test at –6.0 m elevation in CPT-5 (shown on Figure 4.12), which are normalized by the same values of $\Delta u(R)_o$ which would be used to correct the dissipation curves in the “root time” and “log time” correction methods.

When the inferred radial distribution around the U2 filter position at the end of the redistribution period is normalized by $\Delta u(R)_o = \Delta u(R)_{\text{peak}}$ from the “log time” correction method, the normalized distribution compares reasonably well with the Teh & Houlsby and Levadoux & Baligh SPM distributions as well as the CCE distribution (as shown on Figure 4.13a). The normalized distribution obtained using the extrapolated $\Delta u(R)_o$ from the “root time” correction method falls below the theoretical distributions for $r < 5R$.

When the inferred distribution around the U3 filter position at the end of the redistribution period is normalized by $\Delta u_o$ from either the “root time” or “log time” correction methods, the normalized distributions plot well below the corresponding distributions predicted from the SPM-based analyses (as shown on Figure 4.13b). However, reasonable agreement is observed
between the corrected Colebrook distributions and the distribution predicted by cylindrical cavity expansion theory using an elastic-plastic model with \( I_r = 350 \).

In their research, Baligh & Levadoux (1980) generated dissipation curves using differently shaped initial \( \Delta u(r)/\Delta u(R) \), distributions (discussed previously in Section 2.4). It was shown that radial distributions with higher \( \Delta u(r)/\Delta u(R) \) at a given radial distance, \( r \), (or elevated pore pressures over larger radial distances) resulted in larger times required for a given degree of dissipation. Therefore, it can be expected that a theoretical dissipation curve, \( \Delta u(R,t)/\Delta u(R) \) vs \( \log T \), which is based on a \( \Delta u(r)/\Delta u(R) \) distribution that plots above and to the right of the true distribution within the soil, will plot above and to the right of the true dissipation curve. Based on the comparisons of the inferred \( \Delta u(r)/\Delta u(R) \) distributions at the Colebrook site with the theoretical distributions, the following conclusions can be drawn about the suitability of the SPM-based dissipation solutions for comparing to the corrected Colebrook dissipation curves:

- The closest agreement is expected between the SPM-based dissipation solutions for the U2 filter position with “log time”-corrected U2 dissipation curves. The Teh & Houlsby solution may be slightly better than the Levadoux & Baligh solution for the observed response at the Colebrook site.
- The “root time”-corrected U2 dissipation curves will probably plot below the SPM-based dissipation solutions for the U2 position.
- The SPM-based dissipation solutions for the U3 position will probably plot well to the right of either the “log time” or “root time”-corrected U3 dissipation curves, with the deviation being worse for the “root time”-corrected curves.

### 4.5.4 Estimates of Coefficient of Consolidation

To obtain in-situ estimates of the coefficient of consolidation in the horizontal direction, \( c_h \), it has become standard practice to use the time, \( t \), required to achieve 50% dissipation (i.e. \( \Delta u(t_{50})/\Delta u_o = 0.5 \)) during the CPTU dissipation test, according to the following equation:

\[
c_h = \frac{T_{50} \cdot R^2}{t_{50}} \tag{4.3}
\]
where $T_{50}$ is the normalized time factor corresponding to 50% dissipation, values of which are obtained from theoretical dissipation curves. Danziger et al. (1997) suggested determining $c_h$ by matching the measured dissipation curves, after normalizing by $c_h$, $R$ and $\Delta u_0$ (so that $\Delta u(t)/\Delta u_0$ is plotted against logT), with appropriate theoretical curve(s). It is important, however, to exercise caution when comparing observed and theoretical dissipation data unless it is known that the actual pore pressure distribution around the piezo filter is reasonably congruent with the assumed distribution used to generate the dissipation solution. Otherwise, the theoretical dissipation curve will deviate from the observed curve and attempts to match the observed curves with the solutions can result in serious errors in the estimation of $c_h$, since the time scale is logarithmic. Baligh & Levadoux (1980) suggest that the observed dissipation curve should be reasonably congruent with the theoretical curve if the initial excess pore pressure distributions are similarly congruent.

The shape of the post-peak portion of the corrected U2 and U3 dissipation curves was found to be in reasonably good agreement with the SPM-based dissipation solutions by both Teh & Houlsby (1991) and Levadoux & Baligh (1980, 1986). However, the theoretical dissipation curve by Randolph & Wroth (1979), which is based on an initial $\Delta u(r)$ distribution predicted by CCE theory, was found to have a slope that was significantly flatter than that observed from either of the U2 or U3 dissipation data at Colebrook. Therefore, this solution was not used to obtain $c_h$ estimates.

The $c_h$ estimates obtained from the SPM-based dissipation solutions are plotted with depth on Figure 4.14. Most of these estimates were obtained by matching the dissipation curves generated using both the “log time” and “root time”-corrected data measured at both the U2 and U3 filter positions with the Teh & Houlsby (1991) dissipation curves, using an estimated $I_r$ of 350 for all depths.

For any particular method, the estimated $c_h$ appears to be relatively consistent with depth. For any particular depth, the highest and lowest estimates of $c_h$, obtained by matching “log time” and “root time”-corrected dissipation curves with the Teh & Houlsby curves, differ by a factor of 2 to 3. This is primarily due to the differences between the shape of the radial pore pressure distributions assumed by Teh & Houlsby compared to the Colebrook distributions generated.
using the correction methods by Sully et al. (1999). It can be seen that the magnitudes of the $c_h$ estimates obtained using the various methods are consistently in the following order:

$$c_h(\text{“log time’’-U2}) < c_h(\text{“root time’’-U2}) < c_h(\text{“log time’’-U3}) < c_h(\text{“root time’’-U3})$$

This is consistent with the trends between the amount of deviation observed on Figure 4.13 between the $\Delta u(r)/\Delta u(R)_0$ distributions obtained from the four combinations given above and the corresponding theoretical $\Delta u(r)/\Delta u(R)_0$ distributions used by Teh & Houlsby. Based on the conclusions from Section 4.5.3, the “log time”-corrected U2 data is expected to provide the best estimate of $c_h$ while all of the other methods are expected to over-estimate $c_h$.

The “log time”-corrected U2 data from 3 different depths was also correlated to the Levadoux & Baligh (1980, 1986) dissipation solutions for the U2 location, and the estimated $c_h$ were found to be very similar to those obtained from the Teh & Houlsby solutions. This is to be expected since the Levadoux & Baligh and Teh & Houlsby ($I_t \approx 350$) curves are very similar and plot very close together (as can be seen from Figure B-3 in Appendix B). The Levadoux & Baligh estimates were consistently about 10% to 20% higher than the Teh & Houlsby estimates.

Using the $c_h$ estimates from the “log time”-corrected U2 data, shown by the shaded symbols on Figure 4.14, an average $c_h$ of 0.019 cm$^2$/s (standard deviation of 0.05) is obtained throughout the depth profile. This is consistent with the $c_h$ of 20x10$^{-3}$ cm$^2$/s reported by Crawford & Campanella (1991) based on the data from a CPTU dissipation test at 10 m depth near the Colebrook test site.

### 4.6 SUMMARY OF ENGINEERING PARAMETERS

A summary of the engineering parameters that are of interest in the interpretation of the observed pile behaviour is included in Table 4.1 below. For the purposes of this summary, the soil profile between –4 and –10 m elevation, in which the piles mobilized their capacity, has been divided into 3 sub-layers: an upper crust (–4.1 to –4.7 m elevation), a middle layer of soft clayey silt (–4.7 to –7 m), and a lower layer of soft to firm silt and clay.
## Table 4.1
Summary of Average Engineering Parameters

<table>
<thead>
<tr>
<th>Sub-Layer of Marine Silt &amp; Clay</th>
<th>Elevation Range (m)</th>
<th>Plasticity Index</th>
<th>Peak ((s_u)_{FV}) (kPa)</th>
<th>Peak (s_u/\sigma_v)</th>
<th>G/su</th>
<th>(c_h) (cm²/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Crust</td>
<td>-4.1 to -4.7</td>
<td>8%</td>
<td>25-30</td>
<td>0.9</td>
<td>300-350</td>
<td>0.012</td>
</tr>
<tr>
<td>Middle Clayey Silt</td>
<td>-4.7 to -7</td>
<td>10%</td>
<td>19</td>
<td>0.52-0.45</td>
<td>300-400</td>
<td>0.019</td>
</tr>
<tr>
<td>Lower Silt &amp; Clay</td>
<td>-7 to -10</td>
<td>15%</td>
<td>20-32</td>
<td>0.5-0.35</td>
<td>300-400</td>
<td>0.019</td>
</tr>
</tbody>
</table>
Figure 4.1: Variation of Undrained Shear Strength with Depth from CPTU and FVST Data
Figure 4.2: Variation of Undrained Strength Ratio with Depth
Figure 4.3: Variation of Overconsolidation Ratio with Depth
Figure 4.4: Variation of Small-Strain Shear Modulus and Modulus-Strength Ratio with Depth
Figure 4.5: Typical Shear Modulus Reduction with Strain Level for Plasticity Index between 10% and 20% (after Sun et al., 1988)
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Figure 4.6: Inferred Variation of Rigidity Index with Depth

Note:
G is equivalent elastic shear modulus
G/Gmax = 0.45 above -7 m, 0.5 between -7 and -8 m, 0.35 below -8 m assumed,
Gmax calculated from shear wave velocities based on first cross-over point of polarized waves
Figure 4.7: Representative Profiles of Normalized Excess Pore Pressure During Cone Penetration at Different Locations on Cone Probe
Figure 4.8: Variation of $B_q$ Pore Pressure Parameter with Depth
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Figure 4.9: Distribution of Pore Pressure Along Cone Probe during Penetration
Figure 4.10:
Variation in Excess Pore Pressure with Log Time during Break in Cone Penetration
Figure 4.11: Variation in Excess Pore Pressure with Root Time during Break in Cone Penetration
Figure 4.12: Pore Pressure Redistribution Immediately after Stopping Cone Penetration
Figure 4.13: Comparison of Radial Pore Pressure Distributions after Normalizing by Different “Initial” Shaft Pore Pressures
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**Figure 4.14:** Variation in Estimated Coefficient of Horizontal Consolidation with Depth
5.0 TEST PILES & INSTRUMENTATION

The geometry of the test piles investigated in this study, along with descriptions of the instruments that were used to measure pore pressures and load within the piles, are described in this chapter. The instruments include push-in piezometers and piezo-ports located on the pile shaft for measuring pore pressures within the soil surrounding the piles, and strain gauge installations located on the pile shaft to determine the distribution of load within the piles during load testing. Methods of calibrating and monitoring the instruments are also described.

5.1 DESCRIPTION OF TEST PILES

The geometry of the test piles investigated in this study is shown on Figure 5.1. As for conventional helical piles, the helix plates were all located on a single lead section. The shaft of this lead section consisted of 8.9 cm O.D. (7.4 cm I.D.) steel pipe, which had a closed-ended tip that was bevelled at 45°. For the relatively light structural loads that would be normally carried in soft soils, more slender solid square bars would typically be used for the pile shaft. However, the hollow pipe sections, which are also available commercially, allowed the wiring for strain gauges and pore pressure ports mounted on the pile shaft to be protected internally, and also allowed the vertical alignment of the piles to be profiled using an inclinometer probe.

On conventional helical piles, the diameter of the helix plates (which typically range from 20 cm to 36 cm in diameter) decreases from top to bottom. This is done in an attempt to reduce the amount of torque required to embed the lead section into dense strata, while still providing as much combined bearing area as possible. When installing helical piles within soft, fine-grained deposits, minimal torque is required for installation, and therefore the bearing area of the helix plates can be maximized. Thus, for this study, a uniform helix diameter of 0.355 m was selected for all of the plates.

Previous researchers have determined that the spacing of the helix plates is critical in determining the load transfer mechanism between the helices and the soil, as was discussed in Section 2.3.1. For the piles that are manufactured by Chance Anchors, it is common practice to attach the helix plates to the lead section such that the distance between successive plates (S) is 3 times the diameter (D) of the lower plate. Based on previous research, plates spaced at S/D = 3 would be expected to induce individual bearing failures through the soil below each plate. A
basic pile capacity analysis suggests that for an equal length between top and bottom helices, a
greater capacity could be mobilized in soft fine-grained soils under undrained loading conditions
if the plates were spaced such that a cylindrical failure surface was generated. Previous research
indicates that this occurs when the plates are spaced at S/D ≤ 1.5. Therefore, for this study, the
testing was carried out on piles which had either 3 plates at S/D = 3, or 5 plates at S/D = 1.5,
such that the total length from the top to bottom helix was equal for the two pile types (2.1 m).
The pitch of the helix plates was 7.5 to 8 cm, which is the standard pitch used on helical piles
manufactured by Chance Anchors. Examples of the two types of lead sections can be seen in the
photograph on Figure 5.2.

The piles were all installed to a tip depth of about 8.5 m (approximately -9.8 m elevation). In
order to advance the lead section to the desired embedment depth, extension sections were added
as pile installation progressed. Two 1.4 m long extensions were added, followed by a final 3 m
long extension. The bottom of each extension was belled to fit over the top of the preceding
section, and the connection was made using 3 bolts per connection. Steel discs, 15 cm in
diameter, were located at the bottom of each of the 1.4 m long extension sections, as can be seen
on Figure 5.3. These discs displace the soil away from the pile shaft as the pile is pulled into the
ground, thereby forming an annular cavity around the pile shaft. Once the bottom displacement
disc is pulled a short way into the ground this annular cavity (and the larger diameter pilot hole
for the lead) is filled with liquid grout, as shown on Figure 5.4, and installation is resumed. As
the bottom displacement disc is pulled into the ground, the grout is “pulled down” with it and
more grout is added at the surface as necessary to keep the hole full of grout throughout the pile
installation process. This method of encasing the shaft in a column of grout was developed and
patented by Vickars Developments Co. as the “PULLDOWN™ Pile”. Once cured, the grout
gives the shaft connections rigidity and the larger diameter composite column can mobilize much
greater lateral soil resistance, thereby greatly increasing the buckling resistance of the slender
steel shaft.

The grout used for the helical piles in this study is the standard Type A PULLDOWN Pile Grout
manufactured by Ocean Construction Supplies Ltd. of Vancouver, B.C. This is a non-shrink
grout consisting primarily of Portland cement (Type I or II) with silica fume. Water is added on
site to achieve a target water-cement ratio of 0.28 and a target density of 1820 kg/m³, and a small
quantity of mesh fibres are added to the mix. This grout has a specified final set time of 12 hours
with rated 1-day, 7-day and 28-day compressive strengths of 10, 25 and 35 MPa, respectively. A volumetric expansion of 2 to 4 percent is predicted during curing.

In order to minimize the contribution of the upper 2.5 m thickness of variable surficial soils to the overall pile capacity, a twin-walled PVC casing system was designed to isolate the upper 3 m extension from soil friction. A schematic diagram of this casing system is shown on Figure 5.5. The contact area between the inner casing, which is a 15 cm diameter PVC pipe, and the outer 20 cm diameter PVC casing, is limited to a narrow ring (approx. 25 mm wide) along the bottom of the 20-15 cm reducer (shown in white on Figure 5.6). The base of the inner casing and the bottom of the reducer are supported on an 18 cm diameter disc with an inner ring to center the inner casing relative to the pile shaft, as can be seen on Figure 5.7. Once the twin-walled casing system was pulled into the ground, the annular space between the inside of the inner casing and the outside of the pile shaft was filled with grout to provide greater rigidity for the upper extension section, as can be seen on Figure 5.8. During compressive load testing, the pile shaft, grout column, 18 cm diameter disc and inner casing are pushed into the ground while the outer casing remains stationary within the upper 2.5 m of soil. Thus, the only load which can be transferred from the pile shaft to the outer casing and surrounding soil is by the friction between the smooth surfaces of the inner PVC casing and the inner ring of the reducer.

5.2 INSTRUMENTATION

5.2.1 Amplifier & Data Acquisition System

The data acquisition system had up to 16 channels available for simultaneous dynamic monitoring. A commercial data logging program called Virtual Bench was used to record the voltage output of the electronics during dynamic monitoring. This software was limited to a range of ±10 V with 1 mV resolution on 16 different channels. The software enabled the data sampling rate to be specified and also provided 50/60 Hz signal averaging which was useful in filtering out signal noise.

Because of the limits on the voltage resolution imposed by the software, it was necessary to amplify the output of all of the electronic instruments used in this study before recording them with the data logging program, in order to achieve adequate accuracy. The amplifier unit used in this study had independent amplification settings of 10x, 100x or 1000x, available on all 16
channels. The amplified output was found to be sensitive to temperature changes, among other environmental factors, and so it was necessary to take readings of the baseline amplifier output on each channel at various times during any monitoring period. These intermittent baseline readings were then used to determine the increment in the amplified output that was due to just the raw signal output of the instruments.

### 5.2.2 Piezometers

The piezometers used in this study, which were custom-made at the University of British Columbia (UBC), are of a similar design to that described by Jaakkola et al. (1999). The UBC piezometers feature a stainless steel piezo-cell with a conical tip, which is screwed onto steel pipe and pushed into the ground. Once the cell has been installed to the desired depth, a separate electrical pressure transducer unit can be lowered down the inside of the steel pipe and connected to the cell. This concept is illustrated on Figure 5.9. The piezo-cell contains a fluid chamber that is protected by a cylindrical porous polyethylene filter. The top of the fluid chamber is sealed off using a Swagelock quick-connect valve located inside the piezo-cell. The Swagelock valve remains closed until engaged by its female counterpart, which is fixed to the bottom of the drop-in transducer unit.

An assembled piezometer cell connected to the lead (stainless steel) push pipe is shown at the top of Figure 5.10. A drop-in transducer unit is shown on the lower half of Figure 5.10 connected to the male counterpart of the quick-connect valve (shown by itself immediately below), which is fixed to the stainless steel tip and fluid chamber on the right side of the figure. The stainless steel body and porous filter are also shown immediately above the fluid chamber and valve. A series of cable weights were located above each transducer unit to provide a connection force when engaging the transducer unit, and to resist the subsequent uplift force acting on the valve of the transducer unit due to the pressure in the fluid chamber of the piezo-cell.

The tip of the push-in cells was 25 mm in diameter and included the 20 mm long porous filter. This porous filter was located below a tapered shoulder that provides a gradual transition from 25 mm around the filter to 34 mm diameter where the cell flush-threads to 25 mm I.D. stainless steel pipe (shown at the top of Figure 5.10). The purpose of this 25 mm I.D. pipe, which was approximately 0.40 – 0.45 m long, was to provide a guide for the housing of the transducer unit as it is being dropped onto the Swagelock valve inside the piezo-cell. This lead stainless steel

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The push-in piezo-cells were saturated with de-aired glycerin prior to installation. The transducer units were also saturated with glycerin prior to being connected to the piezo-cells in the ground. It was found that an excellent degree of saturation of this system could be achieved if the piezo-cells were assembled while submerged in glycerin, and if the quick-connect valves were closed while submerged in the glycerin. An effective saturation method evolved during the
successive installation phases of this study. Less than perfect saturation was achieved for the piezometers around the first test pile, TP1. When the transducer units were connected to the piezo-cells, the transducers were monitored to determine the rate at which the pore pressure in the transducer unit equalized with the in-situ pore pressure conditions. It was found that when the piezo-cells and transducers were properly saturated, equalization was essentially complete in about 5 minutes. When the degree of saturation was less than perfect, equalization could take a half hour or more.

After repeated disconnections and reconnections of the transducer units, it was found that the piezometer response was noticeably degraded, even though the transducer units were re-saturated before each connection. This was attributed to air entering the quick connectors before the valves closed as the transducer unit was disengaged. Some other difficulties which were encountered with some of the piezometers included:

- Water leakage into the lead push pipe above the piezo-cell: In a number of cases, water seeped into the piezometers and gradually filled the lead push pipe above the quick-connector on the piezo-cell, which made it difficult to connect a transducer unit by dropping it onto the quick-connector.

- Getting the transducer units into the lead pipes: If the piezometers were installed out of plumb by any significant amount, it was sometimes difficult to get the transducer units to drop into the 25 mm diameter lead pipe. This was due to the fact that the 38-25 mm pipe reducers did not have a tapered internal wall to guide the transducer housing into the smaller pipe.

- Failure to obtain sealed connection between Swagelock quick-connectors: Occasionally, the quick-connectors did not mate up properly or the valve seals failed to provide a proper seal, and gradual pressure bleeds or slow pressure fluctuations were observed during monitoring. It was usually possible to correct this problem by disconnecting and then reconnecting the transducer unit. However, this probably led to some degradation in the degree of saturation, as discussed above, and in some rare cases it was not possible to obtain a satisfactory connection.

- Severe degradation in piezo-cell response with time: The installation of piles TP5 and TP6 was carried out almost 5 months after the piezo-cells for these piles had been installed. When
the piezometers were monitored during connection of the transducer units and during pile installation, the response of the piezometers was very poor. This occurred even though the piezo-cells were saturated in exactly the same manner that the TP3&4 piezo-cells were saturated (and the response was found to be excellent), and excellent saturation of the transducer units was ensured. The reason for this apparent severe degradation in performance with time is unknown, but could be due to smearing of the filters during piezo-cell installation, a build-up of gas around the filter, or somehow a result of slow pressure leaking by the Swagelock valve.

Based on the problems mentioned above, the piezometer design used in this study may not be the most reliable for long-term or repetitive use in low-permeability soils. However, for the purposes of this study, this design offered the following essential advantages over more conventional piezometer designs:

- The ability to insert an inclinometer device down the inside of the piezometer to determine with reasonable accuracy the position of the piezo-cell in the ground, which was made possible because the pore pressure transducer and cable was removable.
- The ability to use a relatively small number of transducer units for a large number of piezometers during different stages of the project.
- The ability to re-check the calibration and baseline readings of the transducers before and after different monitoring phases.

### 5.2.3 Piezo-Ports on Pile Shaft

Piezo-ports, which contained an electric pore pressure transducer with a porous filter, were installed within the wall of the pile shaft on the lead sections, as shown on Figure 5.11 (for a piezo-port at the PP3 location below the bottom helix). The transducer housing was inserted into a round hole that was milled into the pile shaft (29 mm in diameter at the surface of the shaft) and the transducer cable was run up the inside of the shaft to the top of the pile. After installation in the pile shaft, the flat face of the housing was recessed below the surface of the shaft, and was held in place with a retaining clip that was seated in a groove within the wall of the shaft. The filters typically consisted of a flat disc, 19 mm in diameter, which was made out of the same material that was used for the piezometer filters. For the piezo-ports on piles TP5 and
TP6, the circular filters were 22 mm in diameter and had a curved rather than flat surface that roughly matched the curvature of the surface of the pile shaft. The filters were pushed down inside of the housing when saturating the port in the field, and were typically held in place by friction along the inner wall of the housing (the filters were tight-fitting). On piles TP5 and TP6, the filters were epoxied into place. The ring-shaped gap between the filter and the wall of the pile shaft above the recessed face of the housing was typically in-filled with grease, except for on piles TP5 and TP6, where it was in-filled with epoxy.

The pore pressure transducers used in the piezo-ports were the same as were used in the piezometers, but all had 690 kPa capacity. Transducer calibration was carried out and initial baselines were taken in a manner similar to the procedure followed for the piezometers. The piezo-ports were also saturated with de-aired glycerin and the porous filters installed in the field before installation. Monitoring of the piezo-ports was carried out in an identical manner to the monitoring of the piezometers.

This system was easy to saturate in the field, but a perfect degree of saturation may not have been maintained in all of the piezo-ports by the time the filters were below the groundwater table, as suggested by some of the test results. Another problem that was encountered on some of the piezo-ports was that negative excess pore pressures were measured over sustained periods of time after pile installation, before the real behaviour of dissipating positive excess pore pressures was captured. This may have been due to a lack of intimate contact between the soil and the piezo-port filter during pile installation, with a positive response being recorded once the soil had deformed inward and established full contact with the filter. The pressures measured from the piezo-port located on each of piles TP5 and TP6 did not change significantly from the atmospheric pressure during pile installation, during the 19 hours of dissipation or during the load tests. It is not known why this occurred, but the behaviour suggests that the transducer membrane did not respond to pressure changes within the soil. Perhaps this was somehow caused by the epoxy which was used to fix the filter in place on these ports.

5.2.4 Strain Gauge Installations

Each strain gauge installation comprised a pair of strain gauges fixed to the outside of the pile shaft on diametrically opposite sides of the pile. Each gauge had both longitudinal and transverse elements, and the gauge pairs were wired together in a full bridge pattern. Thus, the voltage
output corresponded to the average axial stress in the pile and so bending or eccentric load effects on the measured output were minimized. Every other strain gauge installation also had temperature sensors which were mounted to the outside of the pile shaft in order to assess temperature effects on the output of the strain gauge set. The cables containing the wires for the strain gauges and temperature sensors were run up the inside of the pile shaft, along with the piezo-port cables.

The strain gauges and exposed wiring on the outside of the pile shaft were coated with a polyurethane resin for moisture protection and then covered with mastic. Steel covers were machined which were bolted to the outside of the pile shaft to protect the gauges from damage during transportation and installation, as shown on Figure 5.12. The covers that were located within the grout column (shown on Figure 5.12) protruded 13 mm from the outside of the pile shaft. Slimmer profile covers (7 mm protrusion) were used on the shaft of the lead section to reduce the amount of excess disturbance of the clay caused by the covers during pile installation. The space between the pile shaft and mastic and the inside of the cover was filled with silicone sealant. The covers were only bolted at the bottom, with a layer of flexible silicone between the top of the cover and the pile shaft, to avoid changing the axial stiffness of the pile shaft.

The strain gauges used in this study had a capacity of ±2%, but the axial strain level in the pile shaft during load testing was less than 250 $\mu$ε. Therefore, the strain gauges were functioning at strain levels that were less than 1% of full-scale. To compensate for this, the voltage output of the strain gauges was amplified 1000x, and was calibrated directly to the range of loads expected in the pile shaft during the load tests. A load resolution of 0.06 kN was achieved using the amplifier and data acquisition system. The calibrations were done by placing each gauged section in a large loading frame and applying a compressive load to both ends of the shaft which was measured using a load cell. Before carrying out the calibrations, each pile section was cycled through at least 5 full load-unload repetitions in order to exercise the strain gauges. Then, at least 3 full load-unload calibration cycles were carried out to obtain an average calibration factors. The load/voltage relations from each cycle were found to be very repeatable for a given gauge set.

The temperature sensitivity of the strain gauges was assessed by taking baseline (zero applied load) readings at room temperature and at the colder air temperatures. For piles TP3 and TP4
which had full sets of 6 strain gauge installations each, an average temperature dependence of 2 mV/°C (the equivalent of 0.12 kN/°C) was measured. These two piles were used to assess the changes in axial load within the piles during reconsolidation, where the temperature of the gauges in the ground was only 3 degrees warmer than the cold baseline readings. Temperature changes during load testing were negligible.

The performance of the strain gauges during this study was generally found to be excellent, despite the fact that the gauges were functioning at the very low end of their output range. Only one of the 32 strain gauge installations during this study showed signal stability problems. This occurred 3 days after pile installation, but during the load test, signal stability was reaquired at load levels of around 50 kN or higher. The use of strain gauges on the pile shaft to measure the total load carried by the pile at points along the composite grout column was found to be problematic because of load sharing between the steel shaft and the grout column. These problems, and the data interpretation method used to compensate for them, are described in more detail in Appendix G.

5.2.5 Load Cell

A 220 kN capacity Sensotec load cell was used to measure the external load applied to the pile sections during strain gauge calibrations and to measure the load applied to the top of the piles during load tests. The signal was amplified 100x for monitoring with the data acquisition system, which provided a load resolution of 0.15 kN.
Figure 5.1: Test Pile Geometry
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Figure 5.2: Lead Sections of Test Piles

Figure 5.3: Example of Grout Disc

Figure 5.4: Reservoir of Liquid Grout to Supply Grout Column
20 cm diameter PVC pipe (Outer Casing)

15 cm diameter PVC pipe (Inner Casing)

Grout Column

Shaft of Upper Extension

Shaft of Middle Extension

annular cavity filled with Grout

PVC Pipe Reducer

Grease along contact

Grout Disc (18 cm diameter)

Figure 5.5: Conceptual Detail of Casing System
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Figure 5.6: Photograph of Casing

Figure 5.7: Photograph of Grout Disc at Bottom of Casing

Figure 5.8: Top of Pile After Installation
Figure 5.9: UBC Push-In Piezometer System

Assembled Push-In Piezo-Cell with Guide Pipe

Figure 5.10: Piezometer Components

Piezometer with Transducer Connected

Saturated Pressure Transducer Unit

Steel Pipe for Push-In Installation

Saturated Piezo-Cell

Assembled Push-In Piezo-Cell with Guide Pipe

Porous Filter

Cable Weight

Drop-In Pressure Transducer Unit

Swagelock Quick-Connector (female fitting)

Swagelock Quick-Connector (male fitting)
Figure 5.11: Piezo-Port on Pile Shaft

Figure 5.12: Strain Gauge Protection
6.0 PORE PRESSURE CHANGES DURING AND AFTER PILE INSTALLATION

In this chapter, the pore pressure response of the soil during and after pile installation is discussed. The pore pressure response during pile installation is significant because it is indicative of the type and extent of soil deformations caused by pile installation. The distribution of pore pressure around the piles at the end of installation also has an important influence on the subsequent rate of dissipation, which controls the reconsolidation and strength recovery of the soil around the piles. The radial distribution of pore pressures during and at the end of installation are presented in Section 6.2, and the observed dissipation process is discussed in Section 6.3.

First, the equipment and procedures used to monitor the pore pressures during and after pile installation, and to determine the radial distributions of pore pressure within the soil surrounding the piles, are described in Section 6.1 below.

6.1 PIEZOMETER AND PILE INSTALLATION AND MONITORING PROCEDURES

A total of 26 piezometers were located at different depths and radial distances from the 6 test piles in this study, and a total of 10 piezo-ports were located at 3 different positions on the shaft of the piles, as indicated in Table 6.1. The piezometers were pushed into the soil at least 1 week in advance of pile installation to allow for full dissipation of the excess pore pressures generated during piezometer installation.

After installing the piezometers and before connecting the drop-in transducer units, inclinometer soundings were taken inside the 38 mm diameter pushing pipe using 25 mm square tubing which provided a guide track for the wheels of the UBC miniature inclinometer probe (described by Campanella et al., 1994), as shown on Figure 6.1. In each piezometer, opposite corners of the square tubing were aligned along radial lines emanating from the center of the proposed pile position in order to provide a reference direction for the inclinometer sounding. The orientation of the radial lines between the proposed pile center and each piezometer, and the distances from the center of the proposed pile position to the top of the tubing along the radial lines were measured and used to generate a scaled plan of the top-of-piezometer locations.
## Table 6.1
Piezometer & Piezo-Port Locations

<table>
<thead>
<tr>
<th>Test Pile (TP)</th>
<th>Piezometer (PZ) or Piezo-Port (PP)</th>
<th>Filter Elevation (m)</th>
<th>Radial Distance ($r/R_{shaft}$) from Pile Center</th>
<th>Response During Installation and Dissipation</th>
</tr>
</thead>
<tbody>
<tr>
<td>TP1 (S/D=3, 6 week recovery)</td>
<td>PZ-TP1-1</td>
<td>-4.57</td>
<td>5.5</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-2</td>
<td>-6.02</td>
<td>8</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-3</td>
<td>-5.99</td>
<td>14</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-4</td>
<td>-5.96</td>
<td>25</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-5</td>
<td>-7.71</td>
<td>7.8-8.2</td>
<td>Fair</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-6</td>
<td>-8.86</td>
<td>5.6</td>
<td>Fair</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-7</td>
<td>-8.85</td>
<td>12-13</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-8</td>
<td>-8.82</td>
<td>11-12</td>
<td>Fair</td>
</tr>
<tr>
<td></td>
<td>PZ-TP1-9</td>
<td>-9.80</td>
<td>5.6</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>TP1-PP1</td>
<td>-7.91</td>
<td>1</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>TP1-PP2</td>
<td>-9.01</td>
<td>1</td>
<td>Fair (poor amp setup)</td>
</tr>
<tr>
<td></td>
<td>TP1-PP3</td>
<td>-9.92</td>
<td>1</td>
<td>None (poor amp setup)</td>
</tr>
<tr>
<td>TP2 (S/D=1.5, 6 week recovery)</td>
<td>PZ-TP2-1</td>
<td>-4.61</td>
<td>8.0</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>PZ-TP2-2</td>
<td>-5.93</td>
<td>6.6</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>PZ-TP2-3</td>
<td>-5.92</td>
<td>16.5</td>
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<td></td>
<td>PZ-TP2-4</td>
<td>-5.95</td>
<td>30</td>
<td>Good</td>
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<td></td>
<td>PZ-TP2-5</td>
<td>-7.67</td>
<td>7.3</td>
<td>Good</td>
</tr>
<tr>
<td></td>
<td>PZ-TP2-6</td>
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<td>-8.77</td>
<td>11.2</td>
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<tr>
<td></td>
<td>PZ-TP2-8</td>
<td>-8.80</td>
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<td>-9.74</td>
<td>15.8</td>
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<td></td>
<td>TP2-PP1</td>
<td>-7.68</td>
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<td>Good</td>
</tr>
<tr>
<td></td>
<td>TP2-PP2</td>
<td>-8.83</td>
<td>1</td>
<td>Delayed (1 day)</td>
</tr>
<tr>
<td></td>
<td>TP2-PP3</td>
<td>-9.75</td>
<td>1</td>
<td>Delayed (7 days)</td>
</tr>
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<td>TP3 (S/D=3, 7 days)</td>
<td>PZ-TP3-1</td>
<td>-6.03</td>
<td>5.8</td>
<td>Excellent</td>
</tr>
<tr>
<td></td>
<td>PZ-TP3-2</td>
<td>-8.84</td>
<td>8.1</td>
<td>Excellent</td>
</tr>
<tr>
<td></td>
<td>TP3-PP3</td>
<td>-9.68</td>
<td>1</td>
<td>Poor</td>
</tr>
<tr>
<td>TP4 (S/D=1.5, 7 days)</td>
<td>PZ-TP4-1</td>
<td>-6.07</td>
<td>4.8</td>
<td>Excellent</td>
</tr>
<tr>
<td></td>
<td>PZ-TP4-2</td>
<td>-8.88</td>
<td>6.3</td>
<td>Excellent</td>
</tr>
<tr>
<td></td>
<td>TP4-PP3</td>
<td>-9.70</td>
<td>1</td>
<td>Excellent</td>
</tr>
<tr>
<td>TP5 (S/D=3, 19 hours)</td>
<td>PZ-TP5-1</td>
<td>-6.00</td>
<td>6.7</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>PZ-TP5-2</td>
<td>-8.81</td>
<td>4.7</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>TP5-PP3</td>
<td>-9.7</td>
<td>1</td>
<td>None</td>
</tr>
<tr>
<td>TP6 (S/D=1.5, 19 hours)</td>
<td>PZ-TP6-1</td>
<td>-5.99</td>
<td>7.1</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>PZ-TP6-2</td>
<td>-8.80</td>
<td>7.5</td>
<td>Poor</td>
</tr>
<tr>
<td></td>
<td>TP6-PP3</td>
<td>-9.7</td>
<td>1</td>
<td>None</td>
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</tbody>
</table>
The bottom of the square tubing could be lowered to within 0.45 to 0.5 m of the piezometer filter, and therefore the inclinometer soundings could be carried out to within about 0.6 m of the piezometer filter elevation. Each sounding consisted of inclination measurements in orthogonal directions taken at 0.2 m intervals, which were then used to generate profiles of the horizontal deviation (along orthogonal axes) of the tubing inside the piezometer pipe using the commercial GTilt software. In each piezometer, a second sounding was taken after rotating the inclinometer probe 180° in order to eliminate any zero errors from the calibration process. The horizontal deviation (along orthogonal axes) of the piezometer filter relative to the top of the tubing was estimated by extrapolating the profiles over the short distance below the bottom of the soundings. This allowed the location of each of the piezometer filters to be determined in plan.

After pile installation, the position of the top of the pile relative to the top of the piezometers was measured. The square tubing for the inclinometer was inserted down the inside of the pile shaft and a profile of the horizontal deviation of the pile shaft with depth was obtained using the same method as for the piezometers. This allowed the position of the pile shaft at the depth of each level of piezometers to be determined on the same plan as the piezometer filter locations. The radial distance from the pile center to each piezometer filter was then measured from this scale plan.

The nest of piezometers around each pile was used to establish the background piezometric conditions immediately prior to pile installation. The piezometers were then monitored continuously using the multi-channel data acquisition system as pile installation proceeded. The piles were installed using a torque head that was hung from the boom of a backhoe (as shown on Figure 6.2). Torque measurements were not recorded during installation of the test piles, since it was determined during installation of the helical reaction piles at the test site that only a minimal drive torque of about 1750 – 1900 N·m (1300 – 1400 ft·lbs) was required. Installation was interrupted frequently to adjust the position of the boom and torque head to maintain the alignment of the pile as close as possible to vertical. During sustained installation, the average penetration rate of the helical anchor piles was about 10 cm in 7 to 8 sec (1.2 to 1.5 cm/s), which is similar to the standard CPT penetration rate of 2 cm/s.

After installing the lead section to a tip depth of about 2.8 to 3.0 m, there was a break in installation (lasting about 15 to 30 min.), while the first 1.4 m long extension section was added.
A second break at a tip depth of about 4.1 to 4.3 m, lasting between about 10 and 25 min., was required to add the second extension. A third break in installation at a tip depth of about 5.8 to 6.0 m, lasting about 45 to 55 min., was required to add the final extension and casing system. During the breaks in installation, monitoring of the piezometers was continued to capture the ongoing time-dependent changes in pore pressure. During each break, the piezo-ports on piles TP3 and TP4 (TP3-PP and TP4-PP) were also connected shortly after stopping penetration and monitored briefly in an attempt to determine the penetration pore pressure at the pile shaft at a variety of depths.

At the end of pile installation, the transducer wires for the piezo-ports on the pile shaft were connected to the data acquisition system within as little as 2 minutes (for TP3&4) and as much as 13 minutes (TP2) of stopping, in order to monitor the dissipation process. Relatively continuous monitoring of the piezo cells (piezometers and piezo-ports) was continued for the remainder of the day of pile installation, followed by discrete monitoring on a daily basis until complete dissipation of the excess pore pressures was observed or load testing began. During monitoring of the dissipation process, background pore pressure measurements were also made from piezometers located at a distance of at least 4.5 m from the installed test pile(s), and at two or more different elevations.

The response of each of the piezo cells during pile installation and/or during the dissipation period is also indicated in Table 6.1. An “excellent” response indicates that there was no detectable delay in the observed pore pressure response, suggesting that the saturation of the cells was essentially perfect. A “good” response indicates that there may or may not have been minor time lags due to imperfect saturation, but that the overall performance of the instrument was adequate for the purposes of this study. “Fair” to “poor” responses indicate that significant time lags were observed, due to either poor saturation or to delayed contact between the soil and the filter (in the case of the piezo-ports). In the “fair” cases the time lags made interpretation of the recorded data more difficult and the results less reliable, but some of the data was still found to be useful as part of a larger database of more reliable results. In the “poor” cases, however, the usefulness of the data was very limited.
6.2 **Pore Pressures Generated by Pile Installation**

The results of pore pressure monitoring during pile installation are presented in this section. The changes in pore pressures observed within the numerous piezometers surrounding the piles during the pile installation process are presented in Section 6.2.1. The installation pore pressures at the pile shaft are presented in Section 6.2.2 and are compared to those measured along the shaft of the CPT penetrometer. All of the installation pore pressure data were compiled to obtain radial distributions of excess pore pressures during and at the end of installation, which are presented in Section 6.2.3 and are compared to theoretical solutions of excess pore pressure distributions around piezocones and cylindrical cavities.

6.2.1 **Piezometer Measurements During Pile Installation**

6.2.1.1 Pore Pressure Changes with Pile Penetration Depth

The profiles of the excess pore pressures measured at each piezometer location during installation of test piles TP2-4 are included on Figures F-1a&b in Appendix F. Samples of the installation pore pressure profiles measured at different radial distances \(r/R_{\text{shaft}}\) where \(R_{\text{shaft}}\) is the radius of the pile shaft) from the final location of the pile center are presented on Figure 6.3a (\(S/D = 1.5\)) and Figure 6.3b (\(S/D = 3\)). The excess pore pressure during installation, \(\Delta u_i\), has been normalized by the vertical effective stress prior to pile installation, \(\sigma_{vo}'\), as this was found to be a convenient and effective means of normalizing the pore pressure response from different depths at the test site. The profiles of \(\Delta u_i/\sigma_{vo}'\) are plotted against the depth of the pile tip below the elevation of the piezometer filter \((z_{\text{pile}} - z_{\text{piezo}})\), so that the response of piezometers from different depths can be compared directly. For reference, the locations of the different parts of the pile relative to the tip are also shown on the right side of these figures. This way, the changes in pore pressure caused by the penetration of each helix or grout disc can be inferred directly. It should be noted that the portions of the profiles on Figures 6.3a and 6.3b which were recorded after extended breaks in pile penetration (when extension sections were being added) have been shifted to eliminate the decreases in pore pressure which occurred during these breaks. The profiles included in Appendix F have not been adjusted.

From Figures 6.3a and 6.3b it can be seen that there is a very sudden increase in \(\Delta u_i\) as the tip of the pile shaft approaches and then passes the elevation of the piezometer filters. This increase is
particularly abrupt at the piezometers located closer to the pile. It is also readily apparent that the magnitude of excess pore pressure generated within the soil by the pile installation decreases with radial distance from the pile.

Immediately prior to the passing of the pile tip and the sudden increase in $\Delta u_i$, a decrease in pore pressure is observed at most of the piezometers, regardless of depth or distance from the pile. Initial negative excess pore pressures were also measured around static piezo-cells located at depths below the tip of penetrating piles by Roy et al. (1981) and Koizumi & Ito (1967), and below a penetrating cone probe by Baligh & Levadoux (1980). Baligh & Levadoux (1980) suggest that this behaviour is a result of the vertical displacement of soil in advance of a penetrating pile or probe, which is initially downward. Such downward movement of soil relative to the static piezo-cell induces a tensile pressure response. Once the pile tip reaches the piezometer element, the soil displacement vector becomes primarily radial and the pressure along the piezometer filter becomes strongly compressive.

A very sharp peak in pore pressure ($\Delta u_{\text{max}}/\sigma'_{\text{vo}} = 1.5$) is observed at the innermost piezometers between $r/R_{\text{shaft}} = 5$ and 6 (within about 10 cm of the helix edge, or about 10 to 12 times the helix plate thickness) as the bottom helix plate passes the piezometer filters. At PZ-TP3-1, located within 6 shaft radii of an S/D=3 pile, individual pore pressure “pulses” from each of the 3 helix plates can be seen on Figure 6.3b, although the second peak is somewhat distorted, and the third peak includes the passing of the bottom grout disc. For the S/D=1.5 pile, the response to individual plates (shown on Figure 6.3a) is less definitive after the passing of the first plate, although a second increase is observed during the passing of the 3rd, 4th and 5th (top) helices, resulting in a second, lower peak. The poorer definition in the response of the soil beyond the edge of the helices to the penetration of successive helix plates may be due to reductions in strength and stiffness which probably occur within the soil due to strain softening resulting from the penetration of the first plate.

At the piezometers located between $r/R_{\text{shaft}} = 7$ and 8 (within 1 helix radius of the helix edge), the peak in $\Delta u_i$ is much more subtle and is somewhat delayed, occurring after the passing of the bottom helix plate. After this peak, these piezometers exhibit a trend of decreasing $\Delta u_i/\sigma'_{\text{vo}}$ with continuing penetration, with only slight variations during passing of subsequent helices. At the piezometers beyond $r/R_{\text{shaft}} = 8$, the pore pressure is observed to build up gradually with
continuing penetration of the pile, reaching a maximum magnitude and then remaining roughly constant as successive plates pass the level of the piezometers.

From Figures 6.3a and 6.3b, the following general observations can be made about the pore pressure response of the soil to the passing of the helices:

- Only the soil located very close to the outside edge of the helix plates (within about 2 to 2.5 shaft radii, or 10 to 12 times the helix plate thickness, \( t_{hx} \)) appears to respond directly to the penetration of the helix plates. The extent of this zone may be controlled by the stiffness of the partially destructured soil. Within this zone, distinctively different responses are observed for the S/D = 1.5 and S/D = 3 piles.

- The rate of increase in \( \frac{\Delta u}{\sigma'_{vo}} \) following the penetration of the tip of the pile shaft decreases with increasing distance from the pile.

- \( \frac{\Delta u_{max}}{\sigma'_{vo}} \) decreases and the delay to \( \Delta u_{max} \) increases with increasing radial distance from the pile, as indicated by the dashed lines on Figures 6.3a and 6.3b.

- At radial distances larger than about 10-12 \( t_{hx} \) beyond the edge of the helices, the pore pressure response to the penetration of the S/D = 1.5 and S/D = 3 piles is very similar.

Additional pulses of \( \Delta u_i \) are observed at the near-field piezometers located at \( r/R_{shaft} \leq 8 \) during passing of the grout discs, indicating that the grout discs cause additional deformations out to a distance of at least 5 disc radii. No indications of disturbance effects due to the penetration of the grout discs are observed at \( r/R_{shaft} \geq 17 \) (10 disc radii). A significant reduction in \( \Delta u_i \) is observed at the near-field piezometers after the passing of the grout discs, which drops \( \Delta u_i \) below that which was initially generated by the initial penetration of the pile shaft. This suggests that there is a net reduction in total stress within the soil surrounding the annular cavity formed by the grout disc. It is possible that such a reduction is a result of displacement of the disturbed soil back toward the liquid grout-filled cavity.

6.2.1.2 Pore Pressure Changes with Time During Pile Installation

On Figures 6.4a and 6.4b, \( \Delta u/\sigma'_{vo} \) is plotted against the square root of the elapsed time after passing of the pile tip, for piezometers located at different radial distances from the centerline of the piles with S/D = 1.5 and 3, respectively. At time zero, the excess pore pressure is close to
that generated by the penetration of the base of the pile shaft ($\Delta u_{\text{shaft}}$), which is approximately 15 cm long below the bottom helix. The long, relatively flat, linear segments which appear at the end of the curves are characteristic of pore pressure dissipation curves, where the slow, monotonic decrease in $\Delta u$ is a result of an outward flow of pore water. The dashed lines shown on Figure 6.4 correspond to the extrapolation of the dissipation segments of the $\Delta u-\sqrt{t}$ curves (which may or may not appear within the time range shown). Figure 6.4 is useful to differentiate the decreases in $\Delta u$ due to dissipation from changes in $\Delta u$ that result from changes in total stresses which continue after stopping pile installation.

The sharp pulse of additional pore pressure caused by the penetration of the bottom helix plate ($\Delta u_{\text{hx}}$), which was observed at the piezometers between $r/R_{\text{shaft}} = 5$ and 6 on Figure 6.3a and 6.3b, can also be seen above the extrapolated dissipation lines for PZ-TP4-1 and PZ-TP3-1 on Figures 6.4a and 6.4b, respectively. At greater radial distances from the edge of the helix plate, the arrival time of the pore pressure pulse is increasingly delayed, while the duration of the elevated pore pressures increases. For the piezometers located at $r/R_{\text{shaft}} \leq 8$, the pulse plots above the extrapolated dissipation lines, which give relatively good predictions of $\Delta u_{\text{shaft}}$ when extrapolated back to zero time. For $r/R_{\text{shaft}} \geq 11$, the pulse becomes less and less distinctive as the pore pressure level becomes more uniformly elevated, as indicated by the shift of the dissipation line to pore pressure levels that plot above the initial shaft penetration pressure (as shown for PZ-TP2-7&3 on Fig. 6.4a).

From these observations it is inferred that:

- $\Delta u_{\text{hx}}$, which appears to be initially limited to a radial distance within about 10-12 $t_{\text{hx}}$ beyond the edge of the helices, may also be initially limited in vertical extent. This would explain the very short duration of the elevated pore pressures measured at the piezometers between $r/R_{\text{shaft}} = 5$ and 6 after passing of the bottom helix plate.

- The outward migration of the pulse of $\Delta u_{\text{hx}}$ may be accompanied by a vertical spreading which causes an attenuation of the pulse but also increases the duration of the elevated pore pressures at larger radial distances.

- The sustained duration of $\Delta u_{\text{hx}}$ at larger radial distances, suggests that the elevated pore pressures within the outer zone are pervasive throughout a volume of soil that is extensive in
the vertical direction as well as in the radial direction. The significant increases in residual $\Delta u$ measured within this zone may be due to an accumulation of attenuated $\Delta u_{hx}$ pulses from a single helix plate over a range of depths, as well as from multiple helices.

The apparent outward migration of the $\Delta u_{hx}$ pulse may be coupled to a redistribution of stresses caused by time-dependent undrained deformations (both horizontal and vertical) of the soft soil in response to localized strain fields induced by the helices. The additional strains induced by the penetration of a helix plate (“primary deformations”) may be causing strain softening within the soil just beyond the edge of the helices, which was already at failure after penetration of the shaft. This could cause a redistribution of total stresses as the deviator stresses are reduced due to strain softening. “Secondary deformations” caused by the transferred stresses may also cause strain softening within the elements adjacent to the region disturbed initially by the helix plate, thereby forcing the transfer of stress and strain to continue. Under this mechanism, it would be reasonable to expect that some of the stress shed from the zone of primary deformation would propagate in the radial direction all the way out to the soil beyond the plastic zone, where additional shear stress could be accommodated. Thus, the apparent outward migration of the $\Delta u_{hx}$ pulse could be the manifestation of such an outward propagation of stress and strain.

During initial penetration of the pile shaft, the soil that is displaced by the shaft undergoes very rapid deformation and, consequently, $\Delta u_i$ is generated very rapidly, as was observed on Figures 6.3a and 6.3b. The loading and unloading of the soil close to the edge of the helices during penetration of the plates is also rapid and so the corresponding changes in $\Delta u$ are also rapid, as was observed on Figure 6.4. In contrast, the dissipation of $\Delta u$, which occurs as pore water flows away from the areas of high pore pressure, is very slow. However, changes in pore pressure are observed, both during pile installation and immediately after stopping pile installation, which occur at rates between these extremes. For example:

- gradual increases in $\Delta u_i$ at outer piezometers ($r/R_{shaft} \geq 11$) during pile installation (eg. PZ-TP2-3,7 on Figure 6.4a, PZ-TP1-3 on Figure 6.4b),
- gradual decreases in $\Delta u_i$ observed at piezometers between $r/R_{shaft} = 7$ and 8 during continuing pile installation after $\Delta u_{max}$ (eg. PZ-TP2-5 on Figure 6.4a, PZ-TP3-2 on Figure 6.4b),
- gradual decreases in $\Delta u(t)$ immediately after stopping installation until the dissipation line is reached (observed at most piezometers).
If these changes in pore pressure are due to secondary deformations resulting from the redistribution of stress, it is likely that the rate of change in pore pressure would be linked to the limiting strain rate controlled by the viscous properties of the soil. It is expected that this strain rate would be considerably lower than the strain rates imposed on the soil during primary deformations caused by the penetration of the pile shaft or the penetration of the helix plates. Hence the slower pore pressure responses noted above.

### 6.2.2 Installation Pore Pressure at Pile Shaft

#### 6.2.2.1 From Pile Data

The penetration pressures, $\Delta u$, at various piezo-port locations on the pile shaft were estimated by extrapolating back to time zero using root time dissipation curves from measurements obtained within a few minutes of stopping penetration (these curves are included on Figure F-2 in Appendix F). As will be seen in Section 6.3, the observed dissipation at the pile shaft was monotonic from as early as 2 minutes after stopping penetration. This corresponds to a time factor, $T = 0.12$, which is earlier than the time factor at which the peak pore pressures were measured at either the U2 or U3 positions during CPTU dissipation testing. Thus, there does not appear to be any significant vertical pore pressure redistribution occurring along the pile shaft above or below the bottom helix.

The extrapolated estimates of $\Delta u/\sigma'_{vo}$ from piezo-port locations 7.5 cm below the bottom helix (TP3,4-PP3), 22.5 cm below the middle helix (TP1,2-PP2) and 15 cm below the upper helix (TP1,2-PP1) are plotted on Figure 6.5. The range of values which could be obtained using different trend lines to extrapolate to zero time are indicated as error bars on Figure 6.5. Where there was insufficient dissipation data to consider a range of possible extrapolations, the coefficient of correlation ($r^2$) of the trend line used to obtain the extrapolated value is indicated instead.

The extrapolated value of $\Delta u/\sigma'_{vo}$ from TP4-PP3, located 7.5 cm ($8\ t_{hx}$) below the bottom helix, was found to be about 1.4 at 3 different elevations below the crust (-5.6 m, -7.1 m and –9.8 m). At –4.1 m elevation (at the top of the crust), however, $\Delta u/\sigma'_{vo} = 1.8$. If the width of the zone of soil loaded directly by the penetration of the helix plates is about the same below the plates as in
the radial direction (i.e. 10-12 t₁₁x₁₁), it is likely that Δuᵢ measured 7.5 cm below the bottom helix plate will be influenced to some degree by the soil deformation around the plate.

Between the helices, Δuᵢ/σ’ᵥₒ at the pile shaft ranges from 1.4 to 1.6 for both the PP1 location below the top helix and for the PP2 location below the middle helix. The similar responses at these two different locations suggests that the number of helix plates and the number of load-unload cycles due to the penetration of successive plates does not have a major impact on the magnitude of the excess pore pressure at the pile shaft. This may be due to the fact that the additional disturbance caused by the penetration of the upper helices is small compared to the combined disturbance due to the penetration of the pile shaft and the bottom helix plate. However, the proportion of Δuᵢ at the shaft that is generated by the deformations caused by the helix plates (i.e. the magnitude of Δuₜₙxₙ) could not be measured directly.

6.2.2.2 Comparison to CPTU Measurements

The installation pore pressures estimated from the pile piezo-port data are compared to representative CPTU penetration pore pressure profiles from the Colebrook site on Figure 6.6. It should be noted that the steady penetration rate for the helical anchor piles (1.2 to 1.5 cm/s) is about the same order of magnitude as the CPTU penetration rate of 2 cm/s. Therefore, differences in generated pore pressures between the test piles and the CPTU probe due to rate effects should be negligible.

It can be seen that the installation pore pressures at the pile shaft generally fall between the CPTU penetration pressures measured at the U2 and U3 filter locations. Above –6 m elevation, Δuᵢ/σ’ᵥₒ at the pile shaft (determined from the TP4-PP3 piezo-port, located just above the tip of the pile shaft, during breaks in installation) is closer to the U2 measurements. Between –7 and –8 m elevation, Δuᵢ/σ’ᵥₒ at the pile shaft appears to be roughly equivalent to the mean of the U2 and U3 measurements. Below –8 m elevation, Δuᵢ/σ’ᵥₒ at the pile shaft is closer to the U3 measurements. These inconsistencies in the magnitudes of the pile shaft pore pressures relative to the CPTU pore pressures indicates that the response of the near-field soil to the two different penetration mechanisms appears to be somewhat different. This is probably due to the different strain paths followed by the soil elements located close to the shafts of the cone penetrometer and the pile. It is expected that the additional deformations caused by the penetration of the helix
plates as well as the slightly different penetration mechanism for the shaft of the helical piles could both cause the pore pressure response near the piles to differ from that around the cone penetrometer.

6.2.3 Radial Distribution of Pore Pressure

6.2.3.1 From Pile Data

The initial excess pore pressures generated by the penetration of the bottom of the pile shaft (below the bottom helix), $\Delta u(r)_{\text{shaft}}/\sigma'_{vo}$, were interpreted from the piezometer measurements around all 6 test piles and are plotted with radial distance ($r/R_{\text{shaft}}$) from the center of the pile on Figure 6.7. From $r/R_{\text{shaft}} = 5$ to 17, $\Delta u_{\text{shaft}}/\sigma'_{vo}$ decreases steeply with radial distance and is approximately linear with $\log(r/R_{\text{shaft}})$. Beyond $r/R_{\text{shaft}} = 17$, $\Delta u_{\text{shaft}}$ is small ($< 0.1\sigma'_{vo}$) and the slope of the pore pressure decay with distance is much flatter. At $r/R_{\text{shaft}} = 60$ and beyond, negligible pore pressures were generated by shaft penetration.

Linear and logarithmic trend lines have been fitted to the piezometer data from the middle annulus of soil between $r/R_{\text{shaft}} = 5$ and 17, along with a logarithmic trend line fitted to the data from the outer region of soil beyond $r/R_{\text{shaft}} = 17$, and are shown on Figure 6.7. It is useful to note that the logarithmic trend line through the middle zone indicates that the pore pressure would drop to zero at $r/R_{\text{shaft}} = 19$. This distance corresponds to the edge of the plastic zone predicted for cylindrical cavity expansion in an elastic-plastic soil with rigidity index ($I_r = G/s_u$) of 350, which is consistent with the estimates of $I_r$ from Section 4.3.

The end of installation pore pressures at the pile shaft, $\Delta u(r=R_{\text{shaft}})/\sigma'_{vo}$, which were determined from measurements at the piezo-port locations below the top, middle and bottom helix plates, are also plotted at $r/R_{\text{shaft}} = 1$ on Figure 6.7. These measured pore pressures are predominantly due to the excess pore pressure generated by the penetration of the shaft alone, $\Delta u_{\text{shaft}}$, but may also include some additional pore pressure due to the penetration of the helix plate, $\Delta u_{\text{hx}}$. The range (mean ±1 standard deviation) of the CPTU penetration pore pressures measured at the U3 location over the same depth range as the pile piezo-ports are also plotted at $r/R_{\text{shaft}} = 1$ on Figure 6.7. It seems reasonable to expect that the U3 $\Delta u_{i}/\sigma'_{vo}$ may give a reasonable estimate of the excess pore pressure at the pile shaft due to the penetration of the shaft alone, $\Delta u(r=R_{\text{shaft}})_{\text{shafts}}$. The shaded region on Figure 6.7 is proposed as a possible range of $\Delta u(r)_{\text{shaft}}/\sigma'_{vo}$.
distributions through the inner zone of soil between the helices, with the mean and mean +1 standard deviation of the U3-measured $\Delta u/\sigma'_{vo}$ assumed as the lower and upper bounds, respectively, for $\Delta u(r=R_{shaft})/\sigma'_{vo}$.

To inspect the effects of the penetration of the helices on the radial distribution of excess pore pressure, the radial distribution of peak pore pressures, $\Delta u_{max}/\sigma'_{vo}$, measured at any time during installation are plotted on Figure 6.8. It should be noted that some of the recorded peaks plotted on this figure may be slightly less than the actual peak pore pressure in the soil due to slight time lags in imperfectly saturated piezometers. Also, the pore pressures measured at the piezometers located at elevations below the bottom helix were not included on this figure. The light dashed lines shown on Figure 6.8 represent the inferred envelope of maximum pore pressure, $\Delta u(r)_{max}/\sigma'_{vo}$, due to the passing of the helices. The trend lines representing $\Delta u(r)_{shaft}/\sigma'_{vo}$, which were fitted to the piezometer data on Figure 6.7 (for $r/R_{shaft} \geq 5$), are also shown on Figure 6.8, along with a best guess of the distribution between $r/R_{shaft} = 1$ and 5.

The vertical difference between $\Delta u(r)_{shaft}/\sigma'_{vo}$ and $\Delta u(r)_{max}/\sigma'_{vo}$ represents the best estimate of the additional pore pressure generated by the penetration of the helix plates, $\Delta u_{hx}/\sigma'_{vo}$. Out to a radial distance of about $r/R_{shaft} = 6$, an immediate increase in $\Delta u/\sigma'_{vo}$ of between 0.35 and 0.5 is recorded when the bottom helix passes the piezometers. Beyond this distance, $\Delta u_{hx}$ drops off suddenly, and at greater distances $\Delta u(r)_{max}/\sigma'_{vo}$ is very close to $\Delta u(r)/\sigma'_{vo}$ measured 0.1 min after stopping installation. At the end of installation, $\Delta u(r)_{hx}/\sigma'_{vo}$ increases with increasing radial distance from about 0.2 between $r/R_{shaft} = 7$ and 8 to a maximum of about 0.3 to 0.35 near the edge of the initial plastic zone ($r/R_{shaft}$ from about 16 to 19).

6.2.3.2 Comparison of Shaft Penetration Pore Pressures with Theoretical Solutions

In an attempt to determine a means of predicting the radial distribution of excess pore pressure generated by the helical piles, the pile data presented above was compared to pore pressure distributions predicted by the following popular methods:

- Cylindrical Cavity Expansion (CCE) theory with an elastic-plastic soil model, and
It is recognized that the potential applicability of either of the above methods would be limited to the pore pressures generated by the penetration of the cylindrical pile shaft, and the additional pore pressures generated by the helix plates will have to be treated separately.

**From Strain Path Method**

Radial distributions of $\Delta u(r)/\sigma'_{vo}$ predicted for the penetration of a cone penetrometer (with 60° cone angle) by Levadoux & Baligh (1980, 1986) using the Strain Path Method and the properties of resedimented, normally consolidated (NC) Boston blue clay (BBC), are compared to $\Delta u(r)_{shaft}/\sigma'_{vo}$ from the pile data on Figure 6.9. The penetration pressures in the vicinity of the cone are much higher than the stresses around the shaft at greater distances behind the cone. As a result, $\Delta u(r)$ around the shaft of the cone probe is sensitive to the vertical location relative to the cone, so predicted distributions corresponding to locations at both the U2 and U3 locations (2 and 10 radii above the cone tip, respectively) are both shown on Figure 6.9.

The magnitude of $\Delta u(r)$ at locations further behind the cone should be more representative of that caused by just the shaft of the helical piles (if there were no helix plates). The radial distribution of $\Delta u(r)/\sigma'_{vo}$ predicted by Levadoux & Baligh for the U3 location (10 radii above the tip) is in general agreement with $\Delta u(r)_{shaft}/\sigma'_{vo}$ from the pile piezometers for $r/R_{shaft} \geq 5$ to 6, and with the inferred distribution between $r/R_{shaft} = 1$ and 5. The Levadoux & Baligh distribution for the U2 location is in general agreement with the data from the pile piezometers for $r/R_{shaft} \geq 5$, but greatly over-predicts the estimated $\Delta u(r)_{shaft}/\sigma'_{vo}$ for $r/R_{shaft} < 5$.

It appears that the shape of the $\Delta u(r)/\sigma'_{vo}$ distributions predicted by Levadoux & Baligh are quite realistic. However, the pore pressure response of the highly sensitive, lightly overconsolidated Colebrook silts is not likely to be identical to that of resedimented, normally consolidated Boston Blue clay. Therefore, the agreement between the predicted pore pressure magnitudes and those measured at the Colebrook test site (both the pile and CPTU data) may be fortuitous.

**From Cavity Expansion Theory**

Distributions of $\Delta u(r)/\sigma'_{vo}$ were calculated using the solution for $\Delta u(r)$ that is based on cylindrical cavity expansion (CCE) theory and a simple elastic-plastic constitutive model, which was derived in Appendix A.1 (Equation A18a), and is shown below:
\[ \Delta u(r)/\sigma'_{vo} = 2s_u/\sigma'_{vo} \ln[I_r^{1/2}/(r/R_{shaft})] \]  

(6.1)

Representative values of \( s_u/\sigma'_{vo} \) for \(-6.0\) and \(-8.8\) m elevations, and an inferred average \( I_r \) of 350, were substituted into Equation 6.1 to obtain the distributions shown by the chain-dashed lines on Figure 6.10. It should be recognized that the above pore pressure solution assumes that the mean effective stress remains constant, and therefore assumes that there will be no shear-induced pore pressures.

From Figure 6.10, it can be seen that the two lines appear to agree remarkably well with the distribution of pore pressures generated by the penetration of the pile shaft, which were measured at the piezometers between \( r/R_{shaft} = 5 \) and 17. \( \Delta u(r) \) predicted using this solution is based on an assumed zone of perfectly plastic soil. Thus, the close agreement between the predicted and measured pore pressure distributions suggests that the actual shear stress within this region was at or near the peak strength of the material immediately after penetration of the pile shaft.

For a linear elastic soil, CCE theory predicts that no excess pore pressures would be generated outside the plastic zone. However, the pile test results show that small positive excess pore pressures were generated within the “elastic” region beyond \( r/R_{shaft} = 17 \) during initial shaft penetration. This would be consistent with a non-linear pre-failure stress-strain response where the proportion of plastic strains increases as the stress level increases.

From Figure 6.10, it is evident that the pore pressures predicted for \( r/R_{shaft} = 1 \) by CCE theory with an elastic-plastic model are much higher than those measured on the shaft of the piles or measured along the shaft of the cone probe during steady penetration. This is probably due to the differences between the actual strain paths and stress paths followed by the soil compared to that predicted by CCE theory, and to the differences between the actual stress-strain and pore pressure generation behaviour of the soil compared to that assumed using an elastic-plastic model. Consequently, the actual \( \Delta u(r) \) distribution is not logarithmic throughout the plastic zone, as is predicted by CCE theory. Therefore, while it appears that an elastic-plastic CCE solution could be used to estimate the \( \Delta u(r) \) distribution for \( r/R_{shaft} \geq 5 \), the distribution closer to the pile shaft would have to be corrected, possibly using measured CPTU penetration pore pressures from filters located above the friction sleeve.
6.2.3.3 Estimating Excess Pore Pressures Generated by Helix Plates

The additional pore pressure generated by the penetration of the helix plates ($\Delta u_{hx}$), above that which is generated by the penetration of the shaft alone, was clearly shown on Figure 6.8 for $r/R_{shaft} \geq 5$. Based on the $\Delta u_i$ profiles with pile penetration depth shown on Figures 6.3a & 6.3b, the maximum pore pressures close to the helices are generated by the penetration of the bottom helix plate.

During pile installation, the soil resistance to the downward penetration of the pile shaft produces an upward thrust on the pile which must be compensated by downward resistance mobilized along the upper surface of the helix plates (since no external downward force is applied to the piles during installation). The pressure exerted on the soil above the helix plates will induce additional strains and pore pressures. If we assume that all of the downward thrust required to cause penetration of the tip of the pile shaft is developed entirely by the bottom helix plate\(^2\), the induced pressure above the bottom helix, $q_{hx}$, can be calculated as:

$$q_{hx} = q_{tip} \cdot \frac{A_{tip}}{A_{hx}} \quad (6.2)$$

where $A_{hx}$ is the bearing area of the bottom helix plate, $A_{tip}$ is the cross-sectional area of the pile shaft, and $q_{tip}$ is the upward penetration pressure acting on the tip of the pile shaft. Since the pile and CPT penetration rates are similar, $q_{tip}$ can be estimated using $q_T$ measured with the cone probe.

The excess pore pressure caused by $q_{hx}$ can be estimated using the $B_q$ ratio determined from the CPT (with pore pressure measured on the face of the cone) or with some other appropriate $\Delta u/\Delta \sigma_v$ ratio. Accordingly, a profile of $\Delta u_{hx}/\sigma'_vo$ was generated using the $B_q$ (from U1) and $q_T$ profiles from CPT-1 and the following equation:

$$\Delta u_{hx} = q_T \cdot (A_{tip}/A_{hx})_{pile} \cdot B_{q(U1)} \quad (6.3)$$

This profile is presented on Figure 6.11, from which it can be seen that $\Delta u_{hx}/\sigma'_vo$ generally ranges between 0.3 and 0.4 between $-6$ and $-10$ m elevation. This is consistent with the

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\(^2\) i.e. the additional penetration resistance due to the grout discs and friction along the pile shaft is assumed to be compensated by the dead weight of the pile and the additional downward thrust provided by the upper helices.
magnitude of $\Delta u_{hx}/\sigma'_{vo}$ of about 0.3 near the edge of the initial plastic zone, which was interpreted from Figure 6.8. It also appears to be consistent with the estimated $\Delta u_{hx}/\sigma'_{vo}$ within the inter-helix region close to the pile shaft.

6.2.4 Summary and Conclusions from Pore Pressures Observed During Pile Installation

The distribution of excess pore pressure around the piles at the end of installation has a very important influence on the subsequent rate of reconsolidation within the soil through which the pile will be mobilizing its capacity, as was discussed in Section 2.4. The following key observations from the above discussions are important for understanding and predicting the distribution of excess pore pressure generated during pile installation:

- The excess pore pressure generated during installation of the helical piles is primarily due to plastic deformations caused by the penetration of the pile shaft. The excess pore pressures generated by the penetration of the shaft appear to be significant ($\Delta u > 0.1\sigma'_{vo}$) out to a radial distance of at least 17 shaft radii from the center of the pile.

- It appears that the radial distribution of $\Delta u$ due to the penetration of the shaft alone ($\Delta u(r)_{shaft}$) can be estimated using either of the following methods, provided that representative stress-strain and pore pressure generation models are used:

  1) Strain path method solutions for $\Delta u(r)$ around the shaft of the cone penetrometer (for locations above the friction sleeve).

  2) Cylindrical cavity expansion solutions to predict $\Delta u(r)_{shaft}$ for radial distances beyond about 5 shaft radii from the pile center, along with CPTU penetration pore pressure measurements at the U3 location to estimate $\Delta u_{shaft}$ at the pile shaft.

- Additional pore pressure are generated by the penetration of the helix plates ($\Delta u_{hx}$). These pore pressures are believed to be due to the upward pressure exerted by the helix plates on the soil above the plates during penetration. It appears that Equation 6.3 could be used to estimate $\Delta u_{hx}$ within the inter-helix region (any appropriate $\Delta u/\Delta \sigma_v$ ratio can be used instead of $B_{q(U1)}$ when CPTU pore pressure measurements on the cone face are not available).
• Beyond the edge of the helices, only the soil located very close to the helix plates (within a distance of about 10 – 12 times the thickness of the plate) responds directly to the penetration of the plates. The duration of $\Delta u_{hx}$ within this zone appears to be relatively brief.

• There appears to be a gradual outward propagation of $\Delta u_{hx}$ during continuing pile penetration, which is attributed to total stress redistribution effects. The cumulative effects of this outward propagation appear to be greatest near the edge of the initial plastic zone formed by the penetration of the pile shaft. In this area, the residual $\Delta u_{hx}$ appears to be similar to the $\Delta u_{hx}$ estimated within the inter-helix region near the pile shaft, and to that predicted using Equation 6.3. At the end of pile installation, elevated pore pressures of $\Delta u \geq 0.1\sigma'_v$ are observed out to radial distances of 60 shaft radii.

The excess pore pressures generated during pile installation, as well as the initial changes in $\Delta u$ immediately after stopping penetration, are a product of the strains induced within the soil by pile installation. Therefore, the distribution of pore pressure during pile installation is representative of the distribution of stresses and strains, according to the stress-strain behaviour of the soil. For a structured, sensitive soil that is prone to strain-softening, the level of strain can have a severe impact on the available shear strength of the soil, both at the end of installation as well as after re-consolidation. Based on the observed pore pressure responses discussed above, the following disturbance effects are inferred:

1) Based on $\Delta u(r)_{shaft}$, significant plastic deformations are inferred out to a radial distance of at least 17 shaft radii from the center of the pile after the initial penetration of the pile shaft. This suggests that the shear stress is at or near the undrained shear strength of the soil throughout this region immediately after penetration of the shaft. It appears that the radial extent of the plastic zone, $R_p$, can be reasonably predicted using the elastic-plastic formulation from cylindrical cavity expansion theory: $R_p = \sqrt{I_r}$ where $I_r = G/s_u$.

2) Based on the radial distribution of shear strains predicted by cavity expansion theory and the strain path method, it is reasonable to expect that the penetration of the pile shaft would cause some degree of strain softening within the soil close to the pile shaft. As was illustrated conceptually on Figure 2.3, a significant reduction in shear strength may occur
between the wall of the pile shaft and about 3 shaft radii from the center of the pile. Next to the pile shaft, the strength of the soil may be reduced close to the remoulded strength.

3) The penetration of the bottom helix plate will certainly cause additional large strains, as well as strain reversals, within the soil between the wall of the shaft and the edge of the helices. Each helix plate follows a continuous spiral path that is controlled by the pitch of the helix, which was 7.5 to 8 cm for the piles in this study, so that the vertical spacing between successive plate positions during pile installation should be equal to the pitch of the plate. Given this close spacing, it is reasonable to expect that the additional strains caused by the penetration of the first helix plate will be sufficient to significantly reduce the shear strength of most of the soil within the inter-helix zone. Therefore, additional softening due to the penetration of additional plates is unlikely to be as significant.

4) Based on the observed pore pressure response of the piezometers surrounding the piles, it appears that the additional large strains induced by the penetration of the bottom helix plate extend out beyond the edge of the helices for a radial distance of 10 to 12 times the helix plate thickness (9 - 11 cm). Since the soil within this zone was already at failure after penetration of the pile shaft, it is reasonable to expect that the additional large strains induced by the first helix plate will cause some degree of strain softening within this zone. The more subtle pore pressure responses exhibited by the piezometers within this zone during the penetration of additional helix plates, compared to the response during penetration of the first plate, suggest that softening has occurred.

5) The outward migration of the pulse of additional pore pressure induced by the penetration of the helix plate may be caused by a redistribution of stresses propagated by progressive strain softening through the plastic zone initially established by the penetration of the pile shaft. Consequently, the reconsolidated shear strength of the soil beyond the edge of the helices, and extending out toward the edge of the initial plastic zone, may be reduced to some degree below the peak strength of the material.

6) The distribution of residual pore pressures after penetration of different numbers of helix plates at either spacing (S/D = 1.5 or 3) appear to be very similar. This suggests that additional disturbance effects due to the penetration of additional plates are minimal within the soil beyond the edge of the helices.
6.3 PORE PRESSURE DISSIPATION AROUND PILES

The changes in the radial distribution of $\Delta u$ around the helical piles during the dissipation process are presented in Section 6.3.1. The curves of $\Delta u(t)$ vs. log(time) for locations within the soil at different radial distances from the piles are presented in Section 6.3.2. In Section 6.3.3, the dissipation curves for locations along the pile shaft are compared to dissipation curves measured during CPTU dissipation tests and to theoretical solutions for CPTU dissipation.

6.3.1 Changes in Radial Distribution

The radial distributions of $\Delta u/\sigma'_{vo}$ at various times after stopping installation are plotted on Figure 6.12. The data points on Figure 6.12 correspond to locations above the level of the bottom helix (for piles with both S/D = 1.5 and 3) and so have been affected by the penetration of at least one of the helix plates. Continuous curves have been fitted to the data and the degree of consolidation (U) at the pile shaft has also been indicated for each curve. It is clear that, even at early times in the consolidation process, $\Delta u$ generally decreases monotonically throughout the soil around the pile, out to a radial distance of at least 30 shaft radii. The rate of dissipation at different radial distances appears to vary such that the $\Delta u(r)/\sigma'_{vo} - \log(r)$ curve becomes more and more linear as the dissipation process progresses.

The trend lines representing $\Delta u(r)_{shaft}/\sigma'_{vo}$ due to the initial penetration of the pile shaft (from Figure 6.7) are also shown on Figure 6.12 for comparison with the distributions after stopping pile installation. After 10 minutes, no residual $\Delta u_{hx}$ is observed between $r/R_{shaft} = 5$ and about 8, such that $\Delta u(r)/\sigma'_{vo}$ is essentially equivalent to $\Delta u(r)_{shaft}/\sigma'_{vo}$. However, at greater distances ($r/R_{shaft} > 10$) significant residual $\Delta u_{hx}$ remains after 10 minutes, and elevated pore pressures (above $\Delta u(r)_{shaft}/\sigma'_{vo}$) appear to extend out beyond $r/R_{shaft} = 70$ to almost 100 shaft radii. The residual effects from the installation of piles with either S/D = 1.5 or 3 appear to be identical. It can also be seen on Figure 6.12 that the elevated pore pressures at the tail of the distribution ($r/R_{shaft} > 17$), which are due to the penetration of the helix plates, remain above the initial level generated by the pile shaft until about $U_{shaft} = 50\%$ to $60\%$.

The fitted curves of $\Delta u(r)/\sigma'_{vo}$ for the soil above the level of the bottom helix plate, from Figure 6.12, are compared on Figure 6.13 to the radial distributions through the soil opposite the bottom
of the pile shaft below the level of the bottom helix plate. Unlike the soil which has been passed by at least one of the helix plates, no residual $\Delta u_{hx}$ is observed in the soil (from $r/R_{\text{shaft}} = 5$ to at least 16) below the level of the bottom helix within 10 minutes after stopping penetration. From this figure it can also be seen that the dissipation of $\Delta u$ within the soil close to the helices ($r/R_{\text{shaft}} < \text{about 10}$) is much more rapid below the level of the bottom helix than above, at least during the first 20 hours of dissipation.

6.3.2 Dissipation at Different Radial Distances

Dissipation curves for different radial distances from the pile center are plotted on Figure 6.14a. These dissipation curves were generated from “best fit” curves of $\Delta u(r)/\sigma'_{vo}$ at different times (from Figure 6.12), which were based on $\Delta u$ measurements from various depths, and are therefore representative of the average response of the soil over the range of depths considered in this study. The data used to generate the $\Delta u(r)/\sigma'_{vo}$ curves for time $t \leq 60$ minutes was limited to locations opposite the lead section of the piles, after at least one of the helix plates had passed. Within the inter-helix region close to the pile ($r/R_{\text{shaft}} = 1$ to 4), the $\Delta u(t)/\sigma'_{vo} - \log(t)$ curves have a distinctive concave-downward shape. The curve for $r/R_{\text{shaft}} = 1$ appears to form an envelope, inside which all of the other dissipation curves plot. In fact, the dissipation curve for any location appears to form an envelope inside of which the dissipation curves for all larger radial distances plot. At locations close to and beyond the edge of the initial plastic zone, $\Delta u(t)/\sigma'_{vo}$ decreases essentially linearly with $\log(t)$, at least until the curve approaches the envelope established by the dissipation curve for the next distance closer to the pile.

The dissipation curves from Figure 6.14a are shown on Figure 6.14b normalized by the initial excess pore pressure immediately after stopping penetration, $\Delta u_0$, which includes the additional pore pressure due to the helices, $\Delta u_{hx}$. This brings the curves from different radial distances closer together, but does not unify them. This is due to the variation in the initial rate of relative dissipation at different distances from the pile shaft, which can be seen on Figure 6.14b. The initial rate of relative dissipation appears to increase with increasing distance from the pile shaft, and as a result, greater proportions of dissipation occur sooner at larger radial distances.

Dissipation curves based on $\Delta u(t)/\sigma'_{vo}$ data from individual piezometers/piezo-ports located at different radial distances from the piles are shown on Figure 6.15. Curves from locations above
and below the level of the bottom helix are also compared on this figure, and it is very clear that
dissipation below the level of the bottom helix occurs much more quickly than above, at radial
distances close to the pile. Only the data from the piezometers/piezo-ports located between –7.9
and –8.9 m elevation has been shown to represent the dissipation trends above the level of the
bottom helix. However, the dissipation curves from the piezometers located opposite the grout
column (around –6.0 m elevation) were found to be very similar to those measured opposite the
lead section at similar radial distances.

The trends with time of the dissipation curves from different radial distances on Figure 6.15 are
similar to those exhibited by the series of curves on Figure 6.14a. From the numerous
dissipation curves presented on Figures 6.14 and 6.15, it can be seen that, although greater
proportions of dissipation occur sooner at larger radial distances, all of the curves trend toward
convergence near the end of the dissipation process. For piles TP1 through TP6, 100%
dissipation occurred at around 10,000 minutes or about 7 days for most locations around the
piles.

6.3.3 Comparison of Dissipation at Pile Shaft to CPTU Dissipation Curves

The dissipation of excess pore pressure at the surface of a cylindrical pile or probe is usually
characterized by curves of normalized excess pore pressure, $\Delta u(t)/\Delta u_0$, or alternatively, degree of
dissipation, $U(t)$, vs. a normalized time factor, $T$, where:

- $U(t) = 1 - \Delta u(t)/\Delta u_0$, and $\Delta u_0$ is the initial excess pore pressure at the start of the dissipation
  process,
- $T = c_h t/R^2$, and $c_h$ is the coefficient of horizontal consolidation, $t$ is the time since the start of
  the dissipation process, and $R$ is the radius of the cylindrical pile or probe.

Plots of $\Delta u(t)/\sigma'_{vo}$ at the shaft of the helical piles vs. $T$ (calculated using the radius of the pile
shaft, $R_{shaft}$ and $c_h = 1.1 \text{ cm}^2/\text{min}$) are presented on Figure 6.16. The dissipation curves for the
pile shaft are compared to measured CPTU dissipation curves on Figures 6.16a, as well as to
theoretical solutions for CPTU dissipation from Levadoux & Baligh (1986)$^3$ on Figure 6.16b.

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$^3$ The Levadoux & Baligh consolidation solutions were used for comparison since the radial distribution of $\Delta u(r)$,
predicted by Levadoux & Baligh (near the U3 location in particular) is in reasonable agreement with the distribution
of pore pressure generated by the penetration of the pile shaft.
For the measured CPTU dissipation curves, T is calculated using the radius of the shaft of the cone penetrometer.

Dissipation curves from the PP1, PP2 and PP3 piezo-ports, located on the pile shaft 15 cm below the top helix plate, 23 cm below the middle helix, and 7.5 cm below the bottom helix plate, respectively, are all shown on Figure 6.16. It is readily apparent that the dissipation at the locations between the helices is very consistent, regardless of the proximity of the piezo-ports to the helix plates or the spacing of the helices. However, dissipation at the pile shaft between the helices is slower than that recorded below the bottom helix plate until about half of $\Delta u_o$ has dissipated, and consequently, there is a significant delay in the relative amount of dissipation occurring between the helices. The total duration of the dissipation process (98 to 99 percent dissipation) at TP4-PP3, located on the pile shaft below the bottom helix, was about 1.5 to 2.5 times shorter than that recorded on the shaft between the helices. The slower dissipation between the helices may be due to any one of the following reasons, or to some combination:

- At radial distances in excess of about 15 shaft radii from the pile center, the excess pore pressures opposite the helices after pile installation are significantly higher than those measured below the level of the bottom plate.

- The pore water drainage below the bottom helix may occur through soil that is more intact structurally, and therefore, may have a higher $c_h$ value.

- Pore water drainage may be occurring both vertically and radially below the bottom helix, but can only occur radially between the helices.

On Figure 6.16a, it can be seen that the measured pore pressures at both the U2 and U3 filter positions, after the initial pore pressure redistribution described in Section 4.5.2, are greater than those measured at the pile shaft at early times in the dissipation process. The initial increase in $\Delta u$ along the shaft of the cone probe, which occurs immediately after stopping cone penetration, was not observed at any of the piezo-port locations on the pile shaft, even at TP4-PP3 near the tip of the pile shaft. This may be due to the presence of the bottom helix plate which constrains the soil in the vertical direction, so that vertical soil deformations away from the tip, which are believed to cause the stress redistribution along the shaft of the cone probe, cannot occur. Once the excess pore pressures have dissipated below a level of about $\Delta u(t)/\sigma'_{vo} \approx 1.0$, the remainder
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of the U2 and U3 dissipation curves plot close to the dissipation curves for the pile shaft, but do not follow the same trends with time.

On Figure 6.16b, it can be seen that both the U2 and U3 dissipation curves predicted by Levadoux & Baligh (1986) for normally consolidated BBC closely follow the curve for TP4-PP3 (below the bottom helix) through most of the dissipation process, once the excess pore pressures have dissipated below a level of about $\Delta u(t)/\sigma'_{vo} \approx 1.0$ at $T \approx 5$. However, the Levadoux & Baligh curves predict much faster dissipation than that which is observed along the pile shaft between the helices.

On Figure 6.17, the dissipation of $\Delta u$ at the pile shaft is normalized by $\Delta u_o$ instead of $\sigma'_{vo}$. The resulting dissipation curves are compared to theoretical solutions of $\Delta u(t)/\Delta u_o$ vs. $T$ for the U2 and U3 filter positions by Levadoux & Baligh (1986) and by Teh & Houlsby (1991), converting from $T^* = c_h t/(r_o^2 I_r^{1/2})$ to $T$ using $I_r = 350$ on Figure 6.17a. Both of the theoretical solutions for dissipation at $U2$ plot well to the left of the observed dissipation at the pile shaft. The theoretical solutions for dissipation at $U3$ follow the curves for locations on the pile shaft between the helices at early times in the dissipation process, but predict faster dissipation at intermediate times, such that they follow the curve for TP4-PP3 at later times. Based on the reasonable agreement between the theoretical $U3$ dissipation curves and the pile shaft curves, the measured $U3$ curves are also compared to the pile shaft curves on Figure 6.17b. The measured $U3$ dissipation curves have been plotted using the “log time” correction method (Sully et al., 1999) described in Section 4.5.3 in order to eliminate the initial increase in $\Delta u$ which was not observed at the pile shaft. The measured $U3$ pore pressures plotted this way were found to be in good agreement with $\Delta u(t)/\Delta u_o$ at the pile shaft below the bottom helix plate (TP4-PP3) up to about $T=60$ (75% dissipation). At greater degrees of consolidation, the rate of CPTU dissipation becomes much slower than that observed on the pile shaft and so the dissipation curves diverge. The reason for the slowing of the dissipation process during the late stages of the CPTU dissipation test is unknown.

Although the dissipation curves from the helical piles differ from those measured with the piezocone, the normalized time scales are similar when the pile data is normalized using the radius of the pile shaft. This indicates that, while the additional pore pressures generated by the
helix plates do cause slightly longer dissipation times in the Colebrook clayey silts, the duration of the dissipation process is mainly controlled by the diameter of the pile shaft.

6.3.4 Summary and Conclusions from Observed Dissipation Behaviour

The following key points from the discussions in Section 6.3 are important for understanding and attempting to predict the dissipation of excess pore pressures around the helical piles:

1) The total duration of the excess pore pressure dissipation period depends primarily on the radius of the pile shaft, \( R_{\text{shaft}} \), and increases with the square of \( R_{\text{shaft}} \).

2) The dissipation process appears to be essentially independent of the number or spacing of the helix plates.

3) Throughout most of the dissipation process, the time required to achieve a given degree of dissipation (U) decreases with increasing radial distances from the pile shaft.

4) Dissipation occurs much more quickly within the soil below the level of the bottom helix than within the soil above the level of the bottom helix plate, for the first half of the dissipation process. The total duration of the dissipation process at the pile shaft below the bottom helix was found to be about 1.5 to 2.5 times faster than was observed at the pile shaft between the helices.

5) The dissipation at the pile shaft between the helices was found to be slower than the measured or predicted CPTU dissipation curves for either the U2 or U3 filter locations along the shaft of the cone probe. This may be due to higher \( \Delta u \) at the “tail” of the radial distribution around the helical piles, and/or possibly due to a greater reduction in \( c_{\text{h}} \) resulting from destructuring caused over a greater \( r/R \) for the helical piles.

6) The measured CPTU dissipation curves at the U3 location were found to be adequate to estimate the dissipation below the bottom helix plate, when plotted as \( \Delta u(t)/\Delta u_o \) vs. T, and after correcting the CPTU data using the “log time” correction method proposed by Sully et al. (1999).

7) Theoretical solutions in terms of \( \Delta u(t)/\Delta u_o \) vs. T, which were generated from the Strain Path Method (eg. by Teh & Houlsby, 1991) for dissipation around the piezocone probe (for
locations about 5 shaft radii above the shoulder of the cone, or higher) were found to give a rough estimate of the total duration of the dissipation process around the helices ($U \geq 90\%$).

8) Ultimately, the CPTU dissipation curves are useful only as rough order-of-magnitude estimates of the dissipation around the piles. A numerical solution of the consolidation process, based on the initial distribution of $\Delta u(r)$ around the helical piles, would be required to obtain a better prediction of the dissipation around the piles.

6.4 PORE PRESSURE GENERATION & DISSIPATION AROUND HELICAL PILES COMPARED TO CONVENTIONAL DISPLACEMENT PILES

One major advantage of helical piles is that the diameter of the shaft is small relative to the diameter of the helix plates which controls the pile capacity. This is important since the radial distances over which excess pore pressures are generated during pile installation increase linearly with increasing radius of the pile shaft in a given soil. Consequently, the volume of soil subjected to excess pore pressures after driving a cylindrical displacement pile will be much larger than that around a “low-displacement” helical pile. To illustrate this, the radial distribution of $\Delta u(r)/\sigma'_{vo}$ observed around the helical test piles is compared on Figure 6.18 to that predicted around the shaft of a hypothetical 356 mm diameter steel pipe pile (same diameter as the helix plates) installed closed-ended in soil with similar pore pressure behaviour.

The $\Delta u(r)/\sigma'_{vo}$ distribution predicted for the hypothetical pipe piles is based on the normalized $\Delta u(r/R)/\sigma'_{vo}$ distribution (radial distance, $r$, normalized by the shaft radius, $R$) opposite the U3 filter position on the cone penetrometer predicted by Levadoux & Baligh (1980). As discussed in Sections 6.2.3 and 4.5.2, the $\Delta u(r/R)/\sigma'_{vo}$ distribution due to cone penetration in NC BBC predicted by Levadoux & Baligh (1980) appears to be reasonably applicable to the measurements obtained in the Colebrook clayey silt. Closed-ended cylindrical piles will tend to generate normalized excess pore pressure distributions, $\Delta u(r/R)$, that are similar to those predicted for cone penetrometers, at least for locations away from the tip. Thus, the curve of $\Delta u/\sigma'_{vo}$ vs. $r/R$ opposite the U3 filter position (10R above the tip of the cone) by Levadoux & Baligh is plotted on Figure 6.18 against radial distance calculated using $R = 17.8$ cm for the 35.6 cm diameter pipe pile.
The $\Delta u/\sigma\nu_0$ that is shown at the surface of the helical pile shaft on Figure 6.18 is higher than that predicted for the shaft of the conventional pipe pile (or a piezocone probe) due to the additional pore pressures generated by the helices. However, between radial distances of about 40 cm and 300 cm from the center of the piles, the excess pore pressure generated by the penetration of the pipe pile is much higher than that generated by the helical piles. This can have important implications for the spacing of adjacent piles within pile groups. When the zones of excess pore pressure from adjacent piles overlap, the minimum $\Delta u$ between adjacent piles becomes elevated above zero. It can take very long periods of time for such elevated pore pressures within pile groups to fully dissipate.

The duration of the dissipation process increases with the volume of soil that is subjected to elevated pore pressures. Thus, the time required for reconsolidation of the soil after pile installation will be much less for helical piles than for conventional displacement piles which have much larger shaft diameters and induce significant excess pore pressures within larger volumes of soil. To illustrate this point, the dissipation curves obtained at the PP1 and PP3 positions on the shaft of the helical piles are compared on Figure 6.19 to the predicted dissipation around the shaft of the hypothetical 356 mm diameter pipe pile. The dissipation curve for the hypothetical pipe pile was generated using the post-peak (after initial pore pressure redistribution) CPT(U3) dissipation curve measured at -6.3 m elevation at Colebrook, with the time scale corrected to account for the larger shaft radius using the following expression, as suggested by Robertson et al. (1989):

$$t_{\text{pile}} = t_{\text{CPT}} \cdot (R_{\text{pile}})^2/(R_{\text{CPT}})^2 \quad (6.4)$$

where: $t_{\text{pile}}$ = time for given degree of dissipation around pile,

$t_{\text{CPT}}$ = time for same degree of dissipation around cone probe,

$R_{\text{pile}}$ = radius of the pile shaft, and

$R_{\text{CPT}}$ = radius of the shaft of the cone probe.

The observed dissipation data was extrapolated to 95% dissipation using the method described in Appendix E.4.
From Figure 6.19, it is very clear that the dissipation times around the conventional pile will be much slower than around the helical piles. On the shaft of the helical piles between the helices, 120 hours (5 days) is required for 90% dissipation, as compared to an estimated 2000 hours (83 days) to achieve 95% dissipation around a 356 mm diameter pipe pile. For helical piles with more slender shafts, which are more likely to be used in soft soils, the dissipation time would be even shorter.

6.5 Conclusion

The measured radial distribution of excess pore pressure indicates that the initial penetration of the pile shaft causes most of the pore pressure generation. However, additional pore pressure is generated close to the pile by the penetration of the helix plates ($\Delta u_{hx}$), which is believed to be due to the upward pressure exerted by the helix plates on the soil above the plates during downward penetration. The observed pore pressure response suggests that the bottom helix generates most of the pull-down force, since a more subtle pore pressure response appears to accompany the penetration of successive plates.

Beyond the edge of the helices, only the soil located very close to the helix plates (within a distance of about 10 – 12 times the thickness of the plate) responds directly to the penetration of the plates. However, there appears to be a gradual outward propagation of $\Delta u_{hx}$ during continuing pile penetration, which is attributed to total stress redistribution effects. As a result of this outward propagation, excess pore pressures of at least 10% of the initial vertical effective stress are observed out to radial distances of about 60 shaft radii from the pile center. This can have important implications for the spacing of individual piles within large pile groups.

The observed pore pressure dissipation around the piles indicated that primary reconsolidation of the soil was essentially complete after about 5-7 days. This is significantly faster than is typically observed around conventional full displacement piles with diameters similar to that of the helix plates. It appears that the dissipation below the bottom helix plate can be predicted with reasonable accuracy from dissipation data obtained from the piezocone probe with pore pressure measurements above the friction sleeve. However, the dissipation between the helix plates appears to occur more slowly than below the bottom plate. The total duration of the dissipation process on the pile shaft between the helices was found to be about twice as long as was measured on the shaft below the bottom plate.
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Figure 6.1: Inclinometer Probe in Guide Casing

Figure 6.2: Method of Pile Installation
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Excess Pore Pressure during Pile Installation - $\Delta u/\sigma'_{vo}$

Note: Dissipation during breaks in installation removed.

Figure 6.3a: Variation of Excess Pore Pressure with Pile Tip Depth (S/D=1.5)
Chapter 6 – Pore Pressure Changes During and After Pile Installation

Excess Pore Pressure during Pile Installation - $\Delta u/\sigma'_o$

![Graph showing variation of excess pore pressure with pile tip depth](image)

Note: Dissipation during breaks in installation removed.

Figure 6.3b: Variation of Excess Pore Pressure with Pile Tip Depth (S/D=3)
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Figure 6.4: Variation of Excess Pore Pressure with Time During Pile Installation

a) S/D = 1.5

b) S/D = 3

Note: at zero time, \( \Delta u = \Delta u_{\text{shaft}} \) (approx.)

\( r = \) radial distance from pile center
\( R = \) radius of pile shaft
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**Figure 6.5:** Excess Pore Pressure at Pile Shaft During Pile Installation

**Figure 6.6:** Installation Pore Pressures at Pile Shaft Compared to CPTU Penetration Pore Pressures
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Figure 6.7: Radial Distribution of Excess Pore Pressure Generated by Penetration of Pile Shaft
Figure 6.8: Radial Distribution of Maximum Excess Pore Pressure after Penetration of Helices
Figure 6.9: Radial Distribution of Excess Pore Pressure from this Study Compared to Strain Path Method Prediction by Levadoux & Baligh

Figure 6.10: Radial Distribution of Excess Pore Pressure from this Study Compared to Cylindrical Cavity Expansion Prediction
Figure 6.11: Estimate of Excess Pore Pressure Generated by Bottom Helix from CPTU Data
Chapter 6 – Pore Pressure Changes During and After Pile Installation

Figure 6.12: Radial Distribution of Excess Pore Pressure around Helical Piles (Above Level of Bottom Helix) during Dissipation Process

Figure 6.13: Radial Distribution of Excess Pore Pressure Above & Below Level of Bottom Helix during Dissipation Process
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Figure 6.14: Average Dissipation Trends for Different Radial Distances from Pile
Figure 6.15: Dissipation Curves from Piezometers/Piezo-Ports Located at Different Radial Distances from Pile
Chapter 6 – Pore Pressure Changes During and After Pile Installation

Figure 6.16: Dissipation Curves ($\Delta u(t)$ Normalized by $\sigma'_{vo}$) from Piezo-Ports Located on Pile Shaft Compared to CPTU Dissipation Curves
a) Pile Dissipation Compared to Theoretical CPTU Solutions

![Graph showing dissipation curves normalized by Δu₀ compared to theoretical CPTU solutions.]

b) Pile Dissipation Compared to Measured CPTU Dissipation

![Graph showing dissipation curves normalized by Δu₀ compared to measured CPTU dissipation.]

Figure 6.17: Dissipation Curves (Δu(t) Normalized by Δu₀) from Piezo-Ports Located on Pile Shaft Compared to CPTU Dissipation Curves
Chapter 6 – Pore Pressure Changes During and After Pile Installation

Figure 6.18: Radial Distribution of Excess Pore Pressure Predicted Around Conventional Pipe Pile Compared to Observed Distribution around Helical Piles

*Note: Pipe pile prediction based on CPT(U3) prediction by Levadoux & Baligh (1980)
Figure 6.19: Dissipation of Excess Pore Pressure Predicted for Conventional Pipe Pile Compared to Observed Dissipation at Shaft of Helical Piles

Note:
Estimate for conventional pile based on observed CPT(U3) dissipation (after initial redistribution) at -6.3 m elevation, extended to 95% dissipation using method from Appendix E.4, and \( t_{\text{pile}} = t_{\text{CPT}} (R_{\text{pile}})^2/(R_{\text{CPT}})^2 \)
7.0 LOAD TESTING

In this chapter, the data obtained during the load testing program are presented and discussed. The observed pile and soil behaviour that will be discussed in this chapter includes:

- the observed transfer of load from the soil to the pile during the reconsolidation process prior to load testing,
- observations about the general distribution of load carried by the piles at different stages of the tests up to and including failure,
- the load-settlement behaviour of the various sections of the piles,
- the settlement trends with time observed during the brief load intervals,
- the pore pressure response of the soil surrounding the piles during the load tests,
- the changes in the capacity of the various sections of the piles with time,
- comparisons of the ultimate bearing pressures below the helix plates to measured CPT tip pressures,
- the inferred mobilization of undrained shear strength by the different sections of the piles at different times after installation, and comparisons of these strengths with undrained strengths of the intact and remoulded soil measured by the field vane, and
- comparisons of the ratio of the undrained shear strength to vertical effective stress at the end of the reconsolidation period, with strength ratios of the intact soil and of the soil in a normally consolidated state.

First, the apparatus and procedures used to load the helical test piles to failure and to monitor the behaviour of the piles and the soil during the loading process are described in Section 7.1 below.

7.1 TESTING & MONITORING APPARATUS AND PROCEDURES

After allowing a recovery period of 19 hours (TP5&6), 7 days (TP3&4, PP1) or 6 weeks (TP1&2), axial compressive load tests were carried out on each test pile in general accordance with the setup and procedures specified as the “quick load test method for individual piles” in ASTM Standard D1143. An outline of the specific setup and procedures used during this study is provided below.
7.1.1 Test Setup

The load was applied to the top of the piles using a hydraulic ram with a hand pump that allowed for small increments of load. The ram is shown at the top of a test pile on Figure 7.1. The reaction for the ram was provided by a single main loading beam that spanned between a pair of smaller reaction beams at each end, as shown on Figure 7.2. Each pair of reaction beams was supported on timber cribbing and anchored via thread bars to 2 or 3 helical reaction piles, such that at least 4 reaction piles were used to anchor the reaction frame for each test.

An example of the loading and settlement measuring systems at the top of the test piles is shown on Figure 7.1. The applied load at the pile head was measured using the same load cell that was used to calibrate the strain gauges, which was positioned between the base of the ram and the top of the pile cap. The pile cap, which was bolted to the top of the pile shaft, was used as the reference point for the settlement measurements. The settlement was measured using a pair of 2” dial gauges (0.001” resolution) located on diametrically opposite sides of the pile head. Each dial gauge was fixed to a timber beam with supports located at a clear distance of 2.5 m from the test pile. The settlement of the test piles was also checked using a surveyor’s level, which was trained on a measuring tape that was fixed to the top of the pile cap.

The standard strain gauge and piezo-port locations on the test piles are indicated on Figure 7.3. The actual instruments used on each of the test piles, including the piezometers surrounding the piles, which were monitored during the load tests, are listed in Table 7.1. In addition to the 6 fully instrumented test piles, the results of a load test carried out 7 days after installation on a partially instrumented practice pile PP1 (S/D=3) have also been considered and so the instrumentation for this pile has been included.
Table 7.1
Instrumentation for Load Tests

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Helix S/D</th>
<th>Strain Gauges</th>
<th>Piezo-Ports</th>
<th>Piezometers</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP1 (7-day recovery)</td>
<td>3</td>
<td>SG1, SG4, SG6</td>
<td>PP2</td>
<td>PZ-PP1-1</td>
</tr>
<tr>
<td>TP1 (6-week recovery)</td>
<td>3</td>
<td>SG1&amp;2 (data lost), SG3</td>
<td>PP1, PP2, PP3</td>
<td>PZ-TP1-2, PZ-TP1-3, PZ-TP1-4, PZ-TP1-5, PZ-TP1-6, PZ-TP1-7, PZ-TP1-9</td>
</tr>
<tr>
<td>TP2 (6-week recovery)</td>
<td>1.5</td>
<td>SG1, SG2, SG3</td>
<td>PP1, PP2, PP3</td>
<td>PZ-TP2-1, PZ-TP2-2, PZ-TP2-3, PZ-TP2-5, PZ-TP2-6, PZ-TP2-7, PZ-TP2-9</td>
</tr>
<tr>
<td>TP3 (7-day recovery)</td>
<td>3</td>
<td>SG1, SG2, SG3, SG4, SG5, SG6</td>
<td>PP3</td>
<td>PZ-TP3-1, PZ-TP3-2</td>
</tr>
<tr>
<td>TP4 (7-day recovery)</td>
<td>1.5</td>
<td>SG1, SG2, SG3, SG4, SG5, SG6</td>
<td>PP3</td>
<td>PZ-TP4-1, PZ-TP4-2</td>
</tr>
<tr>
<td>TP5 (19-hour recovery)</td>
<td>3</td>
<td>SG2, SG3, SG4, SG5, SG6</td>
<td></td>
<td>PZ-TP5-1, PZ-TP5-2</td>
</tr>
<tr>
<td>TP6 (19-hour recovery)</td>
<td>1.5</td>
<td>SG1, SG2, SG3, SG4, SG5, SG6</td>
<td></td>
<td>PZ-TP6-1, PZ-TP6-2</td>
</tr>
</tbody>
</table>
7.1.2 Test Procedures

During the tests, load was applied in increments with each increment maintained for about 5 minutes. The load increments, total time to failure and the average loading rate used in each load test are indicated in Table 7.2. The load increment of 10 kN used in the initial load tests was selected to correspond to about 10% of the anticipated pile capacity. However, Pile TP1 did not even reach 100 kN and so the time to failure was less than anticipated. For the remainder of the piles (TP2-6), more closely spaced load intervals were desired in order to obtain smoother load-settlement curves and a more confident measure of capacity. Thus, the average loading rates for piles TP2 through TP6 were significantly slower than that of pile TP1.

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Helix S/D</th>
<th>Recovery Time (days)</th>
<th>Total Pile Capacity (kN)</th>
<th>Increment of Load (kN)</th>
<th>Time to Failure (min.)</th>
<th>Average Loading Rate (kN/min.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>PP1 (excl. casing)</td>
<td>3</td>
<td>7</td>
<td>77</td>
<td>~8 (0 to 64), ~4 (64 to 76)</td>
<td>62</td>
<td>1.24</td>
</tr>
<tr>
<td>TP1</td>
<td>3</td>
<td>42</td>
<td>80</td>
<td>10</td>
<td>41</td>
<td>1.95</td>
</tr>
<tr>
<td>TP2</td>
<td>1.5</td>
<td>43</td>
<td>90</td>
<td>4</td>
<td>115</td>
<td>0.78</td>
</tr>
<tr>
<td>TP3</td>
<td>3</td>
<td>7</td>
<td>76</td>
<td>4</td>
<td>103</td>
<td>0.74</td>
</tr>
<tr>
<td>TP4</td>
<td>1.5</td>
<td>7</td>
<td>76</td>
<td>4</td>
<td>99</td>
<td>0.77</td>
</tr>
<tr>
<td>TP5</td>
<td>3</td>
<td>0.8</td>
<td>68</td>
<td>2.5</td>
<td>145</td>
<td>0.47</td>
</tr>
<tr>
<td>TP6</td>
<td>1.5</td>
<td>0.8</td>
<td>63</td>
<td>2.5</td>
<td>129</td>
<td>0.49</td>
</tr>
</tbody>
</table>

In all cases, failure occurred in a rapid plunging manner, and the peak load could not be sustained even with continuous jacking. Once this had occurred, jacking was stopped and the load was allowed to stabilize as the plunging stopped. The remainder of the load was then removed in decrements of about 25% of the maximum load.

Throughout the load tests carried out on test piles TP1 through TP6, the load cell, strain gauges, piezo-ports and piezometers were all monitored continuously using the multi-channel data acquisition system. Dial gauge readings were taken by research assistants about 8 times during every 5 minute load interval. Level readings of the measuring tape attached to the pile cap were taken once every 1 or 2 load intervals.
7.2 **LOAD TEST RESULTS**

The behaviour of the test piles during the load tests will be presented in this section and comparisons will be made between the S/D=1.5 and S/D=3 piles, as well as between the results from tests carried out at 19 hours, 7 days, and 6 weeks after installation.

As was discussed in Section 6.2.4, the process of pile installation is likely to have caused very large strains within the annulus of soil through which the helices have passed, which would lead to some degree of strength reduction due to strain softening. The actual degree and extent of the strength reduction is not known for certain. However, based on the observed pore pressure responses, it seems reasonable that the strength loss around the test piles would be significant from the pile shaft out to at least 10 cm from the edge of the helices. Between the pile shaft and the edge of the helices, the shear strength may be reduced to close to the remoulded strength of the material.

After 19 hours, approximately 65 to 70 percent of the excess pore pressures near the edge of the helices had dissipated, which would have allowed for some strength recovery due to increased effective stresses within the soil surrounding the piles. At 7 days, the dissipation process was essentially complete, and hence, the increase in soil strength coinciding with the increased effective stresses should have been essentially complete. For the tests carried out 6 weeks after installation, almost 1 log cycle of time had elapsed after the completion of the reconsolidation process, which would allow for some aging-related strength gain due to secondary consolidation and thixotropic hardening.

7.2.1 **Load Transferred to Pile During Reconsolidation**

On piles TP3 and TP4, the strain gauges were monitored during the 7-day reconsolidation process. The load within the pile shaft after pile installation was calculated relative to the initial strain gauge baselines measured before pile installation while each pile section was standing upright with no external applied loads. Thus, during these baseline measurements, the only load in the pile section was due to the self-weight of the portion of the pile shaft above the strain gauge.

The net loads transferred to the shaft of the lead sections of TP3 and TP4 during the excess pore pressure dissipation period are plotted on Figure 7.4, along with representative pore pressure
dissipation curves for the pile shaft and for the soil at the edge of the helices. During pile installation, the helices pull the remainder of the pile into the ground, and therefore it is expected that there would be some residual net tensile load in the pile shaft immediately after pile installation. Within about 4 hours after installation, the average net load acting on the pile shaft is about zero. All of the strain gauges shown on Figure 7.4 show a consistent increase in compressive load within the first 3 days after pile installation. After about 19-20 hours, the compressive load within the pile shaft is about 2 kN, and by day 7, it is typically between 3 and 3.5 kN. The magnitude of the loads measured at SG4, SG5, and SG6 are very similar.

As shown on Figure 7.4, the increase in load within the pile shaft does appear to correlate roughly with the degree of dissipation within the soil close to the pile. This suggests that the net increase in load within the pile is a result of loads transferred to the pile by the soil due to the decreasing volume and subsequent settlement of the soil adjacent to the pile during the consolidation process. This is a common soil-pile interaction phenomenon known as “negative skin friction” which produces down-drag forces on the pile which must be resisted by the soil around the lower portion of the pile. The distribution of compressive load developed within the shaft of the lead section indicates that the down drag is developed above SG4 and is resisted almost entirely by the bottom helix plate (and tip of pile shaft). The down-drag is most likely developing along the surface of the grout column, since very little soil settlement is required to cause a drag force along frictional soil-pile contacts. The load development trend between 3 and 7 days after installation suggests that little, if any, additional down-drag would have occurred due to secondary consolidation once excess pore pressure dissipation was complete at about 7 days.

If the pile load development curves follow the pore pressure dissipation curves at times earlier than about 4 hours, the initial tensile load in the pile shaft may be on the order of 1 to 1.5 kN. Thus, the total compressive load transferred to the pile by the consolidating soil may be around 4 to 6 kN at 7 days, and around 3 to 4 kN at 19 hours.

Part of the observed increase in compressive load within the lead section will be due to the weight of the upper extensions and grout column which was not carried by the lead section during the pre-installation baseline readings. The weight of the upper extension sections, including grout, was estimated to be about 2.1 to 2.2 kN. Part or all of this weight may be initially carried by the soil due to upward shear resistance that is mobilized as the extensions and
grout discs are pulled into the ground. However, as the soil adjacent to the grout column consolidates and settles, any weight that was carried by the soil will be transferred to the pile shaft. This will occur before negative skin friction develops. The pile self-weight has been intentionally neglected from the analysis of pile capacity and soil resistance since the total pile weight is very close to the estimated weight of the volume of soil displaced by the pile.

Therefore, the additional weight of the upper extensions and grout column needs to be subtracted from the compressive loads shown on Figure 7.4 in order to obtain the net down-drag force due to negative skin friction. When this is done, the net downdrag force is found to be only about 0.5 kN or less at 19 hours and about 1 kN at 7 days. Since most, if not all, of this load appears to be transferred to the soil by the bottom helix plate (and tip of pile shaft), a corresponding increase in $\sigma'_v$ of about 10 kPa can be expected below the pile by the end of the pore pressure dissipation period.

In most cases, the compressive loads developed in the pile shaft at the strain gauges within the grout column were much greater than those described above, but these loads are believed to be primarily a result of stresses induced within the grout during curing. Therefore, they are not considered to be representative of the true soil-pile load transfer and so have not been included in subsequent analyses.

### 7.2.2 Distribution of Load Along Pile During Loading

Based on the different load transfer mechanisms employed by the structural elements of the pile, and the different soil disturbance effects caused by those elements during installation, the helical piles have been subdivided into the following sections for consideration in this study:

- **upper extension with twin-walled casing system** (load cell to 0.3 m below SG1) – designed to isolate upper extension section from soil resistance,

- **surface of grout column** (0.3 m below SG1 to 0.3 m below SG3) – 2.8 m long,

- **bottom grout disc** (located between SG3 & SG4),

- **shaft of lead section above helices** (0.7 m long between bottom grout disc and top helix, located almost entirely between SG3 & SG4),
• **upper helices** (between SG4 and SG6) – comprised of 2 helices spaced at 3 diameters apart (S/D=3) or 4 helices spaced at 1.5 diameters (S/D=1.5),

• **bottom helix**, including bottom of pile shaft (below SG6).

The distribution of load, Q, carried by the test piles at various load levels during the load tests was determined from the load cell and strain gauge measurements. For some of the strain gauges located within the grout column (SG1, SG2 and SG3 locations), correction factors had to be applied to the laboratory-determined load calibration factors to account for the effects of load sharing between the pile shaft and the grout column. No such corrections were required for the strain gauges attached to the lead sections of the piles. The correction factors used for the various strain gauge installations, and a discussion of the rationale behind them, is provided in Appendix G.1.

Complete load distributions could be established for piles TP3 through TP6, which had full sets of strain gauges, and are included on Figures G-3a to G-3d in Appendix G. Since piles TP1 and TP2 did not have any strain gauges located on the lead section, the distributions for these piles have not been included. It is very important to note that the loads on Figures G-3(a to d) were referenced to baseline readings taken at the beginning of the load test, before any load had been applied to the top of the pile. However, as discussed in Section 7.2.1, there is an initial load distribution that develops within the pile before any load is applied to the pile top, which is ignored by taking strain gauge baselines at the start of the load test. Thus, the load distributions provided in Appendix G represent the distribution of the applied loads within the piles, rather than the true distribution of resistance mobilized by the soil. The portions of the applied load transferred to the soil by the various segments of the test piles are plotted as individual curves on Figures G-4a to G-4d (for piles TP3 through TP6) in Appendix G.

The shape of the load distributions for piles with S/D of 1.5 and 3, tested at 19 hours and 7 days after installation, were found to be similar. An example of a series of load distributions determined for pile TP6 (S/D = 1.5, tested 19 hours after installation) is presented on Figure 7.5. The initial distribution of load in the pile at the start of the test (zero applied load, pile self-weight neglected), which was inferred from the load development curves presented on Figure 7.4, is also shown. The load distributions corresponding to various applied load levels, which are shown on Figure 7.5, have been corrected to account for this initial load distribution,
which was present when the strain gauge baselines were taken prior to loading the piles. The data points shown on Figure 7.5 correspond to the locations along the shaft of the pile where the loads were measured. The dashed lines correspond to the average gradient of load transferred to the soil between successive measuring points (but not necessarily to the actual distribution across the interval). The positions of the strain gauge installations relative to the different structural elements on the test piles, which may mobilize resistance differently, are also indicated on Figure 7.5.

The distribution of load within piles TP3 to TP6 at the onset of pile failure (i.e. at $Q_{ult}$) and at $Q = 0.5Q_{ult}$ (i.e. at a working load corresponding to a factor of safety of 2.0) is summarized in Table 7.3.

<table>
<thead>
<tr>
<th>Test Pile:</th>
<th>TP5</th>
<th>TP6</th>
<th>TP3</th>
<th>TP4</th>
</tr>
</thead>
<tbody>
<tr>
<td>Helix S/D:</td>
<td>3</td>
<td>1.5</td>
<td>3</td>
<td>1.5</td>
</tr>
<tr>
<td>Recovery Time After Installation:</td>
<td>19 hrs</td>
<td>19 hrs</td>
<td>7 days</td>
<td>7 days</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Load Level</th>
<th>Pile Section</th>
<th>Proportion of Total Resistance Mobilized by each Pile Section</th>
</tr>
</thead>
<tbody>
<tr>
<td>$Q = 0.5Q_{ult}$</td>
<td>Shaft above Helices</td>
<td>15%</td>
</tr>
<tr>
<td></td>
<td>Upper Helices</td>
<td>37%</td>
</tr>
<tr>
<td></td>
<td>Bottom Helix</td>
<td>48%</td>
</tr>
<tr>
<td>$Q = Q_{ult}$</td>
<td>Shaft above Helices</td>
<td>8%</td>
</tr>
<tr>
<td></td>
<td>Upper Helices</td>
<td>52%</td>
</tr>
<tr>
<td></td>
<td>Bottom Helix</td>
<td>40%</td>
</tr>
</tbody>
</table>

Note: Shaft above helices includes both grouted and ungrouted segments.

The following observations about the general load transfer behaviour of the piles in this study can be made from the measured load distributions (such as on Figure 7.5) and from the summary in Table 7.3:

- Virtually no load was transferred from the shaft of the test piles to the outer casing, indicating that the twin-walled casing system described in Section 5.1 was effective in isolating the pile shaft from friction through the upper 2.5 m of variable surficial soils. Conversely, the initial casing system used for the practice pile PP1, which consisted of two
tight-fitting PVC pipes, was not effective and significant resistance was mobilized through the upper soils.

- Under working load conditions (at $Q = 0.5Q_{ult}$), the 3.6 m long pile shaft above the helices (including the 2.84 m long grout column) mobilized about 15% of the total resistance at 19 hours after installation. At 7 days after installation (at the end of the pore pressure dissipation period), the resistance mobilized by the pile shaft had increased to 30-35% of the total resistance developed at $Q = 0.5Q_{ult}$.

- The maximum load transferred along the surface of the grout column (bottom of casing to SG3) occurred part way through the test, while the maximum loads transferred from the base of the column (SG3 – SG4) and by the helices (below SG4) tended to occur at or just before failure. This was particularly evident for the piles tested at 7 days.

- The proportion of the total resistance that was mobilized by the helices increased as the piles were loaded beyond $Q = 0.5Q_{ult}$ to failure. At the ultimate capacity of the pile, only about 10% of the total resistance developed at 19 hours after installation and about 20% of the total resistance at 7 days after installation was being picked up by the pile shaft above the helices.

- The resistance mobilized by the bottom helix plate and pile tip was significantly higher than that mobilized individually by any of the upper plates at $S/D = 3$ or by pairs of plates at $S/D = 1.5$. The resistance developed by the bottom plate and pile tip accounted for between 30% and 50% of the total loads measured in the piles tested at 19 hours and 7 days after installation. At $Q = 0.5Q_{ult}$ at 19 hours, the proportion of the total resistance that was developed by the bottom plate was particularly high at 45-50% of the total pile load.

- The maximum load mobilized by the bottom helix of TP4 (22 kN) was significantly lower than for the other piles tested at either 7 days or 19 hours (25 – 28 kN). However, the variation between the capacities mobilized by the bottom helix of the different piles is comparable to the variation observed between the undrained shear strengths measured at similar elevations from different locations at the test site.

- Throughout most of the load test, the distribution of load along the upper helices (above the bottom helix) appears to be relatively uniform, and significant differences in the transferred load magnitudes between the different helices do not appear until near the end of the test.
7.2.3 Load – Settlement Curves

Since the test piles in this study are relatively short, and the maximum applied loads are low relative to the axial stiffness of the piles, the amount of axial shortening of the piles was negligible compared to the measured settlements. Therefore, the settlement measured at the top of the pile could be used to determine load-settlement curves for different segments of the piles, which are presented below.

7.2.3.1 Entire Pile

Curves of total pile load vs. settlement for each pile are presented on Figure 7.6. The pre-failure points on these curves correspond to the settlements measured at the end of each load interval (i.e. after the load had been maintained for about 5 minutes). Since the load intervals were very brief, no attempt has been made to separate the initial settlement, measured during the increments of load, from the total settlement. Plots showing all of the measured settlement points during the tests on each pile are included on Figures G-5a to G-5f in Appendix G. The curves on Figure 7.6 are shown again on Figure 7.7 with the applied load normalized by the ultimate load at failure, and with the pile settlement normalized by the diameter of the helix plates, $D_{hx}$.

Failure of the piles is characterized by a clear maximum load that is accompanied by large settlements that can be seen at the top of the curves on Figures 7.6 and 7.7. Due to the rapid plunging behaviour during failure, it was not possible to sustain a fast enough jacking rate to maintain the load at failure for very long. The subsequent drop in load after jacking was stopped, while the piles continued to plunge, is recorded on the curves in Appendix G, but has not been shown on Figures 7.6 and 7.7.

The initial response of the pile to loading is quite stiff. As a result, the measured settlements at 50% of pile capacity are quite small: about 4 to 6 mm (1.2 to 1.5 percent of $D_{hx}$) at 7 days and 6 weeks, and 6 to 8 mm (1.7 to 2.2 percent of $D_{hx}$) at 19 hours. At load levels closer to failure, the settlement of the S/D=3 piles is noticeably greater than that of the S/D=1.5 piles.
7.2.3.2 Grout Column

Load-settlement curves for the portion of the load transferred to the soil by the grout column are presented on Figure 7.8. On Figure 7.8a, the curves are based on just the load transferred between the bottom of the casing and SG3, and should therefore be governed mainly by the frictional response of the soil. From these curves it can be seen that the initial response of the grout column is very stiff with most of the capacity being mobilized within the first 2 mm of movement. Most of the curves show a more gradual increase in resistance continuing after 2 mm of movement, with the peak resistance being mobilized at around 8 to 10 mm of settlement. For the piles tested at 7 days and 6 weeks (i.e. after dissipation of excess pore pressures), there is a post-peak reduction in resistance along the surface of the grout column. This is much less distinctive at the piles tested at 19 hours.

The surface of the grout column on pile TP3 exhibited a much more brittle response than the other piles, with the peak resistance occurring at 2 mm. This may be indicative of a purely frictional load transfer mechanism, which appears to only require about 2 mm of movement to fully mobilize in the soil at the Colebrook test site. The additional resistance mobilized after 2 mm of movement on the other piles may be due to some limited zones of bearing resistance which require greater movements to fully mobilize. Such zones could occur below the upper (18 cm diameter) and middle (15 cm diameter) grout discs if the edges of these discs were exposed beyond the surface of the grout column.

On Figure 7.8b, the curves are based on the load transferred by the entire grout column from the bottom of the casing to below the bottom grout disc (as measured at SG4, located just above the helices). Thus, the curves on Figure 7.8b include both the frictional resistance mobilized along the surface of the grout column as well as the bearing resistance mobilized by the bottom grout disc. The load-settlement curves for the bottom of the grout column (from SG3 to SG4) are included on Figure G-7 in Appendix G. The load mobilized along this section is believed to be dominated by the bearing resistance generated below the bottom grout disc. The mobilization of resistance at this location is much more gradual than above SG3, particularly at 7 days where the maximum resistance is not mobilized until failure of the pile when the settlement is large.

From Figure 7.8 it is apparent that the ultimate capacity of the grout column is at least doubled between 19 hours (65 to 70 percent dissipation) and 7 days (dissipation essentially complete).
This increase in capacity is attributed to the increase in soil strength that occurs within this zone of highly disturbed, softened soil as the excess pore pressures dissipate and the effective stress increases. There is a further increase in the resistance mobilized along the surface of the grout column (between the casing and SG3) from 7 days to 6 weeks which is attributed to aging effects.

7.2.3.3 Lead Section

The load-settlement curves for the lead section (all helices) are presented on Figure 7.9. The data points correspond to the loads measured at SG4, located just above the top helix. For piles TP1 and TP2, the lower-most strain gauges on the piles were located at SG3 and therefore, an estimate of the load-settlement behaviour between the SG3 and SG4 locations (described in Appendix G.4) was required to produce the load-settlement curves for the lead sections of TP1 and TP2. The upper and lower-bound estimates of the SG3-SG4 load-settlement relationships for the piles tested at 6 weeks are included on Figure G-7 in Appendix G. The corresponding best estimates of the loads carried by the lead sections of TP1 and TP2, along with the associated MPE, are also presented on Figure 7.9.

The curves on Figure 7.9 are very similar in shape to those presented on Figure 7.7, with the failure of the pile occurring once the maximum load of the lead section is achieved. The initial load mobilization by the lead section is not as stiff as for the grout column, with settlements of between 6 and 9 mm required to mobilize 50% of the lead capacity. It is also worth noting that the settlement required to mobilize the full capacity of the helix plates on the S/D=1.5 piles decreases with increasing recovery time.

The mobilization of the combined resistance of the upper helix plates (above the bottom plate, as measured between SG4 and SG6) with settlement is shown on Figure 7.10, while the load-settlement curves for the bottom helix plate are shown on Figure 7.11. A comparison of the curves on Figure 7.10 reveals that the load-settlement response of the upper helices is similar on both the S/D=1.5 and S/D=3 piles. At first, this seems somewhat surprising given the differences between the cylindrical failure mechanism assumed for the S/D=1.5 piles and the bearing failure mechanism assumed for the S/D=3 piles (as indicated by previous research and discussed in Section 2.3). Very little deformation is required to mobilize the full resistance in perfectly plastic or strain-softening fine-grained soils subjected to direct shear along a relatively
rigid interface, as was observed along the surface of the grout column. However, the helix plates transfer load to the soil below the plates in compression. Since this soil has been softened by installation disturbance, substantial deformations probably occur within this soil before there is sufficient displacement of the inter-helix soil relative to the soil outside the helices to cause a complete cylindrical failure surface to develop. The decreasing settlement to failure with increasing recovery time, which was observed for the S/D=1.5 piles, is probably due to the increasing stiffness of the softened soil between the helices during reconsolidation and aging.

7.2.4 Settlement – Time Response

The duration of the load increments used in the quick load tests in this study was not adequate to properly assess the time-dependent settlement (or creep) behaviour of the helical piles. However, during the upper load intervals, the settlement-time trends suggest that creep settlements could be large and so this behaviour does need to be considered. Plots of the incremental settlement vs. the logarithm of time during individual load intervals for each pile are included on Figures G-6a to G-6f in Appendix G. An example of such plots from pile TP3 is provided on Figure 7.12.

Based on the brief settlement-time curves from this study, on-going creep-induced settlements are expected at load levels above about 65 to 70 percent of the pile capacity. It does not appear that creep would be significant at load levels below about 60% of the pile capacity. However, this cannot be ascertained with any degree of confidence based on the brief load intervals used in this study. A sustained load test would be needed to properly assess the time-dependent performance of the helical piles in fine-grained soils such as those at the Colebrook site.

7.2.5 Pore Pressure Response

The monitoring of piezometers and piezo-ports on the pile shaft confirmed that the loading of the test piles was carried out rapidly enough that no dissipation of the excess pore pressures generated during each load increment occurred during the tests. Thus, the shearing behaviour of the soil can be considered to be undrained.

The pore pressure response of the soil to the load applied by the helix plates was evaluated by calculating the ratio of the increment in pore pressure, $\Delta u$, to the increment in vertical stress.
applied by the bottom helix plate, $\Delta \sigma_v$, during each load increment. This was done for both piles TP3 and TP4, where the load carried by the bottom helix plate (and tip of the pile shaft) was known from strain gauge readings (SG6) and where the pore pressure immediately below the plate had been measured (piezo-port PP3). The increment in vertical stress was calculated by assuming that the load measured at SG6 was transferred evenly across the combined area of the bottom helix plate and the tip of the pile shaft. For the pre-failure load increments, $\Delta u$ and $\Delta \sigma_v$ were calculated from measurements at the end of successive load intervals. The variation of $\Delta u/\Delta \sigma_v$ with settlement is shown on Figure 7.13. The pore pressure behaviour observed at TP4-PP3 was considered to be close to the true behaviour of the soil since this piezo-port had been observed to perform well during dissipation monitoring. However, the response of TP3-PP3 during dissipation monitoring appeared to lag, possibly due to poor saturation, and so the gradually increasing $\Delta u/\Delta \sigma_v$ during load testing was considered to be a product of lag effects.

On Figure 7.13, $\Delta u/\Delta \sigma_v$ below the bottom helix of TP4 is observed to increase initially to 0.66 within the first 3 to 4 mm of settlement, which is inferred to be the point at which localized failure occurs within the soil around TP4-PP3. The ratio then remains essentially constant at 0.66 to 0.67 for most of the test, until the onset of plunging failure, which occurs after about 50 mm of settlement. The ratio of $\Delta u/\Delta \sigma_v = 0.66$ to 0.67 measured between about 4 mm and 50 mm of settlement is considered to be the characteristic pore pressure parameter at failure\(^4\) for the soil around the tip of the pile shaft which had been destructured due to disturbance caused during pile installation. Once plunging failure begins, $\Delta u/\Delta \sigma_v$ is observed to increase from 0.7 to about 0.95 at a total settlement of about 95 mm, and then remains relatively constant at about 0.95 to 1.0 for an additional 40 mm of settlement. The higher $\Delta u/\Delta \sigma_v$ ratio of about 1.0 attained during plunging failure is considered to be the characteristic pore pressure parameter at failure of the intact soil, which is inferred to have been encountered by the piezo-port after about 95 mm of downward movement. This is roughly the distance from the piezo-port to the tip of the pile shaft, which suggests that the amount of destructuring due to pile installation within the soil immediately below the pile tip is negligible.

\(^4\) The pore pressure ratio $\Delta u/\Delta \sigma_v$ measured below the pile after the first 4 mm of settlement is analogous to Skempton’s pore pressure parameter at failure, $A_f$, which is used to characterize the pore pressure response to axisymmetric loading in the triaxial cell.
The $\Delta u/\Delta \sigma_v$ ratio measured within the intact soil below the pile tip should be similar to the CPTU $B_q$ parameter ($B_q = \Delta u/(q_T-\sigma_{vo}) = \Delta u/\Delta \sigma_v$) when $\Delta u$ is measured on the face of the cone. The $\Delta u/\Delta \sigma_v$ ratios for TP4 are plotted against the elevation of TP4-PP3 on Figure 7.14 in order to compare these values to the measured $B_q$ values from the U1 and $q_T$ measurements at CPT-1 (presented earlier on Figure 4.8). It is clear that the $\Delta u/\Delta \sigma_v$ ratios measured below the bottom helix of the pile during plunging failure are almost identical to the $B_q$ values over a similar range of elevations.

The generation of $\Delta u$ (normalized by $\sigma_{vo}'$ at the start of the test) with pile settlement during load testing is shown on Figure 7.15 for some of the piezometers and piezo-ports located at different radial distances around the helices. The radial distances ($r$) indicated on Figure 7.15 have been normalized by the radius of the helix plates ($R$). The $\Delta u/\sigma_{vo}'$ vs settlement curves around the upper helices of the S/D=3 and S/D=1.5 piles and below the bottom helix are compared on Figures 7.15a, 7.15b and 7.15c, respectively.

Positive pore pressures, which diminish with radial distance from the pile shaft, are generated within the soil located below the helix plates and within the soil beyond the edge of the helices and below the level of the adjacent plate. Negative pore pressures are generated within the soil at or above the level of the adjacent plate (the change from positive to negative $\Delta u$ generation as the helix plate settles below the level of PZ-TP1-5&6 can be seen on Figure 7.15a). It is also readily apparent from Figure 7.15 that the magnitude of $\Delta u/\sigma_{vo}'$ measured at various locations around the upper helices of the S/D=1.5 piles are significantly lower than at similar locations around the S/D=3 piles. By far the highest $\Delta u/\sigma_{vo}'$ magnitudes were measured below the bottom helix.

The radial distributions of $\Delta u/\sigma_{vo}'$ at pile failure, which were measured below the level of the top and middle helices of the S/D=1.5 and S/D=3 piles and below the level of the bottom helix plate, are compared directly on Figure 7.16. At the pile shaft, $\Delta u$ generated below the helices of the S/D=1.5 pile (TP2) were about one third of those measured below the helices of the S/D=3 pile (TP1). There were no piezometers located around the helices of any of the S/D=1.5 piles between $r/R_{helix} = 1.0$ and 1.5. Therefore, it is possible that $\Delta u$ near the edge of the helices

---

5 $\sigma_{vo}'$ at the start of the test was estimated assuming that the total vertical stress is the same as before pile installation.
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(S/D=1.5) is much higher than was measured elsewhere. The Δu(r) measured by the piezometers indicates that the soil which is loaded by the upper helices on the S/D=1.5 piles only extends out to between r/R_{helix} = 1.5 and 2 (about 14 cm beyond the edge of the helices). By comparison, it appears that the soil loaded by the upper helices on the S/D=3 piles extends out to about r/R_{helix} = 3, while the soil loaded by the bottom helix extends out beyond r/R_{helix} = 4. The larger radial extent of Δu generated by the helix plates spaced at 3 diameters indicates a greater extent of soil deformations caused by the loading of the S/D=3 piles. This is consistent with the different failure mechanisms for the different helix spacings (cylindrical shear for S/D=1.5 and individual bearing for S/D=3) which were described previously in Section 2.3.

The excess pore pressures measured at the piezometers located around the grout column during load testing were negligible. However, the closest piezometer was located at about 14 cm from the surface of the column where the shearing surface is believed to be located. The excess pore pressure generated at this distance from the edge of the closely spaced helices, where a cylindrical shearing surface is also expected, were also found to be negligible.

7.2.6 Capacity – Time Trends

A summary of the maximum capacities mobilized by each of the piles is provided in Table 7.4. Only a slight increase in capacity with time was observed between 0.8 days (19 hours) and 42 days (6 weeks) for the S/D=3 piles. A much more significant increase with time was observed for the S/D=1.5 piles.

The failure loads for the various sections of the piles are also listed in Table 7.4 and plotted against the logarithm of time between pile installation and load testing on Figure 7.17. These loads have been corrected to account for the inferred down-drag forces that were present at the time that the strain gauge baselines were taken, as described in Section 7.1. Since corrected SG3 measurements have been used to estimate the load carried by the lead section of piles TP1 and TP2, a range of uncertainty has been established for this data which is indicated in Table 7.4 and on Figure 7.17. This range of uncertainty corresponds to the best estimate ± the maximum probable error (MPE), as described in Appendix G.6. Explanations for the basis of these estimates and potential error are provided in Appendix G.4. Despite the uncertainty inherent in
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the estimation of the loads transferred by the key sections of the piles tested at 6 weeks, the estimates are consistent with the trends observed between 19 hours and 7 days.

### Table 7.4
Summary of Mobilized Soil Resistance at Pile Failure

<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Helix S/D</th>
<th>Recovery Time (days)</th>
<th>Soil Resistance(^{(1)}) (kN) at Failure</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>Pile Total</td>
</tr>
<tr>
<td>TP5</td>
<td>3</td>
<td>0.8</td>
<td>68.5</td>
</tr>
<tr>
<td>TP3</td>
<td>3</td>
<td>7</td>
<td>76</td>
</tr>
<tr>
<td>PP1</td>
<td>3</td>
<td>7</td>
<td>77(^{(6)})</td>
</tr>
<tr>
<td>TP1</td>
<td>3</td>
<td>42</td>
<td>80</td>
</tr>
<tr>
<td>TP6</td>
<td>1.5</td>
<td>0.8</td>
<td>63</td>
</tr>
<tr>
<td>TP4</td>
<td>1.5</td>
<td>7</td>
<td>76</td>
</tr>
<tr>
<td>TP2</td>
<td>1.5</td>
<td>43</td>
<td>89.5</td>
</tr>
</tbody>
</table>

Notes:
1. loads rounded to the nearest 0.5 kN
2. from bottom of casing to SG4 (entire grout column, incl. bottom grout disc)
3. from SG4 to pile tip (i.e. all helices)
4. from SG4 to SG6 (i.e. all helices except bottom plate)
5. below SG6 (i.e. bottom plate and tip of shaft)
6. does not include load transferred along casing
7. load at SG4 position on pile estimated from measurements at SG3 and using SG3-SG4 load estimate (see Appendix G.4 for more details); therefore, a range of estimated loads is given which corresponds to the best estimate ± maximum probable error

The trends with time shown on Figure 7.17 are summarized below.

**Grout Column & Upper Helices at S/D = 1.5**

The mobilized resistance at failure increases with time.

Both the grout column and the more closely spaced helices are expected to cause shear along vertically oriented cylindrical failure surfaces, and therefore, the increase in horizontal effective stress after pile installation should control the increase in shear strength. The observed increase in capacity between 19 hours and 7 days suggests that the radial effective stresses were increasing during that time, as the excess pore pressures were dissipating. This is consistent with

Between 7 days and 6 weeks, the capacity of both the upper helices of the S/D=1.5 piles and the grout column continued to increase. The increase appears to be less significant for the grout column on Figure 7.17 because the ultimate resistance during pile failure has been plotted, rather than the peak resistance (which was recorded at smaller settlements), which showed a larger increase with time. This post-dissipation strength gain is attributed to aging effects due to drained creep and possibly thixotropic hardening. Since significant destructuring has been inferred within the soil along the surface of the grout column and along the outer edges of the helices, 5 weeks of aging is expected to have significant effects on the strength of the new soil structure.

**Upper Helices at S/D = 3 & Bottom Helix**

*The mobilized resistance at failure is relatively constant with time, even during the pore pressure dissipation period between 19 hours and 7 days.*

A (conical) wedge of soil is expected to form below each of the widely spaced helix plates and below the bottom plate as the pile is loaded to failure (as was observed by Rourk, 1961, for a flat-ended model pile). The failure of the pile will therefore require the displacement and deformation of a significant volume of soil located outside the wedge. The pore pressure distribution observed around the upper helices spaced at S/D = 3 and below the bottom helix during load testing (described in Section 7.2.5) supports this type of failure behaviour.

Thus, the ultimate resistance may depend to a certain extent on the shear strength of soil located beyond the edge of the helices, or below the bottom helix. The outer zone of soil (beyond about 6 shaft radii from the pile center) is not believed to have been significantly de-structured by pile installation and so may have retained some degree of overconsolidation. Therefore, it is possible that the increase in effective stress that is expected during the dissipation process may not have caused the yield stress of the relatively intact soil to be exceeded. Without an increase in yield stress, no increase in shear strength would be expected (as was discussed in Section 2.2.6), which would explain the lack of an increase in capacity of these helix plates between 19 hours and 7 days.
The concept of a zone of relatively intact soil resisting failure would also explain why no aging-related increase in capacity was observed between 7 days and 6 weeks for these helix plates. The effects of 5 weeks of aging on the strength of relatively intact, overconsolidated soil will be negligible compared to the thousands of years of aging that has already affected the structure of the soil.

### 7.2.7 Bearing Pressure of Helix Plates Compared to CPT Tip Pressure

Since the failure mechanism below the bottom helix plate and below the widely spaced (S/D=3) helix plates may be similar to that caused by a cone penetrometer, the ultimate bearing pressure below these plates may be comparable to the CPT tip resistance. To examine this possibility, the average bearing pressures induced by the bottom helix plates and by the upper helices at S/D = 3 at failure ($\Delta q_f$) are plotted on Figure 7.18 along with the profiles of $q_T-\sigma_{vo}$ measured in the intact soil at CPT-2,5&7. The magnitude of the measured CPT tip pressures in excess of the total overburden pressure ($q_T-\sigma_{vo}$) are used for comparison because $\Delta q_f$ calculated from the piles does not include the initial total overburden pressure prior to pile installation ($\sigma_{vo}$). Below each data point representing $\Delta q_f$ from the piles, a vertical bar extends to a distance of 2 helix diameters to represent the inferred zone of bearing influence from the helix plates.

From Figure 7.18, it can be seen that $\Delta q_f$ below the bottom helix of 6 out of 7 of the test piles (all except TP4) is very close to the range of $q_T-\sigma_{vo}$ pressures from the cone testing. Thus, it does appear that the corrected tip resistances measured with the cone could be used to obtain a reasonably accurate estimate of the bearing capacity of the bottom helix. However, the $\Delta q_f$ values below the upper helices are significantly less than the range of $q_T-\sigma_{vo}$ measurements from the cone in the intact soil. This is most likely due to the destructuring of the soil that was caused by the installation of the piles.

### 7.2.8 Inferred Shear Strengths Mobilized By Piles

Equations for estimating the capacity of helical piles in clays based on total stress parameters have been published in the literature and were presented in Section 2.3 as Equations 2.3 through 2.6. The research program on model helical piles/anchors that was carried out at the Indian Institute of Technology showed that these equations are reasonably accurate when the undrained strength of the soil is known. These equations were used to back-calculate the average
undrained shear strength mobilized by the various sections of the piles, \((s_u)_{\text{pile}}\), from the known loads transferred to the soil at failure, as is described in Appendix G.5. The estimated values of \((s_u)_{\text{pile}}\) back-calculated from the loads measured between the strain gauges are listed in Table 7.5.

The back-calculated strength below the top helix at 7 days (TP3) was lower than that at 19 hours (TP5), which seems unusual given that the strengths back-calculated from the capacities of the middle helix at 19 hours and 7 days are essentially identical. This may indicate that the bearing capacity factor, \(N_c = 9\), which was used to calculate the undrained shear strength below the helices in bearing, is not appropriate for the failure surface that developed below the top helix of TP3. If an \(N_c\) value of 8 is used instead, an \((s_u)_{\text{pile}}\) of 18 to 19 kPa would be obtained, which is very close to that obtained from TP5.

### Table 7.5
Back-Calculated Shear Strengths Mobilized by Piles

<table>
<thead>
<tr>
<th>Pile No. + (S/D)</th>
<th>Time after Install (days)</th>
<th>Mobilized Undrained Shear Strength (^{(1)}) - ((s_u)_{\text{pile}}) (kPa)</th>
<th>Bottom Helix</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Grout Column (^{(2)})</td>
<td>Upper Helices</td>
</tr>
<tr>
<td></td>
<td></td>
<td>At Peak Strength</td>
<td>At Pile Failure</td>
</tr>
<tr>
<td>TP5 (3)</td>
<td>0.8</td>
<td>3</td>
<td>2.5</td>
</tr>
<tr>
<td>TP3 (3)</td>
<td>7</td>
<td>10</td>
<td>6.5</td>
</tr>
<tr>
<td>PP1 (3)</td>
<td>7</td>
<td>11</td>
<td>8.5</td>
</tr>
<tr>
<td>TP1 (3)</td>
<td>42</td>
<td>11</td>
<td>8.5</td>
</tr>
<tr>
<td>TP6 (1.5)</td>
<td>0.8</td>
<td>4.5</td>
<td>4</td>
</tr>
<tr>
<td>TP4 (1.5)</td>
<td>7</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td>TP2 (1.5)</td>
<td>43</td>
<td>12</td>
<td>8.5</td>
</tr>
</tbody>
</table>

Notes:

1. \(s_u\) given is best estimate, rounded to the nearest 0.5 kPa
2. 2.5 m length along surface of column, from bottom of casing to SG3
3. top plate (S/D=3) or top 2 plates (S/D=1.5)
4. middle plate (S/D=3) or middle 2 plates (S/D=1.5)
5. all helix plates excluding bottom plate

Some of the values of \((s_u)_{\text{pile}}\) from Table 7.5 are also compared to profiles of peak (intact) and remoulded field vane shear strengths, \((s_u)_{\text{FV}}\), on Figure 7.19. The vertical bars above and/or below each data point representing \((s_u)_{\text{pile}}\) indicate the approximate vertical extents of the zones.
through which the average \( (s_u)_{\text{pile}} \) is inferred to apply. The CPTU-interpreted profiles of peak \( (s_u)_{FV} \) have been shown to better present the variability in natural soil strength with depth and at different locations across the site. The frictional resistance measured by the friction sleeve on the cone penetrometer, \( f_s \), is representative of the large-strain shear strength of the soil next to the sleeve and is typically close to the remoulded strength of fine-grained soils. Therefore the \( f_s \) measurements from CPT-7 have been included on Figure 7.19 for comparison with the remoulded strengths measured by the vane. The mobilized \( (s_u)_{\text{pile}} \), relative to the measured \( (s_u)_{\text{peak}} \) and \( (s_u)_{\text{rem}} \) at similar elevations, for the upper helices and for the grout column, are plotted against the logarithm of time on Figure 7.20.

Within the column of soil above the bottom helix, there has clearly been a reduction in shear strength, likely as a result of soil destructuring caused by pile installation, which persists long after the dissipation of excess pore pressure is complete. The degree to which the destructuring has affected the strength that is mobilized by the different elements of the piles appears to vary depending on the load transfer mechanism. From the data in Table 4 and on Figures 7.19 and 7.20, the following observations can be made:

- **Grout Column:** The peak undrained shear strength mobilized along the surface of the grout column at 19 hours is only slightly greater than the remoulded strength of the soil. The available strength then increases with time, but is still significantly less than the peak \( (s_u)_{FV} \) of the soil before pile installation, over the same range of elevations. It should also be noted that the actual strength available at the point of pile failure is generally less than the peak mobilized strengths shown on Figure 7.19, as can be seen from Table 7.5, particularly after the longer recovery periods.

- **Upper Helices (S/D = 1.5):** The undrained shear strength mobilized by the upper four helix plates is less than the peak \( (s_u)_{FV} \), even after the dissipation of \( \Delta u \) is complete. The reduced strength that is available at early times is particularly evident through the zone of soil between the middle and bottom helices on pile TP6 tested at 19 hours after installation. Such strength loss does not appear to be as severe between the top and middle helices of pile TP6. Clearly though, there has been a significant reduction in the strength available for the upper helices of the S/D=1.5 piles, probably due to significant destructuring caused by pile installation. However, unlike for the grout column, the shear strengths mobilized at 19 hours...
do not suggest that the soil near the edge of the helices (through which a cylindrical failure surface must pass) was completely remoulded during pile installation. The shear strengths mobilized by the upper helices of the S/D=1.5 piles increase with time.

- **Upper Helices \((S/D = 3)\):** The undrained shear strength mobilized by the upper two helix plates is close to but consistently less than the peak \((s_u)_{FV}\) at similar elevations, at both 19 hours and 7 days (and at 6 weeks, based on the estimated loads carried by the upper helices together). This indicates that the effects of soil disturbance close to the pile shaft and between the helices has a more limited effect on the capacity mobilized by the more widely spaced plates. This is probably due to the fact that shearing must occur through a significant volume of soil beyond the edge of the helices during a bearing-type failure, and much of this soil may not have experienced any significant structural breakdown during installation. The back-calculated shear strengths from the upper helices of the S/D=3 piles are quite variable and no definitive trend with time is apparent.

- **Bottom Helix:** The undrained shear strengths mobilized by the bottom helix plates are at or slightly greater than the peak \((s_u)_{FV}\) at similar elevations (interpreted from CPTU and FVST measurements). Clearly, the effects of installation disturbance on the strength available below the piles are negligible. The fact that \((s_u)_{pile}\) mobilized below the bottom helix is slightly greater than the average values of \((s_u)_{FV}\) for most of the piles, may be a result of strength gain due to an increase in the vertical effective stress below the bottom of the pile during reconsolidation due to down-drag forces. The development of a net down-drag force on the pile during the reconsolidation process, which appears to have been resisted by the soil below the bottom helix plate, was noted in Section 7.2.1.

It is very apparent from both Figures 7.19 and 7.20 that the peak \(s_u\) measured within the intact soil would not be appropriate for predicting the capacity of the piles, except for below the bottom helix. This is due to the fact that the structure and void ratio of the soil has been changed as a result of the installation and reconsolidation processes.

### 7.2.9 Shear Strength – Effective Stress Ratio at End of Dissipation

For design purposes, obtaining a reasonable estimate of an appropriate value of \(s_u\) at the end of the pore pressure dissipation period is very important. However, from the discussion in Section 7.2.8, it is clear that predicting such a strength value is not a trivial matter once the structure and
void ratio of the soil have been altered significantly from the conditions under which the strength testing was carried out. The ratio of undrained shear strength to vertical effective stress during consolidation, $\sigma'_{vc}$, may be the most rational way to predict the reconsolidated $s_u$ at any depth.

For this study, average values of $\sigma'_{vc}$ at the end of pore pressure dissipation, over the range of depths applicable to the values of $(s_u)_{\text{pile}}$ in Table 7.5, were calculated based on the estimated overburden pressures, $\sigma_{vo}$, and the measured piezometric conditions. For the bottom helix, an additional 10 kPa was added to $\sigma_{vo}$ to account for the additional end-bearing which develops during the consolidation process to resist the down-drag forces. The calculated $(s_u)_{\text{pile}}/\sigma'_{vc}$ ratios from the tests carried out at 7 days (TP3&4) are provided in Table 7.6 and are plotted on Figure 7.21 along with the CPTU-interpreted profiles of $(s_u)_{\text{FV}}/\sigma'_{vo}$. Each data point representing $(s_u)_{\text{pile}}/\sigma'_{vc}$ corresponds to the average elevation within the inferred zone through which the average $(s_u)_{\text{pile}}$ applies, and the vertical bars indicate the approximate extents of those zones. The values of $(s_u)_{\text{FV}}/\sigma'_{vo}$ are representative of the intact strength of the soil which is a function of the natural micro-structure, stress history and aging effects.

The values of $(s_u)_{\text{pile}}/\sigma'_{vc}$ mobilized along the surface of the grout column at failure are lower than those mobilized by the helices and are much lower than the intact $(s_u)_{\text{FV}}/\sigma'_{vo}$ at comparable depths. The values of $(s_u)_{\text{pile}}/\sigma'_{vc}$ mobilized by the upper helices of the S/D=1.5 pile are also significantly less than the intact $(s_u)_{\text{FV}}/\sigma'_{vo}$ values. The values of $(s_u)_{\text{pile}}/\sigma'_{vc}$ mobilized by the upper helices of the S/D=3 pile are closer to but still lower than the intact values. The values of $(s_u)_{\text{pile}}/\sigma'_{vc}$ mobilized by the bottom of the piles are very close to the intact values.

The above observations are consistent with the observations from Figure 7.19. The differences between the intact and mobilized ratios are indicative of the degree of destructuring that has occurred within the regions of soil sheared by the different pile elements. To quantify this, an “index of soil destructuring”, $I_D$, is proposed, where:

$$I_D = \frac{[(s_u/\sigma'_{vo})_{\text{intact}} - (s_u)_{\text{pile}}/\sigma'_{vc}] / [(s_u/\sigma'_{vo})_{\text{intact}} - (s_u/\sigma'_{vc})_{\text{destructured}}]}$$

and $(s_u/\sigma'_{vo})_{\text{intact}}$ and $(s_u/\sigma'_{vc})_{\text{destructured}}$ are the undrained strength ratios of the soil in its natural intact state and after reconsolidating from a totally destructured state, respectively.
The surface of the grout column is located within a zone of soil that is inferred to have been almost completely remoulded during pile installation, and the mobilized shear strengths at 19 hours were found to be very close to the remoulded strength of the soil. The range of $s_u/\sigma'_{vc}$ values for the grout column (0.18 to 0.22) is consistent with the $s_u/\sigma'_{vo}$ suggested by Kukan (1998) for normally consolidated clays and silts in the Lower Fraser Valley, and with the value interpreted from CPTU measurements near the bottom of the silt and clay deposit at the Colebrook site (which is believed to be essentially normally consolidated). Thus, an average $(s_u/\sigma'_{vc})_{destructured}$ of 0.20 was assumed to be representative of the completely destructured soil, which will no longer have memory of past stress history or aging effects, and is also plotted on Figure 7.21 for reference.

The intact values from the CPTU-interpreted profiles of $(s_u)_{FV}/\sigma'_{vo}$ were used along with $(s_u/\sigma'_{vc})_{destructured}$ to calculate values of $I_D$ for the various sections of the piles tested at the end of the dissipation process at 7 days. These indices are listed in Table 7.6.

<table>
<thead>
<tr>
<th>Section</th>
<th>Pile</th>
<th>$s_u/\sigma'_{vc}$</th>
<th>$I_D$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Grout Column (at failure)</td>
<td>TP3</td>
<td>0.18</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TP4</td>
<td>0.22</td>
<td>100%</td>
</tr>
<tr>
<td>Top Helices (S/D = 1.5)</td>
<td>TP4</td>
<td>0.27</td>
<td>75%</td>
</tr>
<tr>
<td>Middle Helices (S/D = 1.5)</td>
<td>TP4</td>
<td>0.26</td>
<td>70%</td>
</tr>
<tr>
<td>Top Helix (S/D = 3)</td>
<td>TP3 (N_c=9)</td>
<td>0.30</td>
<td>65%</td>
</tr>
<tr>
<td></td>
<td>TP3 (N_c=8)</td>
<td>0.33</td>
<td>50%</td>
</tr>
<tr>
<td>Middle Helix (S/D = 3)</td>
<td>TP3</td>
<td>0.34</td>
<td>30%</td>
</tr>
<tr>
<td>Bottom Helix (and tip of shaft)</td>
<td>TP3</td>
<td>0.40</td>
<td></td>
</tr>
<tr>
<td></td>
<td>TP4</td>
<td>0.32</td>
<td>0</td>
</tr>
<tr>
<td></td>
<td>PP1</td>
<td>0.36</td>
<td></td>
</tr>
</tbody>
</table>

The calculated $I_D$ indices are generally consistent with the magnitude and extents of soil disturbance that were inferred from pile installation, and with the understanding of the zones of soil loaded by the various pile sections. From Table 7.6, it can be seen that the $s_u/\sigma'_{vc}$ ratio for the top 2 helices of the S/D=1.5 pile is nearly identical to that of the middle 2 helices on the same pile. Similarly, if an $N_c$ factor of 8 is used instead of 9 for the top helix of pile TP3 (as discussed...
previously in Section 7.2.7), the $s_u/\sigma'_{vc}$ ratios for the top and middle helices of the S/D=3 pile are almost identical. This is further evidence that the shear strength mobilized along the piles is highly dependent on $\sigma'_c$ (and therefore on depth), which is consistent with the $s_u/\sigma'$ relations during reconsolidation which were discussed in Section 2.2.6.

It appears that the degree of destructuring for the top helix plate(s) is higher than for the middle helix plate(s), even when the $s_u/\sigma'_{vc}$ ratios are almost identical. This is due to the fact that the soil at the Colebrook site has an intact $s_u/\sigma'_{vo}$ ratio that is higher near the top helix plate(s) than near the middle helix plate(s). In a soil where the intact $s_u/\sigma'_{vo}$ ratio is constant with depth, the increase in $I_D$ with increasing numbers of helix plates may be less than that observed in this study.

### 7.2.10 Increase in Shear Strength after Complete Pore Pressure Dissipation

It has already been noted that increases in the capacity of the grout column and the S/D=1.5 helices were observed after the dissipation of excess pore pressure was complete. In these cases, the failure surface is inferred to have passed through soil that had been mostly to completely destructured by the pile installation process. Consequently, aging of the new soil fabric is believed to be responsible for the increased capacity. The average undrained shear strengths back-calculated from the measured capacities of the grout column and the S/D=1.5 helices at 7 days and 6 weeks are listed in Table 7.7, along with the relative increase in strength.

<table>
<thead>
<tr>
<th>Section of Pile</th>
<th>Average $s_u$ (kPa)*</th>
<th>Relative Strength Increase</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At 7 days</td>
<td>At 6 weeks</td>
</tr>
<tr>
<td>Helices (S/D=1.5)</td>
<td>16</td>
<td>18.5</td>
</tr>
<tr>
<td>Grout Column (Casing – SG3):</td>
<td></td>
<td></td>
</tr>
<tr>
<td>At Peak Resistance</td>
<td>9</td>
<td>11.5</td>
</tr>
<tr>
<td>At Pile Failure</td>
<td>7.5</td>
<td>8.5</td>
</tr>
</tbody>
</table>

*Note: Strengths rounded off to the nearest 0.5 kPa.

For a soil that has been destructured under undrained conditions, the void ratio ($e$) will decrease during reconsolidation as the effective stress ($\sigma'$) increases, according to the slope of the
destructured compression line, as is illustrated conceptually on Figure 7.22. Once excess pore pressure dissipation is complete (end of primary consolidation – EOP), the effective stress will be at $\sigma'_c$ and the soil will have a void ratio of $e_{EOP}$. If the soil was normally consolidated before pile installation, or if the soil was completely destructured during pile installation, the yield stress at the end of reconsolidation, $\sigma'_{y}(EOP)$, will be equivalent to $\sigma'_c$.

During aging, the $e$ decreases with time at constant $\sigma'$ due to the secondary compression of the new soil fabric, as is illustrated on Figure 7.22. The decrease in void ratio due to secondary compression, $\Delta e_s(t)$, is typically calculated using the following expression:

$$\Delta e_s(t) = C_\alpha \cdot \log\left(\frac{t}{t_{EOP}}\right)$$  \hspace{1cm} (7.2)

where: $C_\alpha = \text{coefficient of secondary compression, and}$

$$t, t_{EOP} = \text{time since pile installation and time to end of primary consolidation (i.e. full pore pressure dissipation), respectively.}$$

Using $C_\alpha = 0.015$ for the Colebrook site, which was back-calculated from field settlement observations by Crawford & deBoer (1987), and $t_{EOP} = 1$ week for the helical test piles, $\Delta e_s \approx 0.0117$ at 6 weeks.

As the void ratio decreases during aging, the yield stress will tend to increase along the governing $e$-$\log\sigma'$ compression line of the soil (Bjerrum (1967) described this effect as an apparent increase in preconsolidation pressure). This is illustrated on Figure 7.22 for a soil that has been completely destructured, such that $\sigma'_y(EOP) = \sigma'_c$. Accordingly, the relative increase in yield stress depends on the slope of the $e$-$\log\sigma'$ compression line (which is typically referred to as the compression index, $C_c$) according to the following expression:

$$\log\left[\frac{\sigma'_y(t)}{\sigma'_y(EOP)}\right] = \frac{\Delta e_s(t)}{C_c}$$  \hspace{1cm} (7.3)

where $\sigma'_y(EOP)$ and $\sigma'_y(t)$ are the yield stresses at the end of the pore pressure dissipation period and after aging to time $t$, respectively.
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Since the ratio of \( s_u/\sigma'_y \) is essentially constant for a given soil, the relative increase in shear strength should be equivalent to the relative increase in yield stress and can therefore be expressed as follows:

\[
\frac{s_u(t)}{s_u(EOP)} = 10^a
\]

where \( a = (\Delta e_s/C_c) \) (7.4)

where \( s_u(EOP) \) and \( s_u(t) \) are the undrained shear strengths at the end of the pore pressure dissipation period and after aging to time \( t \), respectively.

The \( C_c \) of the Colebrook clayey silt in a partially to totally destructured state is not known at this time. The range of \( C_c \) values reported by Crawford & deBoer (1987) on samples of intact soil from the upper 16 m in the Colebrook area, which appear to have been affected to some extent by sample disturbance, ranged from 0.25 to 0.46. The \( C_c \) for a completely destructured soil may be closer to the intrinsic compression index, \( C_c^* \), as defined by Burland (1990). Burland (1990) showed that \( C_c^* \) tends to increase with increasing liquid limit, and for soils with liquid limits close to 40 percent, which is the approximate average for the Colebrook test site, \( C_c^* \) tends to vary around 0.20 to 0.25.

Using a range of \( C_c = 0.20 \) to 0.25, and \( \Delta e_s \approx 0.0117 \), \( (s_u)_A/(s_u)_{EOP} \) is predicted to range from 1.11 to 1.14; i.e. an increase in shear strength due to aging of 11% to 14%. This is close to but slightly less than the observed increases in back-calculated \( s_u \) between 1 week and 6 weeks, which may also have included additional increases due to thixotropic hardening. Using the same parameters \( (C_\alpha = 0.015, \ C_c = 0.20) \), an increase in strength of 35% would be predicted after 1 year. This would bring the average \( s_u/\sigma'_v \) ratio for the S/D=1.5 helices up to about 0.36 which is close to the intact \( s_u/\sigma'_v \) ratio of the soil between –7 and –10 m elevation.

### 7.3 Summary and Conclusions

From the load testing program carried out during this study, the following key observations and conclusions were made about the way that these helical piles mobilize capacity:

1) The radial distribution of excess pore pressure, \( \Delta u(r) \), generated around the upper helices of the S/D=1.5 piles indicates that soil deformations at failure only extend out to a radial
distance of about 1.8 helix radii from the center of the pile. This provides additional
evidence that shearing is confined to a cylindrical failure surface which develops along the
outer edges of the more closely spaced helices. However, \( \Delta u(r) \) also indicates that the
volume of soil deformed by the bottom helix and by the upper helix plates spaced at \( S/D = 3 \)
extends out from the pile shaft to much greater radial distances \( (r/R_{hx} \approx 4 \) and 3, respectively). This is consistent with the pattern of bearing-type failure surfaces.

2) The peak resistance mobilized by the surface of the grout column occurs at settlements
between about 2 and 10 mm, as compared to settlements of about 40 to 60 mm required to
reach the maximum capacity of the lead section (11 to 17 percent of the helix diameter).
Thus, while the grout column appears to behave like a conventional friction pile, the load-
settlement behaviour of the lead section is more like that observed by end-bearing piles, even
for the helices spaced at \( S/D \) of 1.5.

3) Pile settlements of only 4 to 5 mm were observed at load levels corresponding to 50% of the
total pile capacity (i.e. corresponding to a factor of safety of 2) at 7 days after installation (at
the end of the pore pressure dissipation period). At this stage, the soil resistance being
mobilized by the pile shaft above the helices (including the grout column) accounted for
about one third of the total pile capacity.

4) The reconsolidated soil along the surface of the grout column behaves in a brittle manner
when sheared at 7 days and 6 weeks. The additional settlement that occurs between
mobilization of peak resistance of the grout column and the point of pile failure causes a
reduction in resistance along the surface of the column of about 20% from the peak
resistance. At the ultimate capacity of the piles tested at 19 hours and 7 days after
installation, the proportion of the total resistance that was being mobilized by the pile shaft
above the helices had been reduced to 10% and 20%, respectively.

5) At 19 hours, the capacity mobilized by the bottom helix plate and the tip of the pile shaft
accounted for about 40-45% of the total capacity of the relatively short test piles. Under
working load conditions \( (Q = 0.5Q_{ult}) \), the proportion of the resistance mobilized by the
bottom plate and tip at 19 hours was even higher at about 45-50% of the total pile load.
6) From this study, it appears that the capacity of the bottom helix (including the tip of the pile shaft) can be reasonably predicted using either:

- The net tip resistance \( (q_T - \sigma_v) \) measured during CPT testing in the intact soil before pile installation, or
- Bearing capacity theory for deep foundations, using the intact undrained shear strength of the soil and a bearing capacity factor, \( N_c \), of 9.

7) The undrained shear strength mobilized by the different structural elements of the pile depends on the zone of soil loaded by each structural element. This is believed to be due to the spatial variations in strength reduction caused by the strains generated during pile installation, which were discussed in Chapter 6. The effects of structural breakdown due to pile installation on the strength mobilized by the piles was quantified using an index of soil destructuring, \( I_D \). The effects of installation disturbance on the resistance mobilized by the different sections of the test piles are summarized below:

- The strain-softening and destructuring is greatest near the pile shaft where the grout column mobilizes resistance. This explains why the strengths mobilized by the grout column at 19 hours are close to the remoulded strength of the soil. The soil along the grout column is inferred to be essentially completely destructured based on the undrained strength ratios at the end of the pore pressure dissipation period.
- Significant strain-softening is also believed to occur within the soil near the edge of the helices, although the remaining strength may still be significantly greater than the remoulded strength. The destructuring within this zone is inferred to be 70 to 75 percent complete.
- The soil beyond about 10 cm from the edge of the helices \( (r/R_{shaft} > 6, r/R_{hx} > 1.6) \) is not believed to have been significantly destructured by the pile installation process. The average shear strength mobilized by the upper helices of the S/D=3 piles at failure may be some average of the actual shear strength distribution within the total volume of soil failed during the load test. This probably includes zones of totally and partially de-structured soil between the helices as well as the relatively intact soil beyond the helices. The average degree of destructuring throughout these zones was about 30 to 50 percent.
• The volume of soil below the bottom helix plate that is likely to have been subjected to strain softening during pile installation is inferred to be very limited relative to the volume of soil that is loaded by the bottom of the pile at failure. This explains the relatively high shear strengths mobilized by the bottom helix, which were at least as high as the peak shear strength of the intact soil measured by the field vane before pile installation, even at only 19 hours after installation. Below the tip of the pile shaft, the structure of the soil is inferred to be essentially intact based on the observed pore pressure behaviour during plunging failure.

8) The sections of the pile that are mobilizing most or all of their resistance within de-structured soil (grout column and upper helices of S/D=1.5 piles), were observed to show trends with time of increasing capacity and decreasing settlement required to mobilize full capacity. This is believed to occur as a result of the recovery of soil strength and stiffness as the new (partially to totally de-structured) soil fabric consolidates. The increases in soil strength and stiffness along vertically oriented failure planes appear to occur most quickly during the pore pressure dissipation process (primary consolidation) when the radial effective stress in the soil is inferred to be increasing. However, at the end of the dissipation process, only about 60% of the peak \( (s_u)_{FV} \) of the soil was being mobilized by the upper helices (S/D = 1.5) and less than 50% of full strength recovery was observed along the surface of the grout column.

9) The increases in soil strength and stiffness within the de-structured zones appear to continue, although at a slower rate, after the end of the dissipation process. These changes are believed to be a result of aging processes, which typically result in further reductions in void ratio (secondary consolidation) and thixotropic hardening of the new soil fabric. After 5 weeks of aging, the strength mobilized by the upper helices (S/D=1.5) and the ultimate strength along the grout column at pile failure was, on average, about 15% higher than at the end of dissipation. It appears that this relative increase in strength could be reasonably predicted using Equations 7.4 and 7.5, if \( C_\alpha \) and \( C_c \) of the destructured soil are measured.

10) No significant changes in mobilized capacity with time were observed for the bottom helix and for the more widely spaced upper helices (S/D = 3). This is believed to be a result of the bearing-type failure mechanism, which mobilizes a significant proportion of its resistance within soil that is inferred to be relatively intact structurally.
11) At 6 weeks after pile installation, the average undrained shear strength mobilized by the upper helices of the S/D=1.5 pile is inferred to be still slightly less than that mobilized by the S/D=3 pile, but is probably still increasing. At this point, the capacity of the 4 upper helices of the S/D=1.5 pile was about 25-30% higher than that of the 2 upper helices of the S/D=3 pile. Thus, the findings of this study support the findings of the IIT model pile research (published by Narasimha Rao et al.) that helical piles with an S/D ratio of 1.5 will mobilize larger capacities than piles with an equivalent length of helices in soil with equivalent strength but with larger helix spacings.

12) The $s_u/\sigma'_{vc}$ ratio of the disturbed and reconsolidated soil appears to be the most rational method of predicting the undrained strength mobilized by the upper helices and the grout column. An estimate of the $s_u/\sigma'_v$ ratio of the normally consolidated soil can typically be made based on conventional site characterization and published values of $(s_u/\sigma'_v)_{NC}$ for similar soils. It appears that the $(s_u/\sigma'_v)_{NC}$ ratio should provide reasonable estimates of the strength mobilized by the grout column, and somewhat conservative estimates for cylindrical failures along the outer edge of closely spaced helices. For estimating the average $s_u$ mobilized by the more widely spaced helices, the $s_u/\sigma'_{vc}$ ratio appears to be closer to the $s_u/\sigma'_{vo}$ ratio of the intact soil than to the $(s_u/\sigma'_v)_{NC}$ ratio. Establishing suitable values of the “index of destructuring” for the different bearing elements would enable a better estimate of $s_u$ to be made using Equation 7.1.
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Figure 7.1: Load Test Setup at Top of Test Piles

Figure 7.2: Reaction Frame for Load Tests
Figure 7.3: Locations of Strain Gauges and Piezo-Ports on Test Piles

Figure 7.4: Development of Compressive Load in Pile Shaft during Reconsolidation Process
Figure 7.5: Distribution of Load Within Shaft of Test Pile TP6 During Load Test

*NOTE: Load excludes self-weight of pile
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Figure 7.6: Curves of Total Pile Load vs. Pile Settlement

Figure 7.7: Normalized Curves of Total Pile Load vs. Pile Settlement
Figure 7.8: Load – Settlement Curves for Grout Column
Figure 7.9: Load – Settlement Curves for Lead Section (below SG4)
Figure 7.10: Load – Settlement Curves for Upper Helices (SG4 – SG6)

Figure 7.11: Load – Settlement Curves for Bottom Helix (below SG6)
Figure 7.12: Incremental Settlement – Time Curves for Individual Load Intervals during Load Test on Pile TP3
Figure 7.13: Ratio of Excess Pore Pressure to Induced Vertical Stress below Bottom Helix during Load Testing

Figure 7.14: Pore Pressure Ratio from Pile Load Testing Compared to CPTU $B_q$ Ratio
Figure 7.15: Excess Pore Pressure Generation Around Helical Piles During Load Testing
Figure 7.16: Radial Distributions of Excess Pore Pressures Generated Around Helices at Point of Pile Failure
Figure 7.17: Changes in Mobilized Resistance with Time for Different Pile Sections
Figure 7.18: Bearing Pressure Induced by Helix Plates at Failure Compared to Profiles of CPT Tip Resistance
Figure 7.19: Undrained Shear Strengths Mobilized by Different Pile Sections Compared to Undrained Strengths Measured by In-Situ Tests
Figure 7.20: Changes in Undrained Shear Strength with Time Along Different Pile Sections
Figure 7.21: Undrained Strength Ratios for Different Pile Sections Compared to Undrained Strength Ratio of Intact Soil Interpreted from In-Situ Tests
Figure 7.22: Changes in Yield Stress due to Aging
8.0 ESTIMATION OF SOIL PARAMETERS FOR PREDICTING AXIAL CAPACITY OF HELICAL PILES IN FINE-GRAINED SOILS

The model pile research carried out at the Indian Institute of Technology (IIT) showed that the capacity of helical piles installed in fine-grained soils can be predicted with a good degree of accuracy using the undrained shear strength, $s_u$, of the soil, provided that an appropriate value for this parameter is known. The IIT results (discussed in Section 2.3) suggest that the best accuracy is achieved using the cylindrical shear method to predict the capacity of helix plates at $S/D \leq 1.5$ ($Q_{cyl}$), which is based on the following equation:

$$Q_{cyl} = A_{cyl} \cdot s_u \quad \text{(from Eq. 2.3)}$$

They also found that the capacity of helices spaced at $S/D > 2$ ($Q_b$) could be predicted with a reasonable degree of accuracy using the individual plate bearing method, which is based on the following equation:

$$Q_b = \Sigma (A_{hx} \cdot N_c \cdot s_u) \quad \text{(from Eq. 2.6)}$$

The IIT research was based on piles installed in beds of remoulded clay, and so the residual effects of installation disturbance are expected to have been negligible. Therefore, the shear strength mobilized by the piles was likely very similar to that measured within the soil directly. Conversely, Hoyt & Clemence (1989) showed a great deal of variation in accuracy when the methods were applied to actual field cases without the use of good engineering judgement in the selection of appropriate values for the soil parameters.

The results of this study clearly show that installation disturbance can have a significant impact on the shear strength available to piles in natural soil deposits. Thus, based on the available data from this study and the proposed interpretation of that data, a method of estimating the $s_u$ of normally consolidated to lightly overconsolidated fine-grained soils at various times after pile installation is tentatively suggested below.

The framework for this proposed method is as follows:

- Estimate the duration of the primary reconsolidation process (i.e. the time required for complete dissipation of the excess pore pressures generated during pile installation).
Chapter 8 – Estimation of Soil Parameters for Axial Capacity Predictions in Fine-Grained Soils

- Estimate the appropriate magnitude of $s_u$ at the end of the primary reconsolidation process for the different pile segments.

- For the zones of soil that have been subjected to a significant reduction in strength due to installation disturbance, estimate the aging-induced increase in $s_u$ with time after the pore pressure dissipation period.

These estimates of $s_u$ would then be used as input to the existing cylindrical shear and/or individual plate bearing capacity prediction methods, depending on the specific geometry of the helical pile. An alternative method of estimating the capacity of the bottom helix plate, $Q_{\text{bottom } \text{hx}}$, would be to use the net tip resistance measured during the CPT test, $q_T - \sigma_{vo}$, according to the following simple equation:

$$Q_{\text{bottom } \text{hx}} = A_{\text{hx}} \cdot (q_T - \sigma_{vo}) \quad (8.1)$$

The net tip resistance measured by the CPT in intact soil appears to be applicable for estimating $Q_{\text{bottom } \text{hx}}$ since the amount of disturbance induced with the volume of soil loaded by the bottom helix appears to be limited (as was discussed in Section 7.2.7).

The predictive success of the proposed method will depend on the quality and applicability of the soil data obtained during the site characterization phase of a project. It is very important to realize that the proposed approach described below is based on the interpretation of field data from a single site, and needs verification by application to other well-characterized sites with helical pile load test data. The Colebrook soils are geologically young, and lightly overconsolidated. Therefore, the experience gained at this site, and the proposed methodology described below, may not be applicable to moderately to heavily overconsolidated clays.

### 8.1 Pore Pressure Dissipation Period

In order to obtain a reliable estimate of the time required for the dissipation of excess pore pressure around a helical pile, the following are necessary:

- the distribution of excess pore pressure surrounding the pile immediately after stopping pile installation,
appropriate value(s) of the coefficient of consolidation within the soil surrounding the pile, and

• a reliable solution of the dissipation process that is applicable to the specific pile geometry, or, alternatively, is versatile enough to be used for piles with different shaft and helix plate geometries.

Presently, no published solutions exist for estimating the dissipation times for helical piles. Additional research is required to develop such solutions, as have been done for piezocone probes and conventional cylindrical piles. This would require carrying out detailed numerical consolidation analyses using appropriate soil parameters and boundary conditions, and using an initial distribution of excess pore pressure that is appropriate for the helical piles.

Until dissipation solutions are available that are specific to helical pile geometries, the existing theoretical solutions for cylindrical consolidation may be the best method of predicting the dissipation around helical piles. In this study, the normalized CPTU dissipation solutions by Teh & Houlsby (1991) (or LeVadoux & Baligh, 1986) for locations along the cone probe away from the cone tip and shoulder (e.g. U3 above the friction sleeve) were found to provide $T_{90}$ times that were roughly comparable to the normalized pile dissipation times. The measured times for dissipation along the pile shaft were normalized by the radius of the pile shaft, $R_{\text{shaft}}$, and by the average $c_h$ that was estimated by comparing measured CPT(U2) dissipation data to published solutions for the U2 position by Teh & Houlsby (1991). A reliable estimate of the rigidity index, $I_r$, is also required to use the Teh & Houlsby solutions, as described in Appendix B. The methods used in this study to obtain applicable estimates of $I_r$ and $c_h$ were described in Chapter 4 (Sections 4.3.2 and 4.5.4, respectively). These methods were by no means simple, and reliable estimates were only obtained after a careful and thorough examination of high-quality in-situ test results.

As an alternative to deriving realistic estimates of $I_r$ and $c_h$, actual site-specific dissipation data can be obtained from dissipation tests carried out during CPTU testing. However, it was observed in this study that the dissipation recorded at the standard U2 position behind the shoulder of the cone (normalized by the cone radius) was faster than that observed at any of the positions on the pile shaft (normalized by the radius of the pile shaft). The normalized dissipation recorded at the U3 filter position above the friction sleeve (after correcting for initial
redistribution effects) was found to compare closely with the dissipation observed below the bottom helix plate. It was also observed that the dissipation curves between the helix plates and below the bottom plate tended to converge at the end of the dissipation process. Thus, it appears to be reasonable to use the total duration of the CPT(U3 or higher) dissipation process to obtain an approximate estimate of the duration of the dissipation process around the helical piles, after scaling up the times to account for the larger pile shaft diameter, according to the following equation:

\[ t_{pile} = t_{CPT} \cdot \left( \frac{R_{pile}}{R_{CPT}} \right)^2 \]  

(from Eq. 6.4)

From a pile capacity perspective, thetotal durationof the dissipation process is probably the most important information for the design engineer.

Unfortunately, conventional CPTU equipment does not include pore pressure elements located above the friction sleeve, which makes it unlikely that CPTU data acquired from commercial testing could be used directly to estimate dissipation times around helical piles. The only way to use the U2 dissipation data directly would be to substitute a value for \( R_{pile} \) in Equation 6.4 that is larger than the pile shaft radius (but less than the helix plate radius, based on the results from this study).

Also, it may be uneconomical to carry out CPTU dissipation tests to completion of the dissipation process. Such tests are typically terminated after about 50% dissipation. Therefore, the measured dissipation data would likely have to be extrapolated to at least 90% dissipation using published solutions. However, the shape of the theoretical dissipation curve does not always match the measured dissipation data, which makes the theoretical curve less reliable for extrapolating the measured data to larger times. In this study, a curve matching procedure was used for such extrapolations, which is described in Appendix E.4.

Fortunately, the total duration of the dissipation process for individual helical piles should be relatively brief when compared to typical construction schedules, and therefore may not be a critical factor for a project. It should be noted that the duration of the dissipation process may be significantly delayed for pile groups in which the zones of elevated pore pressures from adjacent piles overlap.
8.2 **Undrained Shear Strength at End of Dissipation Period**

The shear strength of the reconsolidated soil along the shear surface around a pile loaded to failure will mainly depend on:

- the degree of destructuring caused during installation, and
- the mean effective stress during consolidation.

To account for the stress dependency of the undrained shear strength, $s_u$, the estimation method proposed here is based on the undrained strength ratio, $s_u/\sigma'_v$. To account for varying degrees of destructuring caused by pile installation, the destructuring index, $I_D$, from Equation 7.1 is employed. Accordingly, the following expression for estimating the $s_u$ of the soil at the end of the pore pressure dissipation period was derived from Equation 7.1:

$$
\frac{s_u}{\sigma'_v} = \frac{(s_u/\sigma'_v)_D}{(1 - I_D)[(s_u/\sigma'_v)_o - (s_u/\sigma'_v)_D]}
$$

where:

- $(s_u/\sigma'_v)_D =$ the undrained strength ratio of the soil in a completely destructured state (i.e. after reconsolidation from a completely remoulded state); discussed further in Section 8.2.2,
- $(s_u/\sigma'_v)_o =$ the undrained strength ratio of the intact soil (i.e. as would be determined from in-situ measurements of $s_u$, as described in Section 8.2.3),
- $I_D =$ index of soil destructuring, where $0 \leq I_D \leq 1$ (0 for intact structure, 1 for completely destructured); discussed further in Section 8.2.4, and
- $\sigma'_v =$ the vertical effective stress at the end of the pore pressure dissipation period.

In “young” normally consolidated soils, $(s_u/\sigma'_v)_o \approx (s_u/\sigma'_v)_{NC} \approx (s_u/\sigma'_v)_D$ and Equation 8.2 reduces to the simple expression:

$$
\frac{s_u}{\sigma'_v} = (s_u/\sigma'_v)_{NC} \cdot \sigma'_v
$$

Equation 8.2 is a more rational method of calculating a post-installation undrained shear strength than Meyerhof’s purely empirical $\alpha$ correction factor. However, Equation 8.2 cannot account for changes in horizontal effective stress due to pile installation, which could alter the applicable $s_u/\sigma'_v$ ratios, but which are not usually known and are difficult to estimate.

Both Equations 8.2 and 8.3 provide a means of accounting for increased vertical stresses under the helix plates due to negative skin friction or sustained structural loads.
8.2.1 Shear Mode Effects on $s_u$

The peak undrained shear strength that is measured in a given soil will depend on the mode of shearing induced within the soil and on the 3 dimensional stress state in the soil during shearing. As a result, significantly different magnitudes of $s_u$ can be measured by different strength tests within the same soil under the same stress conditions during consolidation. Triaxial compression and plane strain compression tests tend to give the largest magnitudes of $s_u$, while triaxial extension and plane strain extension tests tend to give the lowest values, and direct simple shear tests tend to give intermediate values (e.g. Aubeny et al. 2000).

The helical pile capacity prediction equations (Equations 2.3 and 2.6) were verified in the IIT research program using undrained shear strengths that were measured by “in-situ” vane shear tests carried out on remoulded clay contained within test tanks. In the present study, the undrained shear strengths back-calculated from the load test results using the same equations were compared to $s_u$ and $s_u/\sigma'_v$ profiles which are based on field vane shear test (FVST) magnitudes. Therefore, caution should be used if pile capacity predictions are to be made using values of $s_u$ measured in consolidated undrained triaxial compression tests since these strengths are usually higher than FVST strengths, particularly for fine-grained soils with low plasticity. Chandler (1988) derived the following expression relating FVST strengths, $(s_u)_{FV}$, to $K_o$-consolidated undrained triaxial compression ($C_{K_o}UTC$) test strengths, $(s_u)_{C_{K_o}UTC}$, which clearly demonstrates this trend:

$$\frac{(s_u)_{FV}}{(s_u)_{C_{K_o}UTC}} = 0.55 + 0.008 \cdot I_p$$

where $I_p$ = plasticity index (in %). Thus, for low plasticity soils ($I_p$ between 10 and 20 percent), $(s_u)_{C_{K_o}UTC}$ tends to be about 50% higher than the corresponding $(s_u)_{FV}$.

8.2.2 Destructured Undrained Strength Ratio

The destructured undrained strength ratio, $(s_u/\sigma'_v)_D$, corresponds to a soil that has had all memory of previous stress history and aging effects erased through the destructuring process. Therefore, $(s_u/\sigma'_v)_D$ should be similar to the undrained strength ratio of the same soil in a normally consolidated state, which is typically denoted as $(s_u/\sigma'_v)_{NC}$. 

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Ideally, the \((s_u/\sigma'_v)_{NC}\) ratio should be measured by laboratory shear strength tests (e.g. direct simple shear tests) on reconsolidated samples of completely remoulded soil. Alternatively, \((s_u/\sigma'_v)_{NC}\) can be determined using the SHANSEP procedure (Ladd & Foott, 1974) of consolidating a sample of intact soil to a stress in excess of its natural yield stress. However, the measured \((s_u/\sigma'_v)_{NC}\) can be greater than the true \((s_u/\sigma'_v)_{D}\) ratio of the soil if the consolidation stress does not exceed the yield stress by an amount that is sufficient to completely destructure the soil.

Typical values of \((s_u/\sigma'_v)_{NC}\), which are valid for \((s_u)_{FV}\) are also available in the literature. The following empirical expression was originally given by Skempton (1957):

\[
(s_u/\sigma'_v)_{NC} = 0.11 + 0.0037 \cdot I_p
\]

where \(I_p\) = plasticity index (in %). Bjerrum (1973) proposed a similar relation for \((s_u)_{FV}\) of normally consolidated soils where \((s_u/\sigma'_v)_{NC}\) vs. \(I_p\) is more logarithmic than linear, such that \((s_u/\sigma'_v)_{NC}\) approaches a maximum of about 0.33 for \(I_p\) between 80% and 100% (as compared to 0.41 to 0.48 suggested by the Skempton relation). Bjerrum (1973) also observed that the \((s_u)_{FV}\) measured at relatively high strain rates tended to be higher than those back-calculated from observed embankment failures, such that a FVST strength reduction factor, \(\mu\), was suggested to account for strain rate effects. This factor was shown by both Bjerrum (1973) and Azzouz et al. (1983) to become increasingly less than unity for increasing \(I_p\) (i.e. the rate-dependent increase in strength is more significant for higher plasticity soils). Mesri (1975) used Bjerrum’s \(\mu\) vs. \(I_p\) relation to correct \((s_u/\sigma'_v)_{NC}\) vs. \(I_p\) relation and found that the resulting \((\mu \cdot s_u/\sigma'_v)_{NC}\) factor was constant at about 0.22±0.03, regardless of \(I_p\) (Chandler 1988). A review of various published \((s_u/\sigma'_v)_{NC}\) vs. \(I_p\) relations by Kokan (1998) indicates that for fine-grained soils with low plasticity (\(I_p\) between 10 and 30 percent), \((s_u/\sigma'_v)_{NC}\) typically ranges between 0.19 and 0.24.

**8.2.3 Methods of Determining \((s_u/\sigma'_v)_{NC}\) Profile.**

The \(s_u\) of the intact soil can be measured using either laboratory tests on recovered “undisturbed” samples or using in-situ tests. Unconfined triaxial compression (UU) tests are relatively fast and inexpensive but the measured strengths tend to be lower than in-situ strengths and can be prone to a large degree of variability due to sample disturbance, particularly for structured soils.
Consolidated strength tests can give reasonably good measures of $s_u$, but are time-consuming and expensive. In structured soils such as those at Colebrook, soil sampling can cause destructuring of fragile inter-particle bonds, which can result in the $s_u$ measured after reconsolidating to the in-situ stress conditions to be less than that of the intact soil. Such destructuring can occur as a result of mechanical disturbance, or even as a result of stress relief under otherwise perfect sampling conditions (Leroueil, 2001). Conversely, the strengths measured from samples of normally consolidated soils can be higher than in-situ strengths due to void ratio reductions during recompression, if the samples have been disturbed. Thus, it may not be very practical to rely entirely on laboratory tests to obtain a representative profile of intact $s_u$ throughout the depth range of interest for a piling project.

The field vane shear test is a good means of obtaining a profile of intact $s_u$ that appears to be relevant to the cylindrical mode of shear induced by axially loaded piles. However, FVST measurements of $s_u$ are typically only obtained at intervals of 1.0 m or 5 ft or larger. Since the time to failure in the standard FVST is typically around 1 minute, the measured strengths can be higher than the strength that would be applicable to the much slower failure of a pile during a load test. This effect becomes increasingly significant with increasing soil plasticity, but appears to be small in the case of soils with low plasticity (Chandler, 1988).

Piezocone penetration testing is a more efficient means of characterizing a site, which allows near-continuous measurements of tip resistance, sleeve friction and pore pressure to be obtained very quickly, regardless of depth. CPTU profiles are also very useful in identifying the presence of sand seams within a fine-grained deposit, which can have a significant impact on helical pile installation performance as well as strength recovery rates. However, soil strength is not measured directly in this test and so strength correlation factors need to be used to estimate $s_u$. There is a wide range of values published in the literature for any particular cone-strength factor, and therefore there can be a large degree of error in $s_u$ estimates obtained using assumed cone-strength factors. Unless such information already exists for an area, it is recommended that site-specific CPTU-$s_u$ correlations always be obtained at multiple depths. The method used in this study to obtain such correlations is described in Appendix E.3.2.

When using the net tip resistance, $q_{T-\sigma_{vo}}$, to correlate to $s_u$ in soft soils, it is very important to consider the accuracy of the load cell being used to measure the cone tip pressures when they are
very small. For standard cones with 10 tonne tip capacity, the measured resistances can be less than 1% of the full-scale output of the load cell. If such equipment is to be used in soft soils, it is critical that resolution and accuracy limitations be assessed and instrument calibration be carried out over a range of output similar to that expected in the soft soils. It is preferable that CPT equipment with a more sensitive tip load cell be used in such soils. The excess pore pressure measured during CPTU penetration can provide a more reliable basis for correlating to $s_u$ in normally consolidated to lightly overconsolidated fine-grained soils.

### 8.2.4 Index of Soil Destructuring

Since different load-carrying elements on a helical pile will generate failure surfaces through different zones of soil which have undergone different degrees of destructuring, $I_D$ will vary for the different pile elements. Based on the results from the test piles installed in the highly structured Colebrook clayey silts during this study, the following values for $I_D$ are tentatively suggested:

- **Bottom helix:** $I_D = 0$,
- **All other helix plates above the bottom plate, $S/D \leq 1.5$:** $I_D = 0.75$, and
- **Pile shaft (grouted or ungrouted):** $I_D = 1.0$.

Selecting an average value of $I_D$ that is appropriate for the more widely spaced helix plates ($S/D=3$) appears to be more difficult. The values determined from pile TP3 in this study ranged from $I_D = 0.3$ for the middle helix to $I_D = 0.5$ or 0.65 for the upper helix, depending on whether $N_c = 8$ or 9 was more appropriate in that case. A design value of $I_D = 0.5$ could possibly be used, but this should be re-evaluated based on additional data as it becomes available. The higher degree of variability observed for the $S/D=3$ piles is believed to be due to the bearing-type failures that are induced by the more widely spaced helix plates. It should be recognized that while $N_c$ is typically taken to be 9 for deep foundations, this factor is purely empirical and may or may not be strictly appropriate for a given distribution of strength and stiffness around a particular bearing element. This greater degree of variability is a potential disadvantage for pile geometries with the more widely spaced helix plates.

All of the $I_D$ values given above are based on the results from load tests on a limited number of helical test piles, each with a limited number of helix plates and with a constant ratio of helix
plate diameter to shaft diameter, $D_{hx}/d_{shaft}$. For a given mode of shear, the degree of destructuring along the failure surface may increase with the number of helix plates that had penetrated through a particular zone of soil, and may decrease with increasing $D_{hx}/d_{shaft}$. However, these potential influences still need to be properly investigated. The recommended values for $I_D$ given above should be confirmed or revised based on experience with applying Equation 8.2 at different sites.

### 8.3 INCREASE IN SHEAR STRENGTH DUE TO AGING

It was shown in this study that 5 weeks of aging caused an increase in shear strength of at least 15% over that which was available at the end of the pore pressure dissipation period, in soil that had been significantly destructured during pile installation. Thus, for helical piles with $S/D \leq 1.5$, which mobilize resistance in cylindrical shear, the loss in strength that results from installation disturbance is partially recovered with time after installation. Based on the relatively good success that was found in predicting this aging-related strength gain in this study, the following prediction method is suggested:

$$\frac{(s_u)_A}{(s_u)_{EOP}} = 10^a$$

where $a = \Delta e_s(t)/C_c$ (from Eq. 7.4)

where: $\Delta e_s(t) =$ decrease in void ratio due to secondary compression,

$= C_\alpha \cdot \log(t/t_{EOP}),$ from Equation 7.2,

$(s_u)_A =$ undrained shear strength after aging,

$(s_u)_{EOP} =$ undrained shear strength at end of pore pressure dissipation period, as predicted using Equation 8.2,

$C_c =$ compression index of partially or totally destructured soil (eg. as measured in an oedometer test on a completely remoulded sample),

$C_\alpha =$ coefficient of secondary compression, and

$t, t_{EOP} =$ time since pile installation and time for complete pore pressure dissipation, respectively.
9.0 RECOMMENDATIONS FOR FURTHER STUDY

Additional research is recommended to confirm some of the explanations given for the soil behaviour observed in this study, and to answer some of the outstanding questions raised in this study. The areas of research recommended in this chapter include laboratory testing, numerical modelling and additional field testing.

9.1 LABORATORY TESTING

In this study, an understanding of the effects of pile installation on the Colebrook clayey silt has been determined indirectly from pore pressure measurements during pile installation and load testing and from the observed load-settlement response of the different pile sections during load testing. To confirm the interpretations of in-situ soil behaviour that were suggested in this study, detailed laboratory testing of the Colebrook clayey silt is recommended. In particular, a testing program is recommended that will allow the following important aspects of soil behaviour to be properly assessed:

- stress-strain behaviour of intact soil up to large strains,
- shear-induced pore pressure generation of intact soil with increasing shear strain, up to large strains,
- $s_u/\sigma'_{vc}$ of completely remoulded soil after inducing different degrees of destructuring (intact to) by undrained shearing of intact soil to different post-failure shear strain ($\gamma$) levels, which allows $I_D-\gamma$ relationship to be determined, and
- compression index ($C_c$) of soil during reconsolidation after different degrees of destructuring.

Due to the high sensitivity of the Colebrook soil, very careful sampling and sample handling procedures will need to be employed in order to properly characterize the behaviour of the intact soil. It is doubtful that sampling disturbance can be avoided using conventional piston-tube sampling equipment, based on past experiences with this soil. It may be necessary to use Laval (La Rochelle et al., 1981) or Sherbrooke (Lefebvre & Poulin, 1979) samplers to minimize the disturbance. Furthermore, it may be prudent to concentrate sampling and testing efforts on the zone of soil between −7 and −10 m elevation (below 6 m depth), which corresponds to the zone of soil in which the helix plates were embedded in this study. Greater success was achieved in
this study retrieving and extruding intact samples from this zone, as compared to the overlying zone of softer soil between –4.6 and –7 m elevation.

9.2 Numerical Modelling

Numerical modelling is probably the most effective way of improving our understanding of the distributions of strains, total stresses and pore pressures induced within the soil during pile installation, and of the changes in total stresses and pore pressure during reconsolidation. The characterization of the elemental behaviour of the Colebrook clayey silt, which would be acquired from the laboratory testing program, could be used to develop reasonably realistic stress-strain models for input into the numerical model. It is hoped that such a numerical model could provide similar insights into the effects of helical pile installation in fine-grained soils as the MIT research carried out by Lebadoux & Baligh (1980) provided for cone penetration in clays.

It was shown in this study that existing cylindrical consolidation solutions do not accurately predict the times for excess pore pressure dissipation around the helical piles. This is probably due in large part to the differences between the initial pore pressure distribution generated by helical pile installation and that assumed for the theoretical solutions. Variations in the coefficient of consolidation with distance from the pile (due to varying degrees of destructuring) and with time (due to changes in effective stresses during consolidation) may also be partially responsible for the differences between observed and predicted dissipation. Numerical analyses would allow these potential effects to be assessed and could provide more accurate dissipation solutions for helical piles for use by design engineers.

Also, the following insights would be very valuable in predicting the soil strengths that can be mobilized by the piles after installation:

- the distribution of shear strains caused by pile installation, which will control the degree of destructuring and related strength loss that occurs within the soil surrounding the pile, and

- the changes in vertical and radial effective stresses that occur as a result of pile installation and during reconsolidation, which will have a strong effect on the available shear strength at different times and on our ability to predict the development of shear strength.
9.3 **FIELD TESTING**

The existing capacity prediction methods based on the undrained shear strength of the soil, which were presented in Section 2.3, appear to be reasonably accurate when $s_u$ is known with confidence. Therefore, the prediction method proposed in Chapter 8 focuses on obtaining a reasonable estimate of the $s_u$ of real structured soils after disturbance by pile installation. To use this method in practice, an estimate of the “index of soil destructuring” ($I_D$) is required. The values that were tentatively suggested in Section 8.2.4 were based on back-calculations from the Colebrook data. Therefore, it is highly recommended that this capacity prediction method be verified using available load test and soil data from as many fine-grained soil sites as possible.

Additional field load tests could also be carried out at the Colebrook test site to directly compare piles with different helix plate geometries. For example:

- the effects of smaller shaft sizes for piles with equivalent helix diameters (i.e. larger helix-to-shaft diameter ratios, $D_{hx}/d_{shaft}$) on the mobilized capacity and on the $I_D$ of the soil,

- the effects of larger numbers of helix plates on the $I_D$ of the soil, and

- the difference between helical piles with closely spaced helix plates ($S/D = 1.5$) with a constant diameter compared to a tapered orientation (i.e. diameter increasing above the tip) with the same number of plates and the same average diameter.

The advantage to carrying out such field tests at the Colebrook site is that the pore pressure dissipation and strength recovery behaviour of the soil is already reasonably well understood, and there are detailed load test results from this study which can form a basis for additional geometric comparisons.
REFERENCES


APPENDIX A

EXCESS PORE PRESSURE FROM
THEORY & FIELD OBSERVATIONS

A.1 Cavity Expansion Theory
   A.1.1 General Theory
   A.1.2 Principal Stress Distributions from Elastic-Plastic Solutions
   A.1.3 Mean Normal Stresses and Pore Pressures

A.2 The Strain Path Method
   A.2.1 Strain Paths
   A.2.2 Deviatoric and Total Stresses
   A.2.3 Pore Pressures

A.3 Pore Pressure Measurements During Pile Installation at Different Sites
APPENDIX A: CAVITY EXPANSION THEORY AND THE STRAIN PATH METHOD

A.1 CAVITY EXPANSION THEORY

The value of the cavity expansion theory lies in the relative simplicity that results from the geometrical symmetry of the assumed deformation shape: either spherical symmetry, or cylindrical symmetry about a vertical axis with plane strain conditions in the vertical direction. For cavities expanded under conditions of zero volume change (i.e. undrained conditions), the distribution of strain around the cavity can be readily determined from the displacement field. To derive closed-form solutions of the stress distributions around an expanding cavity, a simple bilinear constitutive model is typically used, in which the soil is assumed to be linear-elastic up to failure and then perfectly plastic (e.g. Gibson & Anderson, 1961; Vesic, 1972). However, there are several methods for determining the stress-strain behaviour of a soil from the pressure-expansion response measured with the pressuremeter (Baguelin, 1972; Ladanyi, 1972; Palmer, 1972), which are based on the strain-displacement relations derived from CCE theory. The corollary to such methods is that the distribution of principal stresses around an expanded cavity can be estimated if the stress-strain curve is known.

A.1.1 General Theory

If a cavity is expanded from an initial radius, $R_0$, to a new radius, $R$, an element of soil at some initial radial distance, $r_0$, from the center of the cavity is displaced to a new radial position, $r$, such that the displacement $u_r = r - r_0$. As the cavity continues to expand, the displacement of the element, $u_r$, is proportional to the expansion of the cavity, $u_R$, according to:

$$
\frac{u_r}{u_R} = \frac{R}{r} \quad (A1)
$$

Due to strain compatibility, the element must deform in response to the displacement, and the resulting radial and circumferential principal strains, $\varepsilon_r$ and $\varepsilon_\theta$, respectively, are expressed as:

$$
\varepsilon_r = \frac{du_r}{dr} \quad (A2)
$$

$$
\varepsilon_\theta = \frac{u_r}{r} \quad (A3)
$$

where $dr$ is the radial width of the element and $du_r$ is the decrease in width. The strains $\varepsilon_r$ and $\varepsilon_\theta$ in Equations A2 and A3 are Cauchy strains that are only applicable for small strains. However,
for soil elements located close to an expanding cavity, the strains can be large (infinitely so at the wall of a cavity expanded from zero initial radius), and Cao et al. (2001) suggested using logarithmic strains instead:

$$\varepsilon_r = \ln(dr/dr_o)$$  \hspace{1cm} (A4)  

$$\varepsilon_\theta = \ln(r/r_o)$$  \hspace{1cm} (A5)

The radial strain is everywhere compressive while the circumferential strain is everywhere tensile. Under undrained conditions, the volumetric strain is zero and, therefore, $\varepsilon_r = -\varepsilon_\theta$ for cylindrical cavities (since $\varepsilon_z = 0$) and $\varepsilon_r = -2\varepsilon_\theta$ for spherical cavities. The shear strain, $\gamma$, is the difference between the major and minor principal strains and, therefore, $\gamma = 2\varepsilon_\theta$ for cylindrical cavity expansion (CCE) and $\gamma = 3\varepsilon_\theta$ for spherical cavity expansion (SCE).

The symmetry of cylindrical and spherical cavities results in equal and opposite forces from the stresses in the circumferential direction, as well as from the stresses in the vertical direction for cylindrical cavity expansion. Thus, equilibrium need only be satisfied in the radial direction, according to:

$$d\sigma_r/dr + (\sigma_r - \sigma_\theta)/r = 0 \quad \text{for CCE}$$  \hspace{1cm} (A6a)  

$$d\sigma_r/dr + 2(\sigma_r - \sigma_\theta)/r = 0 \quad \text{for SCE}$$  \hspace{1cm} (A6b)

where $\sigma_r$ and $\sigma_\theta$ are the total principal stresses in the radial and circumferential direction, respectively. Note that in the spherical case, $\sigma_1 = \sigma_r$ and $\sigma_2 = \sigma_3 = \sigma_\theta$ and therefore $\sigma_r - \sigma_\theta$ is equivalent to the axi-symmetric deviator stress, $q_{AS}$, while in the cylindrical case, $\sigma_r - \sigma_\theta$ is equivalent to the plane strain deviator stress, $q_{PS}$ ($\sigma_1 = \sigma_r$, $\sigma_3 = \sigma_\theta$ and $\sigma_2 = \sigma_z$). Therefore, Equation A6 indicates that the slope, $d\sigma_r/dr$, of the distribution of radial stress with radial distance at any distance, $r$, from the cavity is a function of $q_{oct}$ at that location, which is a function of shear strain according to the governing stress-strain relation. If the soil around the cavity is assumed to be homogeneous and isotropic, every element of soil will tend to follow the same unique relation between shear stress, $\tau = q/2$, and shear strain, $\gamma$, due to the symmetry of loading (Ladanyi, 1972).
Ladanyi (1972) demonstrated how Equation A6 can be integrated and applied incrementally to obtain the variation of q (or $\tau$) with r based on the recorded variation in measured cavity pressure ($\sigma_r$ at $r = R$) with cavity radius, R, from a pressuremeter test. From the q(r) or $\tau(r)$ distribution, the $\tau$-$\gamma$ curve can be derived based on the assumed distribution of $\gamma$ with r and the assumption of a unique $\tau$-$\gamma$ relation. The integrated form of Equation A6, expressed as an increment of radial stress, is as follows:

\[
(\sigma_r)_{r_1} - (\sigma_r)_{r_2} = 2(\tau_{PS})_{r_1,r_2}\ln(r_2/r_1) \quad \text{for CCE} \tag{A7a}
\]

\[
(\sigma_r)_{r_1} - (\sigma_r)_{r_2} = 4(\tau_{AS})_{r_1,r_2}\ln(r_2/r_1) \quad \text{for SCE} \tag{A7b}
\]

where $r_1$ and $r_2$ are two arbitrary radial distances with $r_2 > r_1$, and ($\tau_{PS}$)$_{r_1,r_2}$ and ($\tau_{AS}$)$_{r_1,r_2}$ are the average plane strain and axi-symmetric shear stresses, respectively, over the increment between $r_1$ and $r_2$. From Equation A7 it is apparent that for a constant shear stress, the radial stress will vary linearly with the logarithm of the radial distance from the center of the cavity. Using Equation A7, the distribution of $\sigma_r$ with radial distance from a cavity expanded from zero radius can be constructed incrementally from the theoretical strain distribution if the stress-strain curve for the material is known. The distribution of the minor principal stress, $\sigma_\theta(r)$, can be determined from the distribution of $\sigma_r(r)$ if the $\tau$-$\gamma$ curve is known, since $\sigma_r - \sigma_\theta = 2\tau$.

### A.1.2 Principal Stress Distributions from Elastic-Plastic Solutions

If the soil is modeled as an ideal linear-elastic, perfectly plastic material, the stresses and pore pressures induced within the soil surrounding an expanding cavity can be calculated using closed-form solutions. (eg. Gibson and Anderson, 1961 and Vesic, 1972). In the elastic range, the stress-strain relations are described using Young’s modulus, E, and Poisson’s ratio, $\nu$ ($\approx 0.5$ for undrained loading), assuming that the soil is isotropic. These two parameters also provide the elastic shear stiffness, G, since $G = E/(1+\nu)$, which relates the shear stress and strain in the elastic region linearly according to $\tau = G\gamma$. The soil is assumed to become perfectly plastic when the shear stress reaches the undrained shear strength, $s_u$, of the soil, beyond which $\tau = s_u$ and is independent of strain level. If the soil is assumed to be linear elastic right up to failure, the shear strain at failure is determined simply as $\gamma_f = s_u/G$. 
Around any cavity expanded from zero radius, the strain at the cavity wall is infinite and drops rapidly with increasing radial distance, approaching zero at infinite distance from the cavity. Therefore, the strains within an annulus around the cavity will be larger than $\gamma_f$ and the soil will have failed, becoming completely plastic. Beyond the edge of this “plastic zone”, $\gamma < \gamma_f$ and $\tau < s_u$, and therefore the material is still elastic. For the case of the linear-elastic, perfectly plastic material, it can be shown that the radial distance to the boundary of the plastic zone, $r_p$ (where $\gamma = \gamma_f$), can be expressed as:

$$r_p/R = [G/(s_u)_{PS}]^{1/2} \text{ for CCE} \quad (A8a)$$

$$r_p/R = [G/(s_u)_{AS}]^{1/3} \text{ for SCE} \quad (A8b)$$

where $R$ is the cavity radius. The stiffness ratio, $G/s_u$, is also called the rigidity index, $I_r$.

**Cylindrical Cavity Expansion (CCE) Solutions**

The stress-strain relations for a linear-elastic material are governed by the following set of equations:

$$\varepsilon_r = (\sigma_r - \nu \sigma_\theta - \nu \sigma_z)/E \quad (A9)$$

$$\varepsilon_\theta = (\sigma_\theta - \nu \sigma_r - \nu \sigma_z)/E \quad (A10)$$

$$\varepsilon_z = (\sigma_z - \nu \sigma_\theta - \nu \sigma_r)/E \quad (A11)$$

For plane strain, $\varepsilon_z = 0$. Therefore, from Equation A11:

$$\sigma_z = \nu(\sigma_r + \sigma_\theta) = 0.5(\sigma_r + \sigma_\theta) \text{ for undrained loading} \quad (A12)$$

When Equations A9, A10, A12 and A6a are combined, it can be shown that the distributions of radial and circumferential stresses with radial distance within the elastic zone are given by

$$\sigma_r(r) = \sigma_o + 2G\varepsilon_\theta(r) \quad (A13)$$

$$\sigma_\theta(r) = \sigma_o - 2G\varepsilon_\theta(r) \quad (A14)$$
where $\sigma_o$ is the initial isotropic stress in the soil before cavity expansion. From Equations A13 and A14 it can be seen that the increments in radial and circumferential stresses, $\Delta \sigma_r = \sigma_r - \sigma_o$ and $\Delta \sigma_\theta = \sigma_\theta - \sigma_o$, respectively, are equal and opposite. According to Equation A12, $\sigma_z(r)$ is the average of $\sigma_r(r)$ and $\sigma_\theta(r)$, and therefore $\sigma_z(r)$ reduces to $\sigma_o$ for any radial distance within the elastic zone.

At the edge of the plastic zone, $\varepsilon_\theta(r=r_p) = \gamma f/2 = (s_u/G)/2$, and therefore $\sigma_r(r=r_p) = \sigma_o + s_u$ and $\sigma_\theta(r=r_p) = \sigma_o - s_u$ from Equations A13 and A14. Inside the plastic zone, $\tau$ remains constant at $s_u$, for a perfectly plastic material, and therefore $\sigma_r$ will increase linearly with the logarithm of distance from $r = r_p$ to $r = R$ at the cavity wall, according to the following expression derived from Equation A7:

$$\sigma_r(r_p \geq r \geq R) = \sigma_o + (s_u)_{PS} + 2(s_u)_{PS} \ln \left[ \frac{1}{r/R} \right] \quad (A15)$$

Within the plastic zone, $\sigma_\theta$ is dependent on $\sigma_r$ according to $\sigma_\theta = \sigma_r - 2s_u$ since failure has occurred in the $r-\theta$ plane. Also, the intermediate principal stress, $\sigma_z$, will still be the average of $\sigma_r$ and $\sigma_\theta$ for all radii within the plastic zone, according to Equation A12, provided that the initial stress state is isotropic. Equation A12 still applies within the plastic zone since the $\sigma_r - \sigma_z$ and $\sigma_z - \sigma_\theta$ stress differences will never become large enough to cause yielding along either of the vertical planes once yielding occurs on the horizontal plane.

It can be shown that the radial stress at the wall of the cavity reaches a limiting pressure when the cavity is expanded from zero initial radius. This limit pressure is dependent on the stress-strain properties of the soil as described above, but is independent of the size of the cavity (Gibson and Anderson, 1961). The limit pressure can be calculated readily using the elastic-plastic model. At the cavity wall, $r = R$, and Equation A15 reduces to the following expression for the cylindrical limit pressure, $p_L$:

$$p_L = \sigma_r(r=R) = \sigma_o + (s_u)_{PS} \ln \left[ \frac{1}{r/R} \right] \quad \text{for CCE} \quad (A16a)$$

Similarly, the spherical limit pressure can be shown (eg. by Gibson & Anderson, 1961) to be:

$$p_L = \sigma_o + 4/3(s_u)_{PS} \ln \left[ \frac{1}{r} \right] \quad \text{for SCE} \quad (A16b)$$
A.1.3 Mean Normal Stresses and Pore Pressures

The excess pore pressure, \( \Delta u = u - u_0 \), which is generated around an expanded cavity under undrained conditions can be considered as the combination of two components:

- \( \Delta u_{\text{oct}} \): the pore pressure resulting from the increase in mean total stress (also known as the total octahedral stress), \( \Delta \sigma_{\text{oct}} \), and
- \( \Delta u_{\text{shear}} \): the pore pressure resulting from the changes in deviator stresses.

For spherical and cylindrical cavities, \( \sigma_{\text{oct}} = (\sigma_r + 2\sigma_\theta)/3 \) and \( (\sigma_r + \sigma_\theta + \sigma_z)/3 \), respectively, which reduces to \( \sigma_{\text{oct}} = (\sigma_r + \sigma_\theta)/2 \) for the case of an isotropic initial stress state where \( \sigma_z = (\sigma_r + \sigma_\theta)/2 \), as discussed above. Using the elastic-plastic model and assuming an isotropic initial stress state (with initial stress \( \sigma_0 \)), the distribution of \( \sigma_{\text{oct}} \) with radial distance can be expressed using the following simple equations:

\[
\sigma_{\text{oct}}(r) = \sigma_0 + 2(s_u)_{\text{PS}} \cdot \ln\left[\frac{1}{2}\left(\frac{r}{R}\right)\right] \text{ for CCE} \\
\sigma_{\text{oct}}(r) = \sigma_0 + 4(s_u)_{\text{AS}} \cdot \ln\left[\frac{1}{3}\left(\frac{r}{R}\right)\right] \text{ for SCE} \tag{A17a} \tag{A17b}
\]

The distribution of \( \Delta u_{\text{oct}}(r) \) around the cavity will follow the distribution of \( \Delta \sigma_{\text{oct}}(r) \) since \( \Delta u_{\text{oct}} = \Delta \sigma_{\text{oct}} \) in a saturated soil. No shear-induced pore pressure will be generated if the stress-strain behaviour of a soil is linear-elastic. Therefore, \( \Delta u_{\text{shear}} = 0 \) up to failure for the elastic-plastic model and the effective stress path is vertical. Failure is assumed to occur when the effective stress path reaches the effective stress strength envelope. Once the strength envelope is reached, no change in effective stress will occur during perfectly plastic shearing since \( \Delta \tau = 0 \). Thus, the elastic-plastic model predicts \( \Delta u_{\text{shear}} = 0 \) for all soil elements around the cavity and therefore \( \Delta u = \Delta u_{\text{oct}} \), such that \( \Delta u(r) \) can be expressed as:

\[
\Delta u(r) = 2s_u \cdot \ln\left[\frac{1}{2}\left(\frac{r}{R}\right)\right] \text{ for CCE} \tag{A18a} \\
\Delta u(r) = 4s_u \cdot \ln\left[\frac{1}{3}\left(\frac{r}{R}\right)\right] \text{ for SCE} \tag{A18b}
\]
A.2 **The Strain Path Method**

Baligh (1975), Levadoux & Baligh (1980) and Baligh & Levadoux (1980) describe a method of estimating the stresses and pore pressures induced in saturated clays by deep steady cone penetration called the Strain Path Method (SPM). The steps followed by this method are listed below:

1) The soil is modelled as an ideal incompressible fluid flowing up along a static penetrometer. A velocity field around the penetrometer is assumed such that particles move along continuous streamlines located axi-symmetrically at different radial distances from the axis of the penetrometer.

2) The particle displacements and the resulting deformations between particles are determined by integrating the velocities along streamlines.

3) The strain rates along the streamlines are determined by differentiating the velocities with respect to the spatial coordinates.

4) The strain path ($\varepsilon_{ij}$) of each element is determined by integrating the strain rates along the streamlines.

5) The initial stresses and pore pressures in the soil before penetration are estimated.

6) The deviatoric stresses ($s_{ij}$) and shear-induced pore pressures ($\Delta u_{shear}$) along streamlines are calculated using appropriate $s_{ij} - \varepsilon_{ij}$ and $\Delta u_{shear} - \varepsilon_{ij}$ relations.

7) The total principal stresses ($\sigma_{ij}$) are calculated from the deviatoric stresses and the mean total (octahedral) stress, $\sigma_{oct}$. The octahedral stress is estimated using the deviatoric stresses in an equation of equilibrium for either the radial or axial direction. The effective principal stresses ($\sigma'_{ij}$) are calculated from the initial effective stress state, the deviatoric stresses and the shear-induced pore pressures.

8) The total excess pore pressure is calculated from the estimated $\Delta \sigma_{oct}$ and $\Delta u_{shear}$.

Teh & Houlsby (1991) developed a large-strain finite element solution of the cone penetration process that is based on the strain path method and uses elastic-plastic constitutive laws to derive the deviatoric stresses from the strain path of each soil element. Houlsby & Teh report that the
stress state that is obtained by satisfying equilibrium in one direction will not be in equilibrium in the orthogonal direction. In their numerical solution, they correct this condition at each step of the penetration process by incrementally applying equal and opposite forces until force equilibrium is achieved. Incremental penetration is continued until a steady resistance is achieved. The excess pore pressures are then calculated from the increase in mean normal stress ($\Delta u_{\text{oct}} = \Delta \sigma_{\text{oct}}$) and from the increase in octahedral shear stresses using Henkel’s pore pressure parameter, $\alpha$ ($\Delta u_{\text{shear}} = \alpha \Delta \tau_{\text{oct}}$), where $\alpha = 1$ was assumed, which is generally applicable for normally consolidated clays (Teh & Houlsby, 1991).

Levadoux & Baligh (1980) used the total stress soil model MIT-T1 to calculate deviatoric stresses and shear-induced pore pressures based on observed soil properties from laboratory testing of re-sedimented Boston blue clay (BBC) normally consolidated under $K_o$ conditions. The total principal stresses and octahedral pore pressures were then calculated by satisfying force equilibrium in the radial direction. Equilibrium in the axial direction was also considered, but the estimated distributions of $\sigma_{\text{oct}}$ were found to be sensitive to the values $\tau_{rz}$, which are based on $\gamma_{rz}$ values for an ideal fluid, which may be greater than those of a clay which has some shearing resistance. Thus, radial equilibrium was expected to be more reliable.

The soil constitutive model used by Levadoux & Baligh (1980) is more comprehensive and is expected to be more realistic than the elastic-plastic model used by Teh & Houlsby (1991), at least for soils with properties similar to normally consolidated BBC. However, the Teh & Houlsby solutions should be more widely applicable to soils of varying rigidity, although the solutions may not be as accurate for any one particular soil. Unfortunately, the broader applicability of the Teh & Houlsby solutions is tempered somewhat by the existing difficulties in estimating the rigidity index, $I_r$, of a soil, as will be discussed in Appendix B.

A summary of some of the key results of the SPM-based analyses carried out by Levadoux & Baligh (1980) are provided below.

### A.2.1 Strain Paths

As a cone probe (or pile with conical toe) penetrates into the soil, soil particles initially located at distances between 1R (where R is the probe radius) and 5R from the axis of the probe showed similar trends of movement, although the magnitude of the displacement decreased with radial distance. As the cone approaches the level of the soil particles, they are initially displaced...
downward as well as radially outward. As the face of the cone passes the particles, they are
displaced predominantly outward, and as the shoulder of the cone passes, the particles move
upward again, returning to about their original elevation. The vertical component of
displacement was shown to be greater for cones with larger apex angles (i.e. blunter tips).

Due to this displacement pattern, the strain paths followed by soil elements that are deformed by
pile or cone penetration are very complex, involving large strains in all 3 dimensions, and
including strain reversals. The E2 strain, \( \varepsilon_{\theta\theta} - \varepsilon_{rr}/\sqrt{3} \), in the horizontal plane, which is the
component produced by pressuremeter testing and modelled by CCE, is the largest of the strain
components, reaching magnitudes of around 175% in an element initially located at 0.2R from
the axis of the probe. For the same element, the E1 strain, \( \varepsilon_{zz} \), which is produced in triaxial tests,
is estimated to be about 50%, while the E3 strain, \( 2\varepsilon_{rz}/\sqrt{3} \), which is produced in direct simple
shear tests, is estimated to be about 115%. Neither of the latter two strain components are
considered by cavity expansion solutions.

The fields of radial strain, \( \varepsilon_{rr} \), and octahedral shear strain, \( \gamma_{oct} \), around the shaft of the cone probe
were observed to be almost identical to those predicted by CCE, at distances greater than about 5
radii above the tip of the cone.

**A.2.2 Deviatoric and Total Stresses**

The deviatoric stress paths followed by soil elements located around a penetrating cone probe are
very complex, and can be considered to be a combination of triaxial compression, direct simple
shear and pressuremeter shear modes.

The changes in the octahedral shear stress, \( \tau_{oct} \), and the principal total stresses, \( \sigma_z \), \( \sigma_r \) and \( \sigma_\theta \)
(vertical, radial and circumferential) as the cone approaches and then passes a soil element,
which is initially located on the centerline of the probe, are illustrated on Figure A-1. As the
cone approaches, \( \tau_{oct} \) reaches a maximum within a distance of about 7 radii below the tip. At
distances closer than about 2 radii below the tip, \( \tau_{oct} \) decreases significantly with increasing
strain, reaching a residual value 2.5 times less than the peak strength, just below the tip of the
cone.

All 3 principal stresses increase substantially as the cone approaches, reaching maxima along the
face of the cone which are about 50% higher than those predicted by cylindrical cavity
expansion. At this point, the order of the principal stresses changes from the initial cross-anisotropic stress state ($\sigma_z > \{\sigma_r = \sigma_\theta\}$) to a stress state which is similar to that predicted by CCE theory where $\sigma_r > \sigma_z > \sigma_\theta$, but where the deviator stresses are relatively small due to the post-failure strain softening. Once the shoulder of the cone passes the soil element, all 3 principal stresses decrease significantly, reaching values which are close to a steady state at distances of around 10 to 15 radii above the tip of the cone, and are about 33% less than those predicted by CCE theory.

The greatest decrease is observed in $\sigma_r$, such that the stress state behind the shoulder of the cone changes to $\sigma_z > \sigma_\theta > \sigma_r$. This sharp decrease in $\sigma_r$ behind the shoulder of the conical tip is consistent with the trends of $\sigma_r$ measured by Jardine et al. (1998) using the Imperial College instrumented model pile. The contours of $\sigma_r$ predicted around the cone probe during steady penetration by Levadoux & Baligh (1980) are shown on Figure A-2. It can be seen that, within a distance of at least 7.5 radii above the tip, the gradient of $\sigma_r$ in the vicinity of the shaft of the probe is predominantly upward and slightly inward. This stress regime is certainly conducive to the upward redistribution of total stresses through the softened soil along the shaft of the cone probe once penetration is stopped.

A.2.3 Pore Pressures

The contours of excess pore pressure predicted by Levadoux & Baligh (1980) using both the SPM and CCE theory with the same stress-strain and pore pressure properties are shown on Figure A-3. If the radial distribution of $\Delta u$ predicted by Levadoux & Baligh across a horizontal plane located well above the cone is compared to that predicted by CCE, it can be seen that the SPM predicts lower $\Delta u$ close to the shaft and slightly higher $\Delta u$ at larger distances. The main gradient of $\Delta u$ close to the shaft above the shoulder of the cone is observed to be both upward and outward, as compared to the gradient of $\sigma_r$ which is observed to be predominantly upward and slightly inward from Figure A-2. This is partly due to the distribution of $\Delta u_{\text{shear}}$ predicted by Levadoux & Baligh, which is uniform in the vertical direction, such that the gradient of $\Delta u_{\text{shear}}$ is entirely radial (Levadoux & Baligh predict $\Delta u_{\text{shear}}/\sigma'_{\text{vo}} > 0.4$ at the surface of the shaft). Thus, for lightly over-consolidated soils where $\Delta u_{\text{shear}} \approx 0$, the main gradient of $\Delta u$ close to the shaft above the shoulder of the cone can be expected to be even closer to vertical.
A.3 **Pore Pressure Measurements During Pile Installation at Different Sites**

Table A-1 is a summary of the studies referenced on Figure 2.4 by Levdoux & Baligh (1980).

**Table A-1**

<table>
<thead>
<tr>
<th>Case (from Fig. 2.4)</th>
<th>Reference</th>
<th>Site</th>
<th>Pile Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>Bjerrum &amp; Johannessen (1960)</td>
<td>Southern Norway</td>
<td>Driven Steel Box Section (20 × 20 cm)</td>
</tr>
<tr>
<td>b</td>
<td>Lo &amp; Stermac (1965)</td>
<td>Wallaceberg, Ontario</td>
<td>Driven Steel Pipe (8.9 cm dia.)</td>
</tr>
<tr>
<td>c</td>
<td>Koizumi &amp; Ito (1967)</td>
<td>Otemachi, Tokyo</td>
<td>Jacked Steel Pipe (30 cm dia.)</td>
</tr>
<tr>
<td>d</td>
<td>Roy et al. (1979)</td>
<td>St. Alban, Quebec</td>
<td>Jacked Steel Pipe (22 cm dia.)</td>
</tr>
<tr>
<td>e</td>
<td>Baligh et al. (1978)</td>
<td>Saugus, Massachusetts</td>
<td>Jacked Steel Cylinder (3.8 cm dia.)</td>
</tr>
<tr>
<td>f</td>
<td>Airhart et al. (1969)</td>
<td>Beaumont, Texas</td>
<td>Driven Steel Pipe (41 cm dia.)</td>
</tr>
<tr>
<td>g</td>
<td>Ismael &amp; Klym (1979)</td>
<td>Pickering, Ontario</td>
<td>Driven Steel H-Pile (12” × 53 lb.)</td>
</tr>
<tr>
<td>h</td>
<td>Soderman &amp; Milligan (1961)</td>
<td>Big Pic River, Canada</td>
<td>Driven Steel H-Pile (12” × 53 lb.)</td>
</tr>
</tbody>
</table>
Figure A-1: Predicted Variation of Octahedral Shear Stress and Principal Total Stresses Below and Along Surface of Cone Probe during Penetration in Normally Consolidated Boston Blue Clay (from Baligh & Levadoux, 1980)
Figure A-2: Contours of Total Radial Stress Around Penetrating Cone in Normally Consolidated Boston Blue Clay (from Baligh & Levadoux, 1980)

Figure A-3: Contours of Excess Pore Pressure Around Penetrating Cone in Normally Consolidated Boston Blue Clay (from Baligh & Levadoux, 1980)
APPENDIX B

DISSIPATION SOLUTIONS FOR
PILES AND PIEZOCONE PROBES

B.1 Summary of Dissipation Solutions
B.2 Estimating Rigidity Index
APPENDIX B: DISSIPATION SOLUTIONS FOR PILES & PIEZOCONE PROBES

B.1 SUMMARY OF DISSIPATION SOLUTIONS

The following dissipation solutions were considered for comparing with the dissipation test data obtained in this study:

<table>
<thead>
<tr>
<th>Method</th>
<th>Consolidation Analysis</th>
<th>Initial Pore Pressure Distribution</th>
<th>Proposed Use</th>
</tr>
</thead>
<tbody>
<tr>
<td>Torstensson (1977)</td>
<td>Radial consolidation around impervious cylindrical and spherical cavities</td>
<td>Cylindrical &amp; Spherical Cavity Expansion in Elastic-Plastic soil</td>
<td>Piezocone Probe (non-standard probe geometry)</td>
</tr>
<tr>
<td>Randolph &amp; Wroth (1979)</td>
<td>Radial consolidation around impervious cylindrical cavity</td>
<td>Cylindrical Cavity Expansion in Elastic-Plastic soil</td>
<td>Shaft of driven cylindrical piles</td>
</tr>
<tr>
<td>Baligh &amp; Levadoux (1980), Levadoux &amp; Baligh (1986)</td>
<td>2-D consolidation around piezocone probe</td>
<td>Strain Path Method with properties of resedimented NC BBC</td>
<td>Piezocone Probe (specific filter locations)</td>
</tr>
</tbody>
</table>

All of the above solution methods are based on linear, uncoupled Terzaghi-Rendulic consolidation theory. The consolidation analyses by Torstensson and Houlsby & Teh were carried out using finite difference numerical methods, while the analyses by Baligh & Levadoux were carried out using a finite element approach. The Randolph & Wroth solutions were derived analytically using a closed-form solution. In most solutions, the excess pore pressure, \( \Delta u(t) \), at time, \( t \), is normalized by the excess pore pressure at the start of consolidation, \( \Delta u_i \), and is plotted against a dimensionless time factor, \( T \), where:

\[
T = c_h t / R^2
\]  

(B1)

and \( R \) is the radius of the cone probe or the induced soil cavity.

Torstensson (1977) and Randolph & Wroth (1979) published different dissipation curves for different values of the rigidity index, \( I_r = G/s_{0u} \), since this parameter controls the radial extent of
excess pore pressures generated during cone penetration, and therefore the time for dissipation. Since the time required for dissipation increases with increasing $I_r$, Teh & Houlsby (1991) proposed a modified time factor, $T^*$, where:

$$T^* = \frac{c_t t}{R^2 I_r^{1/2}}$$  \hspace{1cm} (B2)

Using this time factor, Teh & Houlsby were able to generate a unique dissipation curve for different locations along the cone probe. Teh & Houlsby acknowledged that a more rational approach would be to normalize the time factor in terms of the radius of the zone of plastic deformation around the cone, $r_p^2 = (R\sqrt{I_r})^2$. However, they chose to normalize their solution in terms of $R^2 I_r^{1/2}$ because they found that this most successfully unified the results of their numerical solutions for different values of $I_r$.

The normalized dissipation curves generated by Baligh & Levadoux (1980) for the U2 and U3 filter positions on the cone probe (located 2R and 10R above the cone tip, respectively) are compared on Figure B-1 to the corresponding U2 and U3 dissipation curves from Teh & Houlsby (1991) using an $I_r$ of 350. This rigidity index is considered to be representative of the Colebrook clayey silts. It can be seen that the Teh & Houlsby curves plot slightly to the left of the Baligh & Levadoux curves. If an $I_r$ of 500 was used in the Teh & Houlsby solutions instead of 350, the resulting curves would be essentially coincident with the Baligh & Levadoux curves. The dissipation curve published by Randolph & Wroth (1979) for $I_r = 400$ is also plotted on Figure B-1. The dissipation solutions by Torstensson (1977) for cylindrical and spherical cavities have not been shown because published solutions were only available for $I < 167$. However, in Robertson et al. (1992), the SPM dissipation solution by Houlsby & Teh (1988) for the U2 location was shown to be virtually identical to the solution by Torstensson (1977) for cylindrical cavities.

The time for dissipation is strongly dependent on the initial extent of excess pore pressure, which depends solely on the rigidity index ($G/s_u$) for an elastic-plastic soil. Therefore, an estimate of the equivalent $G/s_u$ ratio for a soil is a useful component in ensuring that an estimate of the dissipation duration will be on the correct order of magnitude. It is this feature that makes the

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1 Solutions were published by Teh & Houlsby (1991) for locations at 5 and 10 radii above the shoulder of the cone which are similar in shape and magnitude. The consolidation curve for the U3 location was obtained by interpolating between these two published curves.
dissipation solutions by Houlsby & Teh (1988) attractive. However, obtaining a suitable value of the equivalent elastic shear stiffness, $G$, is very difficult in practice, as will be discussed below.

### B.2 Estimating Rigidity Index

The selection of a suitable rigidity index, $I_r = G/s_u$, to describe elastic-plastic constitutive models which are commonly used in pore pressure generation and dissipation solutions, is not a trivial process due to the difficulty in selecting a suitable value of the shear stiffness, $G$. Much of this difficulty can be attributed to the fact that soils do not behave linearly throughout the pre-failure strain region, and therefore $G$ is not unique, but instead depends on the level of shear strain, $\gamma$. Thus, when modelling non-linear stress-strain behaviour with an equivalent elastic stiffness, a secant shear stiffness, $G_{sec}$, must be selected which provides a best fit of the actual stress-strain curve up to the expected level of shear. For estimating the pore pressure generation around an expanded cavity, the value of $G_{sec}$ that is selected should be roughly representative of the stress-strain behaviour throughout the range of shear stress up to failure.

The above discussion assumes that the stress-strain curve of the material is known. However, even when laboratory testing (e.g. triaxial or direct simple shear) is carried out to obtain stress-strain curves, the effects of sample disturbance must be considered, since disturbance greatly reduces both the initial tangent modulus and any secant moduli determined from the measured stress-strain curve. Ward et al. (1959) showed that the initial stiffness of London clay measured in triaxial tests was 2 to 3 times lower when measured from driven tube samples compared to block samples. Furthermore, a comparison of values of $G$ measured using in-situ methods with those measured from laboratory testing on samples (Wroth et al., 1984 and d’Appolonia et al., 1971) shows that the lab-determined values are almost always significantly less than the corresponding field-measured values.

The “elastic” shear stiffness can be measured in-situ by means of plate load tests (for shallow soils), from back-analysis of the settlement of full-scale foundations (to obtain an average response over a range of depths), and from unload/reload loops measured using the self-boring pressuremeter. Unfortunately, pressuremeter testing in North America is typically limited to large projects due to the expense associated with this type of testing, and so this data is not
usually available. The small-strain shear stiffness, $G_{\text{max}}$, can be readily calculated from shear wave velocity measurements which can be carried out in-situ. In order to obtain an appropriate value of $G_{\text{sec}}$ from $G_{\text{max}}$, however, the $G(\gamma)/G_{\text{max}}$ attenuation relation for the intact soil must be known, along with the average shear strain, $\gamma$, for which $G_{\text{sec}}$ is desired. Relations for $G(\gamma)/G_{\text{max}}$ as a function of $\gamma$ are available in the literature, some of which are dependent on plasticity index. However, using such relations requires a knowledge of the range of $\gamma$ that covers the pre-failure range for a particular soil in a given mode of shear, and this is not usually known.

Thus, for most applications, the engineer can only estimate representative values of $G/s_u$ from those measured at other sites with similar soil conditions. D’Appolonia et al. (1971) compared values of $E_u$ back-calculated from published settlement records with $s_u$ strengths measured from field vane tests and consolidated triaxial tests. They concluded that the ratio of $E_u/s_u$ (and therefore $G/s_u$) decreases with increasing plasticity and is lower for clays with higher organic contents. For inorganic clays of low plasticity and moderate to high sensitivity, they recommend $E_u/s_u$ ratios in the range of 1000 to 1500 (i.e. $G/s_u = 330$ to 500). Based on $K_0$ consolidated direct simple shear testing of 3 different clays with varying plasticity and organic content, Ladd & Edgers (1972) also showed that $G/s_u$ decreased significantly with increasing OCR, for OCR $> 1.5$ to 2.
Figure B-1: Theoretical Dissipation Solutions
APPENDIX C

SAMPLE BORING LOG
(MoTH INVESTIGATION)
APPENDIX C: SAMPLE BORING LOG (MoTH INVESTIGATION)

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Sample Type: A - Auger, C - Core, D - Diamond, S - Split Spoon, T - Shelby Tube
Shear Strength: U - Unconfined Compression, V - Triaxial Compression, P - Cyclic Compression
Tests: M - Mechanical Analysis, DR - Direct Shear, WL - Liquid Plastic Limits, W - Moisture Content
File No.: H181 (REV 84 06)
APPENDIX D

FIELD VANE SHEAR STRENGTH TESTING

D.1 Test Apparatus
D.2 Test Procedure
D.3 Analysis of Test Data
   D.3.1 Determining Undrained Shear Strength from FVST
   D.3.2 Determining Vane Rotation and Torque-Rotation Relations
D.4 Test Results
APPENDIX D: FIELD VANE SHEAR STRENGTH TESTING

D.1 TEST APPARATUS

Field vane shear strength tests (FVST) were carried out using the Nilcon vane boring apparatus, which was supplied and operated by ConeTec Investigations Ltd. for this study. This apparatus includes a torque-recording device that records the change in torque with vane rod rotation on a wax paper disc using a spring-loaded scribing needle. An example of such a record is shown on Figure D-1.

A slip coupling is incorporated in the vane rods just above the vane, which allows up to 15º of rotation with minimal torque. Thus, the initial torque required to overcome the friction of the soil on the vane rods is typically separated from the torque required to turn the vane by a brief period of free slip which can be identified on the test record, as shown on Figure D-1. A typical Nilcon vane with slip coupling is shown on Figure D-2.

Since the rod friction can be accounted for using this method, vane tests can be carried out at multiple depths without having to remove the vane from the hole, advance the borehole and then re-insert the vane. The Nilcon vanes also have a slight pitch which is designed to cause the vane to rotate via the slip coupling in the opposite direction from the direction of rotation during the shear test (approximately 15º during a 1 m push to the new test depth). This is supposed to re-establish the available free slip in the slip coupling for each new test.

The vane used in this study was 8 cm in diameter and 17.2 cm long including a 60º (from vertical) tapered tip, with a corresponding vane constant (C) of 0.05 kg/cm² per kg-m of torque (or 5 kPa per kg-m). The vane rods were 20 mm diameter solid rods.

D.2 TEST PROCEDURE

For each vane boring, the upper 1.2 m of soil was pre-drilled and cased off using a hollow-stem auger and the vane was pushed through the soil below this depth. The tests were generally started within about 1 minute of pushing the vane to the new test depth. In each test, rotation of the vane was continued beyond the point of failure until the rate of post-peak strength reduction had dropped significantly, so that a record of the strain-softening behaviour of the soil could be
obtained. A remoulded strength test was then performed at each depth after rotating the vane through 20 complete revolutions.

The time to failure in the vane tests generally ranged from 1.3 to 2.7 minutes, such that the average rate of vane rotation typically ranged from 20 to 35 degrees per minute. This is significantly faster than the standard rate of 6°/min, but research by Roy and Leblanc (1988) on low plasticity clays ($I_p < 20\%$) from Quebec indicates that there is very little increase in vane shear strength due to viscous rate effects at increasing rotation rates. In fact, a significant increase in strength was observed at rotation rates slower than about 7 or 8 degrees per minute, which they attributed to partial drainage occurring during the tests. Thus, for low plasticity soils, it would appear to be more critical to ensure that the test is carried out quickly enough to ensure undrained conditions, than to be concerned with too fast of a shear rate.

To ensure undrained conditions during the vane test, Chandler (1988) suggests that the degree of consolidation during the test should be less than 10%. Based on a theoretical solution for consolidation around the vane proposed by Blight (1968) and supported by experimental evidence from Blight as well as Roy and Leblanc (shown on Fig. 12 of Chandler, 1988), the corresponding time factor to failure ($T_f = c_v t_f / D^2$, where $c_v$ is the coefficient of consolidation and $D$ is the diameter of the vane) should be less than 0.05. Blight’s theory assumes that excess pore pressure generation around the vane during the shear test is uniform within a spherical zone of influence centered around the vane and that the surface of this sphere acts as a drainage boundary. When the coefficient of consolidation in the horizontal direction ($c_h$) is greater than in the vertical direction, as is typically the case for natural sedimentary clays, it would be reasonable to expect that consolidation around the vane would be controlled by horizontal drainage. Therefore, the value of $c_h = 0.02 \text{ cm}^2/\text{s}$ determined by Crawford & Campanella (1991) from a CPTU dissipation test at 10 m depth should be reasonably applicable to this problem.

Thus, for an 8 cm diameter vane, the maximum time to failure ($t_f$) to ensure essentially undrained conditions is 2.7 minutes. Only one vane test in this study (at 2.1 m depth in VH-1) had a time to failure greater than this, but the results of this test suggest that the upper part of the vane may have been located within a sandy silt seam and so the measured strength is not considered to be reliable anyway. It should be noted that Blight’s theory also assumes that the excess pore pressures generated by insertion of the vane have completely dissipated before the start of the shear test. For standard tests in which shearing is commenced within 5 minutes of vane
insertion, excess pore pressures due to insertion disturbance will undoubtedly still be present and somewhat longer times to failure than those predicted by Blight should be acceptable while still maintaining “undrained” conditions.

D.3 ANALYSIS OF TEST DATA

D.3.1 Determining Undrained Shear Strength from FVST

The torque applied to the vane at any time during the vane shear test \( T_v(\theta) \) where \( \theta \) is the rotation of the vane) is determined from the Nilcon test record by measuring the radial distance \( M \) from the zero torque reference line to the test curve \( M \) – representing the torque applied at the top of the rods) and subtracting the radial distance from the zero line to the free slip portion of the test curve \( M_f \) – representing the torque required to overcome rod friction and turn the slip coupling), as indicated on Figure D-1. The torque applied to the vane is then calculated from the difference of these two radial measurements using a calibration factor, K, according to the following relation:

\[
T_v(\theta) = (M(\theta) - M_f) \cdot K
\]  
\( \text{(D1)} \)

A K factor of 0.975 kg·m per cm of radial distance on the test record was determined specifically for the torque head used in this study.

The undrained shear strength \( (s_u) \) is calculated using:

\[
(s_u)_{FV} = C \cdot (T_v)_{\text{max}} = (M_{\text{max}} - M_f) \cdot K \cdot C
\]  
\( \text{(D2)} \)

where the vane constant \( C \) is typically determined based on the following key assumptions (Greig, 1985):

- the peak shear stress is equal to the undrained shear strength of the soil,
- failure occurs along a cylindrical surface circumscribed by the tips of the vane flights,
- the shear stress is distributed uniformly over the surface of the failure cylinder, including the horizontal surfaces at the top and bottom of the vane, and
- the undrained shear strength of the soil is isotropic.

The remoulded shear strength is calculated in the same manner, as it is assumed that remoulding is limited to the cylindrical rupture surface along which failure occurs initially.
It should be noted here that after the first test in each boring, the free slip portion of the recorded test curves was generally much less than 15° in length, and was frequently not detectable at all. This indicates that the vane was not reversing the free-slip portion of the rotation from the previous test during pushing of the vane to the new test depth, as it was designed to do. This is likely a result of the extreme softening which probably occurs around the vane blades in these highly sensitive soils as the vane is pushed into the soil. The extremely soft remoulded soil may not provide sufficient lateral force to cause the necessary rotation of the vane. In the future, more care should be taken to rotate the rods at surface to re-establish the free slip in the coupling before pushing to the new test depth. This should also be done after remoulding the soil before carrying out a remoulded strength test so that the rod friction can be measured for this test separately. Where the free-slip segment on the recorded test curves was identifiable, the measured rod friction was generally found to be minimal after the first test in each boring, typically requiring about 0.2 to 0.25 kg·m of torque to overcome, and did not appear to increase with depth. Thus, where no free-slip segment could be identified, the vane torque $T_v$ was calculated from Equation 4.1 using a constant $M_f$ of 0.25 cm.

In a study by Roy & Leblanc (1988), $s_u$ measured with a standard Nilcon vane with 2 mm thick blades was estimated to be about 6 to 9 percent (at different sites) less than that which would be measured by an equivalent vane with zero blade thickness (based on extrapolation of strengths measured from vanes with various blade thicknesses). This is attributed to the disturbance caused by vane installation. Therefore, it seems reasonable to expect that the $s_u$ measured with the vane will slightly underestimate the true strength of the soil due to disturbance effects, but that the actual degree of disturbance will vary for different soils depending on a number of factors, including sensitivity. Where additional installation disturbance is caused by soil stuck to the blades or by the movement of inclusions, such as fibrous organics, shells or stones, the underestimation of $s_u$ may be significantly greater than that observed by Roy & Leblanc.

D.3.2 Determining Vane Rotation & Torque-Rotation Relations

The Nilcon vane test record also allows a vane torque-rotation curve to be generated. Although it is not possible to discern actual stress-strain behaviour from such curves, a comparison between curves from different depths can provide insight into relative differences in both pre-failure and post-peak deformation behaviour.
The rotational displacement $\theta$, that is recorded on the test records includes both the rotation of the vane rods at the torque head and the forward arc of the scribing needle as it moves radially inward with increasing torque $\theta$ excludes any initial rotation due to tightening of rods, rod friction, and the free slip in the slip coupling). The rotational displacement of the scribing needle, $\theta_s$ at each torque level was determined graphically and was subtracted from the corresponding $\theta$ on the scribed record to obtain the rod rotation ($\theta_r$). An example of this process is shown on Figure D-3 for determining $\theta_r$ at failure. To determine the rotation of the vane ($\theta_v$) from the measured rod rotation, it was necessary to subtract the estimated twist of the vane rods ($\theta_t$) between the torque head and the vane, where rod twist can be estimated according to:

$$\theta_t = \frac{T \cdot L}{G \cdot J}$$  \hfill (D3)

where:  
$T$ = torque in rods (above that required to overcome rod friction), at any degree of rotation
$L$ = length of rods between torque head and vane,
$G$ = shear modulus of steel = 80 GPa
$J$ = polar moment of inertia of the rods = $\pi \cdot d^4/32$
$d$ = diameter of rods = 20 mm

## D.4 TEST RESULTS

A summary of the vane test results is provided in Table D-1. Complete torque-rotation curves were generated from the FVST data at five different test depths, using the method outlined above, and are presented on Figure D-4.

In all cases, the slope of the pre-failure segment of the torque-rotation curve appears to be essentially linear over most of its length. The slope of this line will be herein termed the “vane modulus” ($M_v$), where:

$$M_v = \frac{\partial T}{\partial \theta}$$  \hfill (D4)

The vane modulus represents the stiffness of the soil under the mode of deformation imposed by the vane test, and the relative differences in $M_v$ at the different depths are inferred to be a result of differences in the large-strain shear modulus ($G$) of the soil. The values of $M_v$ (in kg m/deg
and kg·m/rad) determined from the pre-failure torque-rotation curves for vane tests carried out at 9 different depths are included in Table D-1.

The initial portion of the pre-failure torque rotation curves is typically flatter than $M_v$, as is apparent from Figure D-4, which indicates that some of the initial vane rotation occurs through softer soil, likely within the disturbed zone around the vane blades. To correct for this inferred disturbance effect, the linear segment of the pre-failure curves was extrapolated back to zero torque to obtain the excess rotation due to vane insertion disturbance. The estimates of this initial rotation for vane tests carried out at 9 different depths are included in Table D-1. The rotation due to disturbance was usually between 2 and 8 degrees, but was found to be 12 degrees in the tests at –6.8 and –7.4 m elevation, suggesting a greater extent of disturbance around the vane blades in these tests.

The curves from Figure D-4 are also shown on Figure D-5, where the torque is normalized by the peak torque measured during the test (i.e. $T/T_{\text{max}}$) and where the net vane rotation (extra rotation due to disturbance removed) is shown in radians. From this figure, it is evident that the torque-rotation relationship of the lightly overconsolidated clayey silt remains linear up to about 80% to 90% of the peak torque. In this format, it also appears that there is less variation in the slope of the linear segment of the pre-failure curves after normalizing by the maximum torque (i.e. $M_v/T_{\text{max}}$). This suggests that the variations in the large-strain shear stiffness are roughly proportional to the variations in undrained shear strength.

The net vane rotation to failure is plotted with depth on Figure D-6a, and ranges between 40 and 60 degrees, averaging about 50 degrees. This relatively large amount of rotational movement to failure suggests that the pre-failure shear deformation induced in this silty soil extends across a relatively large radial distance compared to finer-grained clays.

Values of $M_v$ are plotted with depth on Figure D-6b. The stiffness of the soil measured with the vane is about 20% higher, on average, for the tests below –7.5 m elevation than for the tests above -7.5 m. This is a similar trend to the measured undrained shear strengths which are 25% higher, on average, below –7.5 m elevation than above –7.5m.

The values of vane modulus normalized by the peak torque ($M_v/T_{\text{max}}$, as discussed above) are plotted with depth on Figure D-6c, and it appears that the values are relatively consistent, with an
average of 1.4 and a standard deviation of 0.2. The peak value of 1.8 occurs at an elevation of -7.8 m. This parameter should be analogous to the more commonly used rigidity index, $I_r$ (where $I_r = \frac{G}{s_u}$), and suggests that $I_r$ should be relatively constant throughout the depth of interest in this study.
Table D-1
Summary of Vane Test Results

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<th>Test Elevation (m)</th>
<th>Peak Strength (kPa)</th>
<th>Remould Strength (kPa)</th>
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<th>Rod Rot. to Failure (deg.)</th>
<th>Max. Torque $T_{\text{max}}$ (kg m)</th>
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</tr>
<tr>
<td>VH-1</td>
<td>6.15</td>
<td>-7.414</td>
<td>19.4</td>
<td>1.3</td>
<td>14</td>
<td>65</td>
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<td>12.7</td>
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<td>2.67</td>
<td>20</td>
<td>12.4</td>
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<tr>
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<td>-7.834</td>
<td>23.4</td>
<td>1.1</td>
<td>22</td>
<td>69</td>
<td>4.8</td>
<td>16.5</td>
<td>53</td>
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<td>59</td>
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<td>5.5</td>
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<td>13</td>
<td>77.5</td>
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<td>2.17</td>
<td>36</td>
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</tr>
</tbody>
</table>
Figure D-1: Example of FVST Torque Record

- Increasing torque record
- Increasing rotation
- Torque applied to vane
- Post-peak softening
- Peak torque
- Zero torque line
- \( M_{\text{max}} \)
- Free-slip
- \( M_r \)
Figure D-2: Typical Nilcon Vane with Slip Coupling (after Greig, 1985)
Figure D-3: Interpreting Vane Rod Rotation from Torque Record

recorded rotation = $\theta_r + \theta_s$

where:

$\theta_r$ = rotation of vane rods

$\theta_s$ = rotation due to arc of scribing needle

arc of scribing needle

increasing rotation

free-slip

$\theta_s$

$\theta_r$
Figure D-4: Field Vane Torque-Rotation Curves

![Field Vane Torque-Rotation Curves](image1)

Figure D-5: Normalized Torque-Rotation Curves

![Normalized Torque-Rotation Curves](image2)

*Corrected to exclude additional rotation due to disturbed soil around vane blades.*
Figure D-6: Variation with Depth of Vane Rotation to Failure, Vane Modulus and Vane Modulus-Strength Ratios

- **a) Net Vane Rotation to Failure (deg)**
- **b) Vane Modulus - $M_v$ (kgm/rad)**
- **c) Modulus-Strength Ratio - $M_v/T_{max}$**

Figure D-6: Variation with Depth of Vane Rotation to Failure, Vane Modulus and Vane Modulus-Strength Ratios
APPENDIX E

PIEZOCONE PENETRATION TESTING

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APPENDIX E: PIEZOCONE PENETRATION TESTING

E.1 TEST APPARATUS

Piezocone penetration tests (CPTU) were carried out at the test site using 3 different electronic piezocone penetrometers:

- the University of British Columbia standard cone (UBC-10), with 10 cm$^2$ projected cone area (3.57 cm diameter) and 10 tonne capacity (980 bar), with pore pressure elements at both U2 and U3 locations, an internal temperature sensor and a geophone located within a seismic module above U3 for measuring shear wave arrival times,
- a 2.5 tonne cone (245 bar capacity) operated by ConeTec, with 10 cm$^2$ projected cone area and pore pressure element at the U1 location, and
- a high-sensitivity cone (40 bar capacity) operated by ConeTec, with 15 cm$^2$ projected cone area (4.37 cm diameter) and pore pressure element at the U2 location.

The cone tips on all of the above penetrometers had a 60° apex angle with a 0.85 net area ratio, and the friction sleeves had equal end areas. The pore pressure elements, including porous polypropylene filters, were saturated with glycerin.

The UBC CPTU data acquisition system is set up to take readings every 2.5 cm of depth, while the ConeTec system takes measurements at 5 cm intervals. Upon stopping cone pushing, recording of the pore pressure (as well as $q_c$ and $f_s$) is started automatically and immediately. During pore pressure dissipation tests, the UBC system was set up to take readings every 0.01 sec for the first 0.40 sec, every 0.2 sec between 0.4 and 10 sec, and every 5 sec for the rest of the test. These rapid sampling rates, which were established in order to capture excess pore pressure dissipation in relatively permeable soils such as silty sand to sand, provided excellent detail of the initial pore pressure changes in the clayey silts at the Colebrook site during the first 1 to 2 minutes after stopping penetration. However, the data storage buffer is presently limited to 999 records, which is exceeded after 76 minutes at the set sampling rate described above. During the longer dissipation tests in this study, it was discovered that the storage of dissipation data for a particular sounding would become corrupted once a test exceeded 76 minutes.
In the seismic tests, shear waves were generated by hitting opposite ends of the large steel bearing pad for the front hydraulic jacks on the UBC In-Situ Testing truck, using identical swing hammers with identical drop heights, in order to generate waves with repeatable amplitudes. The centerline of the pad (along the impulse axis) was located a distance of 1.08 m from the cone rods. Since the entire weight of the truck is supported on these pads, intimate contact between the ground surface and the source of the shear waves is ensured, and therefore clear, repeatable shear waves are generated. The vertically propagating horizontal shear waves are then recorded with a geophone oriented horizontally and perpendicular to the axis of the shear wave source.

E.2 TEST PROCEDURES

The cone penetrometers were advanced at the standard rate of 2 cm/s. Baseline readings of tip resistance ($q_c$), sleeve friction ($f_s$) and pore pressure (U) were obtained with the penetrometer suspended just above the ground surface. These readings were taken before starting a sounding, and again after pulling the penetrometer out of the hole at the end of a sounding, to check for possible drifts in the electronics.

This study is only concerned with the properties of the soil between 2.5 m and about 9 to 10 m depth which will influence the behaviour of the test piles. Therefore, pilot holes were typically pushed through the upper 1 to 1.5 m of soil using a dummy cone (at CPT-5,6,7) in order to avoid damage to the electronic penetrometer due to possible debris within the fill and at the original ground surface. At CPT-2, the piezocone sounding was started from the bottom of a 2 m deep solid-stem auger hole. CPT-1 was the only sounding that was started from ground surface.

During pore pressure dissipation tests, the cone rods were clamped to prevent additional pore pressure generation due to creep-induced straining. The rods were not clamped for the test carried out at 3.35 m depth at CPT-5, however, and the clamp was mistakenly released for a period of about 8 minutes, 11 minutes into the dissipation test at 3.35 m depth at CPT-6. The CPTU dissipation tests were carried out to obtain pore pressure dissipation curves that could be compared to those observed for the test piles. Due to the great lengths of time required to obtain full dissipation of excess pore pressure (8.4 hrs to obtain 85% dissipation at the U3
location), only one test was carried to this extent. Dissipation monitoring at other elevations was generally continued until 60 to 70 percent complete at U3 (75 to 85 percent complete at U2) so that a reasonable estimate of the coefficient of horizontal consolidation, $c_h$, could be obtained. This allows the time plots to be normalized and gives an indication of the variation in $q_v$ with depth, which is particularly important when comparing dissipation records from the various depths around the piles.

Seismic tests were carried out at 1 m intervals of depth in CPT-5&7 and at 0.5 m intervals in CPT-6. During each test, shear waves were generated and recorded four separate times (twice for each direction of the initial source impulse to ensure repeatability) during a single break in penetration. By hitting opposite ends of the bearing pad, shear waves are generated with opposite polarities which can be superimposed to determine the time for the first crossover of the polarized waves. This point has been found to provide a consistent reference point for the shear wave arrival times for each test (Robertson et al., 1986). Seismic testing at each depth was usually started within about 1 minute of stopping cone penetration. At 5.0 m tip depth in CPT-6, however, seismic testing was first carried out 20 minutes into the dissipation test and again at the end of the dissipation test, 8.4 hours after stopping penetration.

E.3 ANALYSIS OF TEST DATA AND RESULTS

E.3.1 Tip Resistance, Sleeve Friction and Penetration Pore Pressure

The general methods and sources of potential error in calculating corrected tip resistance ($q_T$), sleeve friction ($f_s$) and pore pressure ($u$) from the CPTU measurements are discussed in detail in Campanella et al. (1995) and by Gillespie (1990). The following discussion is limited to particular aspects that had significant implications on the interpretation of the CPTU data obtained during this study.

The tip resistance ($q_c$) measured in the soft Colebrook silt and clay typically ranged between 2.5 and 5 bar, which is only 0.25% to 0.5% of full-scale output for the UBC cone and 1% to 2% of full-scale for the ConeTec 2.5 tonne cone. Therefore, any sources of error in the measurements of $q_c$ using these cones can have a significant effect on the calculated values of
q_T used to interpret soil parameters. Using the high-sensitivity 40 bar cone, the measured values of q_c are 6% to 13% of full-scale, and therefore the q_c and q_T profiles from CPT-2 should be the most accurate.

The recorded friction sleeve measurements at the site were also very low, generally less than 0.05 bar, and hence are at the limits of instrument accuracy. The resolution of the friction sleeve measurements was not adequate at CPT-1&2 since f_s was only recorded with 0.01 bar resolution on readings that were typically 0.01 to 0.03 bar. The friction sleeve measurements at CPT-5,6,7 was recorded with 0.001 bar resolution. However, the data from CPT-5&6 is not considered to be reliable due to uncertainties about the correction for temperature effects in these soundings, the magnitude of the potential correction being large relative to the recorded values.

The output of the electronics in the cones is known to be temperature sensitive and errors can result from calculating q_c, f_s and u using baselines taken at a temperature which is significantly different from the cone temperature during the sounding. From temperature readings obtained with the UBC cone and from temperature sensors on the piles, it is known that the ground temperature between 2 m and 10 m depth is typically between 11 and 12 degrees C. For CPT-5&6, which were carried out in July, baseline readings were taken at cone temperatures ranging from 17 to 21 deg C. Differences in baseline readings at different temperatures were observed to be significant for the q_c and U2 measurements. The baseline readings were plotted against temperature and shifts of 0.08 bar/°C for q_c and 0.06 m(H_2O)/°C for U2 were determined over the 17 to 21 deg C range. If extrapolated to the equilibrium temperature in the ground of about 11°C, estimated shifts in baselines of about 0.8 bar and 0.6 m would be expected over a 10 deg differential between the temperature for the initial baseline readings and that during most of the sounding. For q_c in particular, the correction for these temperature shifts is large relative to the measured values during the soundings. Temperature data was not available for CPT-1,2,7, but these soundings were carried out at the end of March and beginning of April when the air temperature was much closer to the ground temperature, and so temperature errors are expected to be much less.
Profiles of measured tip resistance, $q_c$ (with CPT-5 & CPT-6 corrected for temperature effects), and net tip pressure, $q_T - \sigma_v$, from all five CPTU probings carried out at the test site during this study are presented on Figure E-1a and E-1b, respectively. It should be noted that the $q_T - \sigma_v$ profile for CPT-1 on Figure E-1b tends to plot to the left of all the other cone tests below –4.6 m elevation (below the crust). This may be due to error in the calculated values of $q_T$ from CPT-1 since U2 measurements from CPT-7 (located 6 m away from CPT-1) had to be used to correct for unequal pore pressure effects, since there were no U2 pore pressure measurements in CPT-1.

Profiles of excess pore pressure, $\Delta u$, measured at the U1, U2 and U3 filter positions on the cone probe (midway up the face of the cone, behind the shoulder of the cone, and behind the friction sleeve, respectively) during steady penetration are plotted on Figure E-2. For the profiles from CPT-5,6,7, the pore pressures that were measured immediately after resuming penetration following breaks to carry out seismic tests and pore pressure dissipation tests, have been omitted. During such breaks, the soil that had already been disturbed by cone penetration would have been subjected to some volumetric changes due to excess pore pressure drainage. Upon resuming penetration, the pore pressures measured within the soil that had been disturbed by the cone and partially reconsolidated would not be representative of $\Delta u_i$ generated during penetration in undisturbed soil.

**E.3.2 Correlation of CPTU Parameters to Undrained Shear Strength**

For the purposes of this study, it was desired to have continuous profiles of undrained shear strength throughout the thickness of soil influenced by the test piles. Therefore, depth-specific cone-strength correlation factors (N) were derived which relate $s_u$ from the field vane to various parameters determined from the CPTU measurements. The various methods of calculating $s_u$ from different cone parameters, along with the advantages and disadvantages of each method, are discussed in Campanella et al. (1995), and have been evaluated for different fine-grained soil sites in the Lower Fraser Valley by Greig (1985).
Calculating Cone-Strength Factors

Two different cone factors were used in this study, one that is based on cone bearing \( N_{KT} \) and one that is based on pore pressure measurements \( N_{\Delta u} \), which are defined as follows:

\[
N_{KT} = \frac{q_T - \sigma_{vo}}{s_u} \tag{E1}
\]

\[
N_{\Delta u} = \frac{\Delta u}{s_u} \tag{E2}
\]

where \( \sigma_{vo} \) is the total overburden pressure and \( \Delta u \) is the excess pore pressure generated by the penetration of the cone.

\( N_{KT} \) and \( N_{\Delta u} \) factors were calculated for CPT-1,2,7 at each vane test depth (from both VH-1 and VH-2) using \( q_T - \sigma_{vo} \) and \( \Delta u \) values averaged over appropriate depth ranges. For \( N_{\Delta u} \), values of \( \Delta u \) were averaged over a 20 cm thickness of soil centered around the 17 cm long vane blade. The zone of influence of the cone tip extends approximately 5 to 10 cone diameters above and below the tip, depending on the stiffness of the soil (Campanella et al., 1995). Therefore, \( N_{KT} \) factors were calculated from \( q_T - \sigma_{vo} \) values averaged over different thickness intervals from 20 cm ( +/- 3 cone diameters from center of vane for 10 cm\(^2\) cones) to 20 cone diameters ( +/- 35 cm from vane center for 10 cm\(^2\) cones and +/- 45 cm for 15 cm\(^2\) cone). The \( N_{KT} \) profiles obtained from the largest averaging interval were found to have the least amount of scatter, and so the \( q_T - \sigma_{vo} \) values were averaged over a zone extending to 10 cone diameters above and below the center of the vane.

Cone-Strength Factors for Colebrook Site

The \( N_{KT} \) and \( N_{\Delta u} \) factors for each of CPT-1,2,7, which are calculated from \( s_u \) measured at each vane test elevation, are plotted with depth on Figures E-3 and E-4, respectively. For each set of cone data, the larger darker symbols represent \( N_{KT} \) and \( N_{\Delta u} \) factors calculated from vane tests carried out in borings located relatively close to the cone hole (eg. CPT-1/VH-1 correlations), while the smaller lighter symbols represent cone-strength factors from cross-site correlations (eg. CPT-1/VH-2). The calculated \( N_{KT} \) factors are consistent with average values reported by Greig (1985) for other fine-grained soil sites around the
Lower Mainland of B.C. The $N_{KT}$ factors calculated from CPT-1 were generally lower than those from CPT-2&7. This is due to the lower values of $q_T$ calculated for CPT-1 compared to the range of $q_T$ from the rest of the cone holes, which may be due to error associated with correcting the measured $q_c$ values using U2 pore pressures from an adjacent hole, or could be a result of an error in the calibration factor that was used by ConeTec. The calculated $N_{\Delta u}$ factors from the Colebrook site are less than the site-averages reported by Greig (1985), particularly for $\Delta u$ measured at the U2 location. However, the geologic conditions within the depth of interest at the Colebrook site are different than those at any of the sites assessed by Greig.

It can be seen from Figures E-3 & E-4 that $N_{KT}$ and $N_{\Delta u}$ are not constant with depth. This is due to the fact that cone-strength factors are not unique parameters since both $q_T$ and $\Delta u$ are influenced by properties other than just shear strength, such as stress history, stiffness and sensitivity (Campanella et al., 1995). For example, the higher $N_{KT}$ values between –7 and -9 m elevation may be due to a localized increase in lateral stress and/or an increased large-strain shear strength and/or an increase in rigidity ($G/s_u$), all of which will increase $q_T$ relative to $s_u$. Equation E2 is particularly sensitive to changes in the overconsolidation ratio (OCR) since the excess pore pressures measured along the shaft of the penetrometer tend to decrease substantially with increasing OCR (due to decreasing $\Delta u_{shear}$) while $s_u$ will tend to increase with OCR. The increase in $N_{\Delta u}$ observed from the U2-based data on Figure E-2 correlates well with decrease in OCR, and vice-versa. This is particularly evident through the crust between –4 and –5 m elevation where the soil is moderately over-consolidated and negative $\Delta u_{shear}$ is inferred.

**Establishing Continuous Profiles of $N_{KT}$ and $N_{\Delta u}$ with Depth**

Continuous profiles of $N_{KT}$ and $N_{\Delta u}$ with depth for each cone hole were subjectively fitted to the calculated discrete values at the various vane test elevations, as shown on Figures E-3 & E-4, respectively, based on personal judgement of the quality of each cone-vane data pairing. For example, better correlation was expected between the cone data from CPT-1&7 and the vane strengths from nearby VH-1, than for the strengths measured from VH-2 on the
opposite side of the site. Similarly, cone factors calculated for CPT-2 from VH-2 strengths were given greater priority for line fitting than VH-1 strengths.

The apparent peak in $N_{KT}$ at about $-7.4$ m is due to the fact that the cone tip resistance increases significantly between about $-7$ and $-7.5$ m elevation, while the increase in undrained strength measured by the vane seems to occur between $-7.4$ and $-8.4$ m. It is possible that the $s_u$ measured by the vane at $-6.8$ and $-7.4$ m elevation have been slightly underestimated due to disturbance effects since larger initial vane rotations were measured at both of these depths. Considering this possibility, the increase in the interpreted $N_{KT}$ profiles was assumed to occur between $-6.4$ and $-7.8$ m elevation, rather than between $-6.4$ and $-7.4$ m as suggested by the calculated $N_{KT}$ factors at $-6.8$ and $-7.4$ m.

It was observed that the shape of the $N_{\Delta u}$ profiles for both U1 and U2 pore pressure measurements closely resembled the shape of the corresponding profiles of pore pressure ratio, $B_q = \Delta u/(q_T - \sigma_{vo})$. The advantage to using $B_q$ to estimate $N_{\Delta u}$ is that the substantial influence of OCR on $\Delta u$ will affect both parameters and the remaining variability in the correlation between $s_u$ and $\Delta u/N_{\Delta u}$ will again be due to variations in the $s_u$ vs. $(q_T - \sigma_{vo})$ relation. Therefore, values of $N_{\Delta u}$ calculated at each vane test elevation were plotted against the corresponding $B_q$ ratios (averaged over a 20 cm thickness of soil centered around the vane blade), as shown on Figure E-5, and reasonably good, linear correlations were found, at least for the U2 data. The $\Delta u_2$ data from CPT-7 includes data between $-11$ and $-17$ m elevation (where $N_{\Delta u}$ was calculated from vane strengths obtained by Crawford & Campanella, 1991) since larger values of both $B_q$ and $N_{\Delta u}$ were calculated from the data below $-11$ m. It should be noted that the ratio of $N_{\Delta u}/B_q$ is equivalent to $N_{KT}$, and therefore, the slope of the best-fit lines on Figure E-5 is the average $N_{KT}$ over the entire range of depths. The $N_{\Delta u}/B_q$ ratios determined for each cone hole, which are listed in Table E-1, were then used to objectively calculate continuous profiles of $N_{\Delta u}$ from the $B_q$ profiles. The $B_q$ profiles were smoothed for this purpose using a running average of $B_q$ values within +/-5 cm of the elevation for which $N_{\Delta u}$ was calculated. The resulting $B_q$-based $N_{\Delta u}$ profiles with depth are shown on Figure E-4.
Table E-1
\(N_{\Delta u}/B_q\) Ratios

<table>
<thead>
<tr>
<th>Cone Hole (Piezo Filter Location)</th>
<th>(N_{\Delta u}/B_q) (= avg (N_{KT}))</th>
<th>Coefficient of Correlation ((r^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT-1 (U1)</td>
<td>9.6</td>
<td>0.61</td>
</tr>
<tr>
<td>CPT-2 (U2)</td>
<td>12.6</td>
<td>0.79</td>
</tr>
<tr>
<td>CPT-7 (U2) &lt;10 m depth</td>
<td>11.0</td>
<td>0.92</td>
</tr>
<tr>
<td>CPT-7 (U2) &lt;16 m depth</td>
<td>12.6</td>
<td>0.88</td>
</tr>
</tbody>
</table>

For CPT-2 and CPT-7, the \(B_q\)-based \(N_{\Delta u}\) profiles were found to give a better fit of the \(N_{\Delta u}\) factors calculated from the vane strengths than the manually fitted profiles, and were therefore used to calculate the \(N_{\Delta u}\)-based \(s_u\) profiles for CPT-2 and CPT-7. For CPT-1, where \(\Delta u\) was measured on the cone face, a better fit was obtained using the manually fitted \(N_{\Delta u}\) profile, and so this profile was used instead of the \(B_q\)-based profile to calculate the \(N_{\Delta u}\)-based \(s_u\) profile for CPT-1. The undrained shear strength profiles calculated using the \(N_{KT}\) and \(N_{\Delta u}\) profiles were also smoothed using running averages of \((q_T - \sigma_{vo})\) and \(\Delta u\), respectively, within +/-5 cm of the elevation for which \(s_u\) was calculated. The undrained shear strengths calculated from the cone data using the interpreted \(N_{KT}\) and \(N_{\Delta u}\) profiles were found to be generally in good agreement.

This study demonstrates the importance of determining site-specific, cone-specific and depth-specific cone-strength factors. Applying a regionally applicable average cone factor to the cone data over a range of depths can result in significant error in the calculated strengths, if there are variations in the soil properties with depth. Based on the results of this study, the following procedure is recommended for estimating undrained shear strength from piezocone data:

1. Using reliable measures of \(s_u\) (eg. from FVST or consolidated undrained laboratory strength tests on undisturbed samples) from several depths, determine \(N_{\Delta u}\) factors for each depth at which the strength was measured using \(\Delta u\) measured at the U2 location behind the shoulder of the cone.
2. Correlate the $N_{\Delta u}$ factors to $B_q$ ratios from the same depths. The ratio of $N_{\Delta u}/B_q$ that best fits the available data represents the average $N_{KT}$ over the same range of depths.

3. The ratio of $N_{\Delta u}/B_q$ can be used to obtain a continuous profile of $N_{\Delta u}$ with depth based on the continuous $B_q$ profile.

4. Use the calculated $N_{\Delta u}$ values along with the measured $\Delta u$ from the cone to obtain a continuous profile of $s_u$.

This procedure is recommended over the $N_{KT}$ method for the following reasons:

- the measurement of $\Delta u$ in soft fine-grained soils is inherently more accurate than the measure of $q_T$ (large $\Delta u$ vs. very small $q_T$),
- if properly saturated, the pore pressure element on the cone only responds to the pore pressure within the soil that is in direct contact with the filter, while the cone tip resistance is influence by the properties of a large volume of soil surrounding the cone; as a result, there is some uncertainty about what is the most appropriate depth interval over which $(q_T - \sigma_{vo})$ should be averaged when calculating $N_{KT}$,
- determining the variation in $N_{KT}$ with depth requires a greater number of reliable $s_u$ measurements; and then manually fitting a representative profile can be difficult and time consuming if variations with depth are observed (it may be difficult to differentiate between data scatter and true variations with depth.

### E.3.3 Overconsolidation Ratio

Profiles of OCR (where OCR is herein defined as the ratio of vertical yield stress, $\sigma'_vy$, to in-situ vertical stress, $\sigma'_vo$) were estimated from the CPTu data using the following methods:

1) The pore pressure difference (PPD) method proposed by Sully et al. (1990):

$$OCR = 0.50 + 1.50 \cdot PPD$$  \hspace{1cm} (E3a)

and $PPD = (u_1 - u_2)/u_0$  \hspace{1cm} (E3b)
where \( u_1 \) is the total pore pressure measured at the U1 location (at CPT-1 in this study), \( u_2 \) is the pore pressure measured at U2 location (at CPT-7, located approx. 6 m away from CPT-1 or at CPT-2, located approx. 17 m from CPT-1), and \( u_i \) is the in-situ pore pressure before cone penetration.

2) The ratio of \( s_u/\sigma'_{vo} \) to \( (s_u/\sigma'_{vo})_{NC} \) for the normally consolidated soil which was correlated to OCR by Schmertmann (1978) based on lab test results, and can be represented mathematically using the general relation

\[
OCR = \left[\frac{(s_u/\sigma'_{vo})}{(s_u/\sigma'_{vo})_{NC}}\right]^b
\]  

(E4a)

where the value of the exponent, \( b \), was proposed by Schmertmann (1978) using the following statistical correlation:

\[
b = 1.13 + 0.04\cdot\frac{(s_u/\sigma'_{vo})}{(s_u/\sigma'_{vo})_{NC}}
\]  

(E4b)

The values of \( s_u/\sigma'_{vo} \) used in this method, and in the following method, were calculated for each of CPT-1,2,7 from the average of the two undrained shear strength profiles generated from cone data using the \( N_{KT} \) and \( N_{\Delta u} \) profiles.

3) Equation E4a, where \( b = 1/m \), for which Ladd et al. (1977) suggest that \( m \) should be estimated as 0.8, but Wroth (1984) suggests that \( m \) is actually the plastic volumetric strain ratio, \( \Lambda = (\lambda - \kappa) / \lambda \), from critical state soil mechanics theory. \( \Lambda \) can be estimated as \( 1 - C_o/C_c \) using odometer test data (Wood, 1990). An average value of \( \Lambda = 0.95 \) for this site was assumed based on the consolidation test results published in Crawford et al. (1994), which allows Equation E4a to be expressed as:

\[
OCR = \left[\frac{(s_u/\sigma'_{vo})}{(s_u/\sigma'_{vo})_{NC}}\right]^{1.05}
\]  

(E5)

4) The theoretical equations derived by Mayne (1991) from cavity expansion theory and critical state soil mechanics theory using \( q_T - u_1 \) and \( q_T - u_2 \):

\[
\text{OCR} = \left[\frac{(s_u/\sigma'_{vo})}{(s_u/\sigma'_{vo})_{NC}}\right]^{1.05}
\]
$$OCR = 2\left[\frac{(q_{T} - u_{I})/\sigma'_{vo} - 1}{1.95M}\right]^{1/\Lambda} \tag{E6a}$$

$$OCR = 2\left[\frac{(q_{T} - u_{2})/(\sigma'_{vo} (1+1.95M))}{1.95M}\right]^{1/\Lambda} \tag{E6b}$$

where $M$ is the slope of the critical state line and can be expressed as

$$M = \frac{6\sin\phi'/(3-\sin\phi')}{(3-\sin\phi')} = 1.42 \text{ for } \phi' = 35^\circ \tag{E6c}$$

5) The simplified statistical correlations of $q_{T} - u_{I}$ and $q_{T} - u_{2}$ normalized by $\sigma'_{vo}$ proposed by Chen & Mayne (1995):

$$OCR = 0.75(q_{T} - u_{I})/\sigma'_{vo} \tag{E7a}$$

$$OCR = 0.50(q_{T} - u_{2})/\sigma'_{vo} \tag{E7b}$$

Profiles of OCR were generated using each of the 5 methods described above along with the data from each of CPT-1,2,7, as shown on Figures E-6a to E-6c, respectively. The PPD method (Method 1 by Sully et al., 1990) gives estimates of OCR which were typically found to be considerably lower than the other methods above -6.5 m elevation, and are less than unity in a couple of locations. Below -6.5 m elevation, the PPD estimates were found to fluctuate between the high and low extremes of the range of estimates from the other methods. Also, the relation proposed by Mayne (1991) using $q_{T} - u_{I}$ (Method 5, Eq. E6a) was found to give estimates of OCR which are considerably lower than the other methods below about -6 m elevation, and are frequently less than unity, which is not reasonable. In general, the OCR estimates for CPT-2&7 (U2 pore pressure measurements) showed more consistency between the various methods than was obtained using the CPT-1 data where pore pressures were measured at the U1 location. This is consistent with the poorer correlations which were found in this study between U1-based parameters such as $B_{q1}$ and $N_{du1}$ and the average OCR determined using all of the various interpretation methods, as compared to those obtained using the equivalent U2-based parameters. This is due to the fact that $\Delta u$ measured on the face of the cone is dominated by the increase in mean normal stress and is less affected by changes in $\Delta u_{shear}$ than at the filter locations along the shaft behind the cone. The effects of OCR on $\Delta u_{shear}$ appear to be much more pronounced than on $\Delta u_{oct}$. The
apparent difficulties in estimating OCR using U1 measurements appears to be most pronounced within the organic deposit above the upper crust (above –4 m elevation).

E.3.4 In-Situ Lateral Stress

Profiles of the in-situ coefficient of lateral earth pressure, \( K_o = \sigma'_{ho}/\sigma'_{vo} \), were estimated using the following methods:

- **The PPSV method** proposed by Sully and Campanella (1991):

  \[
  K_o = 0.5 + 0.11 \cdot \text{PPSV} \quad (E8a)
  \]

  and \( \text{PPSV} = (u_1 - u_2)/\sigma'_{vo} \quad (E8b) \)

  where \( u_1 \) was measured at CPT-1 while \( u_2 \) measurements from both CPT-7 and CPT-2 were used to assess consistency.

- **From OCR and \( \phi' \)** according to Mayne & Kulhawy (1982):

  \[
  K_o = (1 - \sin \phi') \cdot \text{OCR}^{\sin \phi'} \quad (E9)
  \]

  where \( \phi' = 35^\circ \) was assumed and values of OCR at each depth were obtained using an average of the upper and lower values from the median range for each of CPT-1,2,7. However, Equation E9 was derived from laboratory tests in which the overconsolidation resulted from expanding samples to a lower stress state after consolidating to a maximum effective stress (true mechanical overconsolidation). Therefore, this expression may not be strictly applicable if the OCR at this site is significantly affected by quasi-preconsolidation effects.

Profiles of the coefficient of lateral earth pressure at rest, \( K_o \), interpreted from the CPTU data are shown on Figure E-7. Below –6.5 m elevation, the profiles estimated using the PPSV method (Sully et al., 1990), with the CPT-1,2 and CPT-1,7 pore pressure combinations, agree relatively well with those estimated from the average OCR profiles from each of CPT-1,2,7 using the OCR-\( \phi' \) relation (Mayne & Kulhawy, 1982). Above –6.5 m, however, the PPSV estimates are significantly less than the OCR-\( \phi' \) estimates. It is possible that the \( u_1 - u_2 \) pore
pressure difference measured in the zone above -6.5 m does not fit the trend of the data in the database used by Sully et al. (1990) to determine the PPD and PPSV correlations.

Similar trends of $K_0$ with depth are predicted by both the PPSV and OCR-$\phi'$ methods. Both methods indicate a clear peak in $K_0$ within the moderately overconsolidated upper crust, as well as a second, more subtle peak between -7.2 and -7.65 m elevation which may correspond to a lower, weakly desiccated crust, as was inferred from the friction sleeve data. It should also be noted that the shape of the $K_0$ profile below -6.5 m elevation is similar to that of the $N_{KT}$ profiles. This suggests that the observed variation in $N_{KT}$ through this zone at CPT-2&7 is a result of increased lateral stress which will tend to increase $q_T$ but is not accounted for in the net tip resistance when expressed as $q_T - \sigma_{vo}$.

### E.3.5 Shear Wave Velocity

Shear wave velocities ($V_s$) were calculated using the pseudo interval technique described by Gillespie (1990) and validated by Rice (1984). A schematic of the interval technique is provided on Figure E-8. By subtracting successive shear wave travel times, equal length portions of the shear wave paths which travel through similar soil conditions tend to be negated and the increment in travel time corresponds to the wave velocity between the successive depths of measurement.

In soft fine-grained soils, the effects of installation disturbance around the penetrometer will tend to result in a reduction in $V_s$ within an annulus of soil near the probe. It can be shown that the minimum shear wave travel time will correspond to the path that has the shortest segment within the disturbed annulus, which corresponds to a path that is close to horizontal through the disturbed zone. The portion of the wave path that travels through the disturbed zone would then tend to be of similar length at successive receiver depths. Therefore, the effects of the reduced $V_s$ within the disturbed annulus will tend to be negated when the successive travel times are subtracted (provided that the conditions within the disturbed zone are similar between the successive depths). As a result, the increment in shear wave travel time will tend to correspond to the portion of the wave path between the two test depths and outside the disturbed zone.
The increments of travel time were calculated using total measured travel times from the source to a common marker point on successive wave traces. Four options for a marker point are indicated on Figure E-9: the first arrival of the wave, the first peak, the first cross-over of the polarized waves, and the second peak. Gillespie (1990) found that the first cross-over point provided a repeatable marker provided that a repeatable wave amplitude was ensured, as is the case using the bearing pad and swing hammers on the UBC in-situ testing truck. The first arrival time has also been found to be a very repeatable marker, but is harder to identify, particularly when there is signal noise. The first arrival and the peak of the first wave were found to be badly distorted by signal noise at shallow depths during the seismic testing carried out in this study. The second peak, which had the largest amplitude (approximately double that of the first peak), was found to be a clearer marker. Thus, shear wave profiles were calculated using both the first cross-over and second peak marker points for comparison.

The profiles of the shear wave velocity, \( V_s \), calculated using the pseudo-interval technique from measurements at CPT-5,6,7 are presented on Figure E-10. Values of \( V_s \) determined using both the first cross-over and second peak (peak amplitude) markers are compared on Figure E-10. For CPT-5&7, in which 1 m intervals were used, the two different marker points gave similar magnitudes and similar trends with depth. For CPT-6, in which 0.5 m intervals were used, the peak marker tended to result in larger variations in \( V_s \) from one depth to the next. Therefore, only the values of \( V_s \) determined using the traditional first cross-over marker were used to calculate \( G_{\text{max}} \).

The elevations of the cone tip during dissipation tests, which were carried out in the same hole as the seismic tests, are also shown on Figure E-10, along with the duration of the tests. Seismic tests were carried out at \(-6.15\) m elevation at 20 minutes after stopping penetration and again at the end of the dissipation test, 8.4 hours after stopping penetration. During this time, \( \Delta u/\sigma'_\text{vo} \) at the U3 filter location, just below the geophone location, decreased from about 1.25 (approx. 40% dissipation) to about 0.3 (approx. 85% dissipation). The corresponding shear wave travel time to the first cross-over decreased from 63.0 to 61.7 ms, indicating an increase in \( V_s \) and \( G_{\text{max}} \) due to the dissipation of excess pore pressure around the shaft of the cone. This is to be expected since researches have shown that \( G_{\text{max}} \) increases
with increasing effective stress and with decreasing void ratio (eg. Hardin & Drnevich, 1972).

The above observation points out the need for caution when interpreting profiles of $V_s$ or $G_{\text{max}}$ determined from shear wave travel times which have been influenced by varying degrees of dissipation within the disturbed zone around the penetrometer. Fortunately, the length of the travel path through the disturbed zone, in which significant changes in the soil properties occur during consolidation, should be small in comparison to the incremental path length through the undisturbed zone, particularly for tests carried out at 1 m intervals. Therefore, the effects of changes in the soil properties within the disturbed zone on the values of $V_s$ and $G_{\text{max}}$ calculated using the interval technique should be relatively small, although they cannot be ignored completely. The general agreement in the $V_s$ profiles below -7 m, between CPT-6&7, in which no dissipation tests were carried out, and CPT-5, in which dissipation tests were carried out, suggests that this is the case.

The greatest variation between the different test locations seems to occur in the vicinity of the upper crust and just above the lower stiffer zone. However, this can be explained by the different spacing (0.5 vs 1 m) and elevations of the seismic tests, which results in different average values being obtained depending on the relative thickness of the two distinctly different zones of soil intersected by the test interval. At both of these locations, abrupt changes in $q_T$, $f_s$, $s_u$, OCR and $K_o$ have been indicated in the preceding sections, and therefore, different interval averages can be expected for different interval locations.

**E.3.6 Small-Strain Shear Modulus**

The small strain shear modulus, $G_o$ or $G_{\text{max}}$, can be calculated from the shear wave velocity, according to:

$$G_{\text{max}} = \rho V_s^2$$  \hspace{1cm} (E10)

where $\rho$ is the mass density of the soil, which was calculated from moisture content measurements. Since $V_s$ is calculated using an interval technique, each value corresponds to an average across the interval. Therefore, average densities were used for each interval to
calculate average values of $G_{\text{max}}$. For CPT-5&7, 1.0 m intervals of depth were used, while 0.5 m intervals were used for CPT-6.

Numerous researchers have shown the dependence of $G_{\text{max}}$ on the in-situ effective stress and $G_{\text{max}}$ can be expressed in general terms as:

$$G_{\text{max}}/p_a = A \cdot F(e) \cdot (\sigma'/p_a)^m$$  \hspace{1cm} (E11)

where:  $p_a$ = atmospheric pressure, used to make the expression unitless,

$A$ = constant which depends on size and shape of soil particles

$F(e)$ = some function of the void ratio, $e$, which depends on the soil type and stress history, and

$\sigma'$ = mean effective stress, $\sigma'_m$, which should be the average (geometric mean is preferred) of the stresses in the direction of wave propagation ($= \sigma'_v$) and in the direction of oscillation ($\sigma'_h$), but not in the third direction (Stokoe et al., 1986).

From laboratory tests, the exponent, $m$, has been determined to be typically about 0.5 for most soils. However, these tests neglect the important effects of aging on $G_{\text{max}}$ and higher values of $m$ have been observed for fine-grained soils in the field: eg. for $\sigma' = \sigma'_v$, $m = 0.82$ for marine clayey silt at Onsøy, Norway, and $m = 0.76$ for normally consolidated clayey silt at McDonald Farm in Richmond, B.C. (Gillespie, 1990).

Correlations between $G_{\text{max}}/p_a$ and both $\sigma'_v/p_a$ and $\sigma'_m/p_a$ (calculated using the $K_o$ estimates from Figure E-7) in the moderately to lightly overconsolidated Colebrook clayey silt between –4.1 and -11.4 m elevation are provided on Figure E-11. Based on these correlations, average values for $m$ of 0.8 and 1.0 were found for $\sigma'_v$ and $\sigma'_m$, respectively. The profiles of $G_{\text{max}}/p_a/(\sigma'/p_a)^m$ determined using both $\sigma' = \sigma'_v$ and $\sigma' = \sigma'_m$ are shown on Figure E-12. The two sets of profiles were found to be very similar in shape, although less variation with depth was observed using $\sigma'_m$. The normalized profiles for CPT-5&7 (tests carried out at 1.0 m intervals) are almost constant with depth. For CPT-6, where tests were carried out at
0.5 m intervals, greater detail is evident and the normalized profile resembles that of the $s_u$ and $q_T - \sigma_{vo}$ profiles.

E.3.7 Estimating Rigidity Index from $N_{KT}$

Teh & Houlsby (1991) present a solution of $N_{KT}$ in terms of rigidity index, $I_r = G/s_u$, which is based on their finite element solution of the Strain Path Method. Therefore, if the $N_{KT}$ for a site is known from CPT and $s_u$ measurements, $I_r$ could be back-calculated using this theoretical solution. In the Teh & Houlsby study, the effects of the difference between the initial vertical and horizontal stresses in the soil on the cone factor were assessed. This is important since the definition of $N_{KT}$ (from Equation E1) does not include the in-situ horizontal stress, $\sigma_{ho}$, which was found to have an even greater influence on $q_T$ than $\sigma_{vo}$. The following closed-form equation, which also includes the effects of different degrees of shaft and tip roughness, was derived by Teh & Houlsby to approximate the results of their finite element analyses:

$$N_{KT} = \frac{4}{3} \left(1 + \ln(I)\right) \left(1.25 + \frac{I}{2000}\right) - 1.8\Delta + 2.4\alpha_f - 0.2\alpha_s$$  \hspace{1cm} (E12)

where:  
$\Delta = (\sigma_{vo} - \sigma_{ho})/2s_u$ (between -1 and 1)  
$\alpha_f$ is the cone roughness factor (between 0 and 1)  
$\alpha_s$ is the shaft roughness factor (between 0 and 1)

The $N_{KT}$ values calculated at each vane test elevation from the $s_u$ and $q_T$ measurements during this study were substituted into Equation E12, along with estimated values of $\sigma_{ho}$ based on the estimated $K_o$ values. Both cone roughness factors were assumed to be equal to zero since the shear stresses on the cone were expected to be negligible based on the extremely low friction sleeve measurements. The value of $I_r$ back-calculated using this method is highly sensitive to the $N_{KT}$ value. Therefore, any errors in the measured values of $q_T$ and $s_u$ or the estimated $K_o$ values, or any influences on $N_{KT}$ which are not accounted for in Equation E12, such as sensitivity, can result in large variations in the back-calculated $I_r$ values. For example, the strain path method used by Teh & Houlsby to develop the above solution uses elastic-plastic constitutive laws to derive the deviator stresses in the soil.
Therefore, it is expected that Equation E12 will not consider the potential effects of large-strain strain-softening behaviour on $q_T$ and therefore $N_{KT}$. There is also a question of whether the $N_{KT}$ calculated from the $s_u$ measured with the field vane is strictly applicable to the above solution.

### E.3.8 $B_q$ – OCR Correlation

The $B_q$ parameters that were calculated from the CPTU measurements at CPT-1,2,7 are plotted against the interpreted OCR at the same depths on Figure E-13. For a given depth and cone probing, the OCR value plotted on Figure E-13 corresponds to the mean of the 3 median values from the 5 different methods described in Section E.3.3.

At the U2 filter location, $B_q$ appears to decrease linearly with the logarithm of OCR (above – 12 m elevation and between OCR = 1 and 6), and the trends are very similar for the data from CPT-2 and CPT-7. A similar trend was observed at the U1 location (CPT-1), but the relationship is not quite linear (smaller decrease in $B_q$ with increasing OCR) and there is slightly greater scatter to the data.

### E.3.9 CPTU Dissipation Tests

A summary of the dissipation tests carried out at the Colebrook test site during this study is provided below.

<table>
<thead>
<tr>
<th>Test Location</th>
<th>Tip Depth (m)</th>
<th>Tip Elevation (m)</th>
<th>Duration of Test (min)</th>
</tr>
</thead>
<tbody>
<tr>
<td>CPT-5</td>
<td>3.35</td>
<td>-4.69</td>
<td>26</td>
</tr>
<tr>
<td>CPT-5</td>
<td>4.75</td>
<td>-6.08</td>
<td>51</td>
</tr>
<tr>
<td>CPT-5</td>
<td>5.55</td>
<td>-6.88</td>
<td>29</td>
</tr>
<tr>
<td>CPT-5</td>
<td>6.45</td>
<td>-7.78</td>
<td>71</td>
</tr>
<tr>
<td>CPT-5</td>
<td>7.60</td>
<td>-8.93</td>
<td>56</td>
</tr>
<tr>
<td>CPT-5</td>
<td>8.50</td>
<td>-9.83</td>
<td>66</td>
</tr>
<tr>
<td>CPT-6</td>
<td>3.35</td>
<td>-4.69</td>
<td>76</td>
</tr>
<tr>
<td>CPT-6</td>
<td>5.00</td>
<td>-6.34</td>
<td>504</td>
</tr>
</tbody>
</table>
E.4 EXTRAPOLATING DISSIPATION DATA TO LARGER TIMES USING THEORETICAL SOLUTIONS

A curve matching procedure is described below, which was used to extrapolate the measured CPTU dissipation data to at least 90% dissipation using published solutions when the shape of the theoretical dissipation curve does not match that of the measured dissipation data. An example of a CPT(U3) dissipation test from –6.3 m elevation (log-time-corrected for initial redistribution effects), for up to 40% excess pore pressure remaining (i.e. 60% dissipation), is shown by the data points on Figure E-14.

A theoretical curve for the U3 position is plotted on Figure E-14 (dashed line) by using the Teh & Houlsby (1991) normalized curve, which is based on their modified time factor, $T^* = \frac{c_h}{R^2 I_r^{1/2}}$, with $R = 1.78$ cm for the cone probe and best estimates of $I_r = 350$ and $c_h = 0.019$ cm$^2$/s for the Colebrook soil. The resulting curve of $\frac{\Delta u(t)}{\Delta u_o}$ vs time (t) does not match the measured CPT(U3) data. This is because the assumptions used to generate the theoretical dissipation curve are not the same as the soil conditions governing the observed dissipation process. Instead of using a constant $c_h$ for the entire curve, an artificial set of values (denoted by $\chi$) are substituted for $c_h$ when deriving the $\frac{\Delta u}{\Delta u_o}$ vs t curve from the theoretical $\Delta u(t)/\Delta u_o$ vs T solution. Different $\chi$ values are selected for different $\Delta u/\Delta u_o$ levels in order to generate a complete dissipation curve that best fits the available measured data. For example, the solid curve shown on Figure E-14 was generated using the $\chi$ values shown on the inset of Figure E-14, where $\chi$ was taken as a linear function of $\Delta u/\Delta u_o$. Using this approach, any error in the estimated value of $G/s_u$ is also compensated for by this site-specific curve fitting procedure.
Figure E-1: CPT Tip Resistance Profiles

a) Measured Tip Resistance $q_c$ (bar)

b) Net Tip Resistance $q_T - \sigma_v$ (kPa)

*Note: $q_c$ corrected for temperature effects
Figure E-2: CPTU Penetration Pore Pressure Profiles
Figure E-3: Variation of $N_{KT}$ with Depth

$N_{KT} = (q_T - \sigma_{vo}) / (s_u)_{vane}$

Note: $N_{KT}$ based on average $q_T - \sigma_{vo}$ for middle of vane elevation (+/-10 cone diameters)
Figure E-4: Variation of $N_{\Delta u}$ with Depth

$N_{\Delta u} = \Delta u / (s_u)_{vane}$

Note: $N_{\Delta u}$ based on average $\Delta u$ for elevations across length of vane.
Figure E-5: Site-Specific Correlation between $N_{\Delta u}$ and $B_q$ Factors

CPT-7 (<16 m):
$N_{\Delta u} = 12.57 B_q$
$R^2 = 0.88$

CPT-1:
$N_{\Delta u} = 9.57 B_q$
$R^2 = 0.61$

CPT-2:
$N_{\Delta u} = 12.60 B_q$
$R^2 = 0.79$

CPT-7 (<10 m):
$N_{\Delta u} = 11.01 B_q$
$R^2 = 0.92$

Figure E-5: Site-Specific Correlation between $N_{\Delta u}$ and $B_q$ Factors
Figure E-6a: OCR Interpreted from CPT-1 Data

Method 1: $OCR = 0.5 + 1.5(\frac{u_1 - u_o}{u_o})$ (Sully et al., 1990)

Method 2: $OCR = \left[\frac{s_u}{\sigma'_{vo}}/\sigma'_{vo,NC}\right]^b$ (Schmertmann, 1978),
$b = 1.13 + 0.04(\frac{s_u}{\sigma'_{vo}})/\sigma'_{vo,NC}$ (Schmertmann, 1978)

Method 3: $OCR = \left[\frac{s_u}{\sigma'_{vo}}/\sigma'_{vo,NC}\right]^b$ (Schmertmann, 1978),

Method 4: $OCR = 2[\frac{(q_T - u_1)/\sigma'_{vo} - 1}{1.95M}]^{1/\Lambda}$ (Mayne, 1991)

Method 5: $OCR = 0.75(q_T - u_1)$ (Chen & Mayne, 1995)
Overconsolidation Ratio - OCR

Figure E-6b: OCR Interpretted from CPT-2 Data
Overconsolidation Ratio - OCR

Method 1 (CPT-1,7)  
Method 2  
Method 3  
Method 4  
Method 5  
from Odometer (Crawford & Campanella, 1991)

Method 1:  OCR = 0.5 + 1.5(u_1-u_2)/u_o (Sully et al., 1990)

Method 2:  OCR = \left(\frac{s_u}{\sigma'_w}/(s_u/\sigma'_w)_{NC}\right)^b (Schmertmann, 1978),  
b = 1.13 + 0.04\left(\frac{s_u}{\sigma'_w}/(s_u/\sigma'_w)_{NC}\right) (Schmertmann, 1978)

Method 3:  OCR = \left(\frac{s_u}{\sigma'_w}/(s_u/\sigma'_w)_{NC}\right)^b (Schmertmann, 1978),  
b = 1/\Lambda = \left[1-C_e/C_c\right]^{-1} (Wroth, 1984, Wood, 1990)

Method 4:  OCR = 2\left[q_T - u_2\right]/(\sigma'_w(1+1.95M))^{1/\Lambda} (Mayne, 1991)

Method 5:  OCR = 0.5(q_T - u_2) (Chen & Mayne, 1995)

Figure E-6c: OCR Interpretted from CPT-7 Data
Figure E-7: Variation of Estimated $K_o$ with Depth
Figure E-8: Interval Technique for Determining Shear Wave Velocity from Seismic CPT Tests (after Gillespie, 1990)
Figure E-9: Example of Shear Waves from Seismic CPT Tests

CPT-6
Geophone Elevation = -6.15 m,
Seismic test carried out 8.4 hrs after stopping penetration

Signal Measured by Geophone

Volts

0.75
0.5
0.25
0.0

50.0ms
60.0ms
70.0ms
80.0ms

TIME

First Arrival
First Peak
Second Peak
First Cross-Over
Shear Wave Velocity - $V_s$ (m/s)

Figure E-10: Shear Wave Velocity Profiles from Seismic CPT
\[ G_{\text{max}}/p_a = C (\sigma'/p_a)^m, \text{ therefore} \]
\[ \ln(G_{\text{max}}/p_a) = m \ln(\sigma'/p_a) + \ln(C) \]

\[ \sigma'_m (\text{CPT-5}): \]
\[ y = 0.77x + 7.38 \]
\[ R^2 = 0.98 \]

\[ \sigma'_m (\text{CPT-6}): \]
\[ y = 0.96x + 6.41 \]
\[ R^2 = 0.99 \]

\[ \sigma'_m (\text{CPT-7}): \]
\[ y = 0.89x + 6.62 \]
\[ R^2 = 0.80 \]

\[ \sigma'_{vo} (\text{CPT-5}): \]
\[ y = 0.85x + 6.89 \]
\[ R^2 = 0.97 \]

\[ \sigma'_{vo} (\text{CPT-6}): \]
\[ y = 1.18x + 5.06 \]
\[ R^2 = 0.88 \]

\[ \sigma'_{vo} (\text{CPT-7}): \]
\[ y = 1.03x + 5.90 \]
\[ R^2 = 0.99 \]

\[ \sigma'_m = (\sigma'_{vo} \times \sigma'_{ho})^{1/2} \]

Figure E-11: Site-Specific Correlations between Normalized \( G_{\text{max}} \) and Normalized In-Situ Effective Stress
Note:

\[ m = 0.8 \text{ for } \sigma'_v \]
\[ m = 1.0 \text{ for } \sigma'_m \]
\[ \sigma'_m = (\sigma'_v \times \sigma'_h)^{1/2} \]

Figure E-12: Variation with Depth of \( G_{\text{max}} \)-\( \sigma' \) Correlation Factors
Figure E-13: Site-Specific Bq - OCR Correlations (above -12 m elev.)
Figure E-14: Method of Extrapolating Measured CPTU Dissipation Data Using Theoretical Solutions
APPENDIX F

PORE PRESSURE GENERATION
DURING INSTALLATION OF TEST PILES
Figure F-1a: Excess Pore Pressures during Installation of Test Pile TP2
Figure F-1b: Excess Pore Pressures during Installation of Test Piles TP3 & TP4
Figure F-2: Estimating Installation Pore Pressures on Pile Shaft from Root Time Dissipation Plots
APPENDIX G

PILE LOAD TESTING

G.1 Correcting for Load Sharing Between Steel Shaft and Grout Column
G.2 Distribution of Loads within Piles Load Tested at 19 Hours and 7 Days
G.3 Load-Settlement and Settlement-Time Curves for Individual Piles
G.4 Estimating Load Distribution within Lead Section of Piles TP1 & TP2
G.5 Estimating Undrained Shear Strength Mobilized by Different Pile Sections
G.6 Estimating Maximum Probable Error and Propagation of Errors
APPENDIX G: PILE LOAD TESTING

G.1 CORRECTING FOR LOAD SHARING BETWEEN STEEL SHAFT AND GROUT COLUMN

After analyzing the results of the load tests carried out on Piles TP3 and TP4, where strain gauge data was available at locations SG1,2,3&4, it was discovered that the compressive load within the pile along the grout column was being shared between the pile shaft and the grout. Therefore, the load calibration factors determined in the laboratory by applying a known load to the pile shaft and measuring the voltage output of the strain gauges mounted on the shaft, were not applicable to the field situation where the shaft was encased in grout. As a result, when the laboratory calibration factors were applied to the strain gauge voltages measured during the load tests on piles TP3 and TP4, the calculated loads at SG1,2&3 within the grout column of both piles were below the loads measured at the SG4 location. Such a situation is physically impossible, unless the soil was transferring load to the pile, which is not believed to be the case during the load tests where the pile is displacing downward relative to the soil.

This problem had not been anticipated because of the much greater Young’s modulus of the steel (200 GPa) as compared to the modulus of the grout (15 to 20 GPa). However, since the cross-sectional area of the grout is much larger than that of the steel shaft, the axial stiffness of the grout can end up being close to that of the steel shaft. Consequently, load sharing between the grout column and the steel shaft can be significant, depending on the cross-sectional area of the grout column in the ground.

To correct this problem, correction factors were applied to the load calibration factors measured in the laboratory in an attempt to account for the increased axial stiffness of the composite column. Since the actual diameter of the grout column in the ground is not known, except for at SG1 within the PVC casing, these correction factors could not be calculated analytically. Instead, correction factors for the SG3 strain gauges (located near the bottom of the grout column) at TP3 and TP4 were established such that the corrected loads at SG3 were greater than those measured at SG4 (located just above the top helix) throughout the test. For pile TP2, a correction factor for the SG3 strain gauges was selected which was similar to those used for piles TP3&4, since similar problems were known to have occurred.
At SG1, located within the casing, the uncorrected loads measured by the strain gauges were correlated to the loads measured by the load cell at the top of the pile, as shown on Figures G-1a to G-1c for piles TP2,3,4. The correlations were found to be linear throughout the range of applied loads during the load tests, and so the use of a linear correction factor to determine the total load in the composite column from the load measured in the pile shaft seemed justified. The linearity was also considered to be a strong indicator that the load on the pile shaft within the casing was being transferred to the grout but not to the soil.

Once the loads at SG1 and SG3 had been corrected, correction factors were selected for SG2 (located between SG1 and SG3) such that the corrected loads were greater than those at SG3 and less than those at SG1 for all load increments. The correction factors applied to the data from strain gauges at the SG1,2&3 locations on piles TP1 through TP6 are listed in Table G-1. For the SG3 data, the credible range of correction factors which could have been used without the results becoming unreasonable have also been indicated, since this data was used to assess the pile behaviour that was discussed in Chapter 7.

In an attempt to eliminate the problem of the grout column bridging load across the strain gauge installations, construction joints were added at the top of each strain gauge cover within the grout columns of piles TP5 and TP6. These construction joints were designed to provide a compressible break in the grout column so that all of the load carried by the composite column would have to be transferred through the much stiffer steel shaft at the strain gauge locations. The construction joints consisted of layers of both open-cell foam and rubber which were fixed to the top of the strain gauge cover and a steel disc (same diameter as other grout discs) located near the top of the strain gauge cover, as shown on Figure G-2. At four out of the five installations at which these joints were used, they were found to be very successful at minimizing the proportion of load carried by the grout column across the strain gauge installation, as shown by the strain gauge – load cell correlation for TP6-SG1 on Figure G-1d. At TP5-SG3, a correction factor similar to those used at TP3&4-SG3 had to be used. The reason that the construction joint failed at this location is not clear.
<table>
<thead>
<tr>
<th>Pile No.</th>
<th>Strain Gauge Location</th>
<th>Calibration Correction Factors</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Best Estimate</td>
</tr>
<tr>
<td>TP3</td>
<td>SG1</td>
<td>1.73</td>
</tr>
<tr>
<td></td>
<td>SG2</td>
<td>~1.3 (avg.)</td>
</tr>
<tr>
<td></td>
<td>SG3</td>
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</tr>
<tr>
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<td>SG1</td>
<td>1.80</td>
</tr>
<tr>
<td></td>
<td>SG2</td>
<td>~1.4 (avg.)</td>
</tr>
<tr>
<td></td>
<td>SG3</td>
<td>1.15</td>
</tr>
<tr>
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<td>SG1</td>
<td>1.31</td>
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<td>SG2</td>
<td>1.00</td>
</tr>
<tr>
<td></td>
<td>SG3</td>
<td>1.20</td>
</tr>
</tbody>
</table>

**Table G-1**

*Strain Gauge Calibration Correction Factors*

**Note:** Correction factors in italics determined from linear correlation between uncorrected strain gauge loads and load cell measurements.

**G.2 DISTRIBUTION OF LOADS WITHIN PILES LOAD TESTED AT 19 HOURS AND 7 DAYS**

Plots of the load distributions within the piles obtained via the strain gauge measurements on the pile shaft at various load levels during the load tests on piles TP3 to TP6 are included in this appendix as Figures G-3a to G-3d. Curves of the load transferred to the soil by the different sections of the piles vs. the applied load at the pile top are shown for piles TP3 to TP6 on Figures G-4a to G-4d.
G.3 Load-Settlement and Settlement-Time Curves for Individual Piles

Load-settlement curves for piles TP1 to TP6 and PP1 are included in this appendix as Figures G-5a to G-5g. Plots of settlement vs. time measured during individual load intervals for each pile are included as Figures G-6a to G-6g.

G.4 Estimating Load Distribution within Lead Section of Piles TP1 & TP2

No strain gauges were located on the lead sections of piles TP1 and TP2 which were load tested at 6 weeks. Therefore, the loads transferred to the soil by the grout column (including the bottom grout disc) and by the lead section (all helices) of TP1 and TP2 had to be estimated from the loads measured at SG3 and from estimates of the loads transferred between the SG3 and SG4 locations. The load settlement curves for the section of pile between SG3 and SG4 from piles TP3 and TP4 are shown on Figure G-7, along with the error bars representing the maximum credible error (MCE) associated with the correction of the SG3 strain gauge loads described above. The load-settlement curve for the piles at 6 weeks was assumed to have the same shape as those at 7 days, which was approximated using a logarithmic function. Parameters were selected for the logarithmic function to obtain the curves for 6 weeks shown on Figure G-7, which represent the maximum and minimum credible limits on the estimated load-settlement relationship. The lower limit was selected to be about the same as the estimated loads measured at 7 days. The upper limit was selected such that the average of the two limits was roughly 30% higher than the estimated loads measured at 7 days, which is similar to the relative increase in capacity observed along the grout column above SG3 between 7 days and 6 weeks.

In order to estimate the capacity of the upper helices at 6 weeks, an estimate of the load carried by the bottom helix of piles TP1 and TP2 had to be made. Thus, the pore pressures measured at the PP3 location on these piles was combined with the $\Delta u / \Delta \sigma_v$ response observed at the same location on piles TP3 and TP4 to obtain an estimate of the bearing pressure at failure. Therefore, the ultimate capacity of the bottom helix can be calculated according to:

$$Q_{hx-b} = \Delta u_{PP3} / (\Delta u / \Delta \sigma_v) \pi R_{hx}^2$$
The $\Delta u/\sigma'_v$ – settlement curves for TP1&2-PP3 are provided on Figure G-8, along with the curves for TP3&4-PP3. Piezo-port TP3-PP3 is known to have experienced some response lag, and hence the $\Delta u/\sigma'_v$–settlement curve falls below that for TP4-PP3, even though the load carried by the bottom helix of TP3 is substantially higher than that of TP4. The best indicator of the lag in the pore pressure response of TP3-PP3 is the reduced slope of the $\Delta u/\sigma'_v$–settlement curve at small settlements when the load-settlement response of the different piles is known to be very similar. From the initial slopes of the curves on Figure G-8, it appears that TP1&2-PP3 also experienced lag effects. Therefore, the $\Delta u/\Delta \sigma_v$–settlement relation observed at TP3-PP3, which included lag effects, was used to estimate the load-settlement response of the bottom helix of TP1 and TP2, which is compared to that of piles TP3 and TP4 on Figure G-9. The load-settlement response of the bottom helix of TP1 and TP2 was also calculated assuming a constant $\Delta u/\Delta \sigma_v$ ratio of 0.65 (similar to that observed below the bottom helix of pile TP4) and is shown on Figure G-9. It can be seen that the two different $\Delta u/\Delta \sigma_v$ assumptions generate very different load-settlement curves:

- Assuming a $\Delta u/\Delta \sigma_v$–settlement relation similar to that at TP3-PP3, the load-settlement response is predicted to be stiff and much more brittle than that measured at 7 days, with peak capacity occurring at relatively low settlements and with significant post-peak softening. This may not be entirely realistic based on the inferred bearing-type load transfer mechanism, and suggests that the response lag of the TP1&2 tip piezo-ports may not have been as great as that of the TP3 piezo-port. Therefore, the assumed $\Delta u/\Delta \sigma_v$ ratios may have over-compensated for the true lag effects.

- Assuming a constant $\Delta u/\Delta \sigma_v$ ratio = 0.65, the load-settlement response is predicted to be softer and more ductile than that measured at 7 days. This is not realistic and suggests that the $\Delta u/\Delta \sigma_v$ ratio from TP4 under-predicts the true loads transferred by the bottom helix of piles TP1 & TP2, as would be expected due to the apparent lag in the TP1 & TP2 piezo-port response.

Thus, the true load-settlement curves for the bottom helix of piles TP1 & TP2 probably plot somewhere between the two extremes shown on Figure G-9 (probably closer to the TP3-PP3-based estimates). Thus, a best estimate for the capacity of the bottom helix of 27 kN, with a credible range of error of ±4.5 kN, was assumed for both piles TP1 and TP2. This best
estimate is consistent with the mean bottom helix capacities measured at all 5 of the other test piles, and the credible range includes the unusually low capacity measured at TP4.

**G.5 Estimating Undrained Shear Strength Mobilized by Different Pile Sections**

The average undrained shear strength mobilized along the *surface of the grout column*, \((s_u)_{gr}\), was calculated from Equation 2.3 using the diameter of the grout discs, \(D_{gr}\), and an \(\alpha\) factor of 1 (which assumes that the adhesion between the grout surface and the soil is perfect), such that:

\[
(s_u)_{gr} = \frac{Q_{gr}}{\pi D_{gr} L_{gr}}
\]

This assumes that the average diameter of the grout column is equivalent to the disc diameter and that there is no additional contribution due to bearing below any of the discs. The diameter of the grout may be less than that of the discs along the portions of the column midway between the grout discs, due to inward soil squeeze before the grout sets. If this is the case, the reduction in frictional resistance will be compensated to some extent by an increase in bearing resistance near the grout discs.

The average undrained shear strength mobilized by the *bottom helix plate* (including tip of pile shaft), \((s_u)_{hx-b}\), was estimated from Equation 2.2a using \(N_c = 9\) (maximum probable error on \(N_c\) of ±1), such that:

\[
(s_u)_{hx-b} = \frac{Q_{hx-b}}{9\pi R_{hx}^2}
\]

The average undrained shear strength mobilized by the *upper helices spaced at S/D = 1.5*, \((s_u)_{hx-cyl}\), was estimated from Equation 2.1 for cylindrical failure, and using the appropriate length between the applicable helices involved in the measured loads, such that:

\[
(s_u)_{hx-cyl} = \frac{Q_{hx-i,j}}{\pi D_{hx} L_{hx-i,j}}
\]

where \(L_{hx-i,j}\) is the length between the \(i^{th}\) and \(j^{th}\) helices and, similarly, \(Q_{hx-i,j}\) is the load transferred between the \(i^{th}\) and \(j^{th}\) helices.
The average undrained shear strength mobilized by the upper helix plates spaced at $S/D = 3$, $(s_u)_{hx-ind}$, was estimated from Equation 2.4 for individual plate bearing using $N_c = 9$ (maximum probable error on $N_c$ of $\pm 1$), such that:

$$(s_u)_{hx-ind} = \frac{(Q_{hx-i,j} - Q_{shaft})}{n \cdot 9\pi (R_{hx}^2 - R_{shaft}^2)}$$

where $Q_{hx-i,j}$ is the total load transferred between the $i^{th}$ and $j^{th}$ helices, $n$ is the number of helices carrying $Q_{hx-i,j}$, and $Q_{shaft}$ is the resistance mobilized along the shaft between the plates. $Q_{shaft}$ was estimated according to:

$$Q_{shaft} = \pi D_{shaft} n L_{eff} \alpha (s_u)_{shaft}$$

where:

- $L_{eff}$ is the effective adhesion length along the pile shaft beyond the bearing zone of each plate ($L_{eff} = \text{helix spacing} - L_{ineff}$). According to the data from Narasimha Rao et al. (1993), the ratio of $L_{ineff}/D_{hx}$ typically falls between 1.5 and 2.0, although a wider range of 1.1 to 2.3 was observed.

- A possible range of adhesion factors, $\alpha = 0.5$ to 1.0, was considered.

- The undrained shear strength along the pile shaft, $(s_u)_{shaft}$, was assumed to be equal to the average $(s_u)_{gr}$ from the piles tested at the same time after installation.

The maximum probable error associated with the above methods of estimating $s_u$ for the different sections of each pile are given in Table G-2 along with the best estimates of $s_u$.

**G.6 Estimating Maximum Probable Error and Propagation of Errors**

In this appendix, methods of estimating the load carried by the piles at different locations, and for estimating the undrained shear strength mobilized by the different pile sections, have been described, and each estimate has a certain degree of uncertainty and potential error associated with it. A maximum range of credible values (or MCR), which is typically based on available information and engineering judgement, is also associated with each estimate. The difference between the upper or lower limits of this range and the best estimate will be called the “maximum credible error” or MCE.
### Table G-2
Back-Calculated Shear Strengths & Maximum Probable Error

<table>
<thead>
<tr>
<th>Pile No. + (S/D)</th>
<th>Mobilized Undrained Shear Strength(^{(1)}) - ((s_u))(_{\text{pile}}) (kPa)</th>
<th>Grout Column(^{(2)})</th>
<th>Upper Helices</th>
<th>Bottom Helix</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>At Peak Strength</td>
<td>At Pile Failure</td>
<td>Top(^{(3)})</td>
<td>Middle(^{(4)})</td>
</tr>
<tr>
<td>TP5 (3)</td>
<td>3.2±0.2 2.7±0.4</td>
<td>19.0±2.1 22.2±2.5</td>
<td>20.6±2.3</td>
<td>31.5±3.5</td>
</tr>
<tr>
<td>TP3 (3)</td>
<td>10.1±0.3 6.4±1.2</td>
<td>16.3±1.9 21.7±2.4</td>
<td>19.0±2.2</td>
<td>31.8±3.5</td>
</tr>
<tr>
<td>PP1 (3)</td>
<td>10.9±1.3 8.3±1.6</td>
<td>21.3±2.4 19.7±3.1</td>
<td>28.7±3.2</td>
<td>30.1±4.6</td>
</tr>
<tr>
<td>TP1 (3)</td>
<td>10.9±1.3 8.3±1.6</td>
<td>13.8 10.5 12.2</td>
<td>30.6±3.4</td>
<td></td>
</tr>
<tr>
<td>TP6 (1.5)</td>
<td>4.5±0.2 4.0±0.3</td>
<td>15.0 16.9 16.0</td>
<td>25.2±2.8</td>
<td></td>
</tr>
<tr>
<td>TP4 (1.5)</td>
<td>8.1±0.6 8.1±1.1</td>
<td>18.5±1.7</td>
<td>30.1±4.8</td>
<td></td>
</tr>
<tr>
<td>TP2 (1.5)</td>
<td>12.1±1.7 8.5±1.8</td>
<td>18.5±1.7</td>
<td>30.1±4.8</td>
<td></td>
</tr>
</tbody>
</table>

**Notes:**

1. \(s_u\) given is best estimate, ± maximum probable error
2. 2.5 m length along surface of column, from bottom of casing to SG3
3. top plate (S/D=3) or top 2 plates (S/D=1.5)
4. middle plate (S/D=3) or middle 2 plates (S/D=1.5)
5. all helix plates excluding bottom plate

If the estimates were characterized as probability density functions, then the best estimate would have the highest probability of occurrence, while values at or near the credible limits would have the lowest probability of occurrence. If such probability density functions are assumed to be normally distributed, such that the best estimate is equivalent to the mean (\(\mu\)) of the distribution, then the probability that the true value is within 1, 2 or 3 standard deviations (\(\sigma\)) of the best estimate is 66.3%, 95.4% or 99.7%, respectively. Thus, the MCE would be essentially the same as 3\(\sigma\), and can therefore be used to estimate \(\sigma\):

\[
\sigma = \frac{\text{MCE}}{3} \quad \text{or} \quad \sigma = \frac{\text{MCR}}{6}
\]

Using a normal distribution, it can be seen that the difference in probability between a value being within 2\(\sigma\) or 3\(\sigma\) of the best estimate is small (about 4%) while the difference between the potential errors is large (50%). Thus, it is more reasonable to present estimates with an associated “maximum probable error” (MPE) that corresponds to 2\(\sigma\) (i.e. 95% probability that the actual value is within 2\(\sigma\) of the estimate).
When estimating a quantity, such as a load or shear strength, which is a function of a number of input parameters, some of which have uncertainty associated with them, the total potential error associated with the calculated quantity must also be calculated. For a general function, \( Y = Y(x_1, x_2, \ldots, x_n) \), the mean and variance \( (\sigma^2) \) for \( Y \) are calculated as follows:

mean \( Y \): \( \mu_Y = Y(\mu_{x1}, \mu_{x2}, \ldots, \mu_{xn}) \),

variance for \( Y \): \( (\sigma_Y)^2 = \sum_{i=1}^{n} \left( \left( \frac{\partial Y}{\partial x_i} \right)^2 \sigma_{x_i}^2 \right) \)
Figure G-1a: Field Correction of Calibration Factor for Strain Gauge TP2-SG1

Figure G-1b: Field Correction of Calibration Factor for Strain Gauge TP3-SG1
Figure G-1c: Field Correction of Calibration Factor for Strain Gauge TP4-SG1

Figure G-1d: Field Correction of Calibration Factor for Strain Gauge TP6-SG1
Figure G-2: Assembly of Grout Column Construction Joints

- Silicone Sealant between top of cover & pile shaft
- Steel Disc
- Strain Gauge Cover
- Open-Cell Foam on top of cover & disc
- Rubber over open-cell foam
Figure G-3a: Load Distributions During Load Test on Pile TP3
Figure G-3b: Load Distributions During Load Test on Pile TP4
Figure G-3c: Load Distributions During Load Test on Pile TP5
Figure G-3d: Load Distributions During Load Test on Pile TP6
Figure G-4a: Loads Transferred to Soil during Load Test on Pile TP3

(Pile TP3: S/D = 3, Loaded 7 days after installation)
Figure G-4b: Loads Transferred to Soil during Load Test on Pile TP4

(Pile TP4: S/D = 1.5, Loaded 7 days after installation)
Figure G-4c: Loads Transferred to Soil during Load Test on Pile TP5

(Pile TP5: S/D = 3, Loaded 1 day after installation)
Figure G-4d: Loads Transferred to Soil during Load Test on Pile TP6

(Pile TP6: S/D = 1.5, Loaded 1 day after installation)
Figure G-5a: Load-Settlement Curves from Load Test on Pile TP1

(Pile TP1: S/D = 3, Loaded 42 days after installation)
Figure G-5b: Load-Settlement Curves from Load Test on Pile TP2

(Pile TP2: S/D = 1.5, Loaded 43 days after installation)
Figure G-5c: Load-Settlement Curves from Load Test on Pile TP3

(Pile TP3: S/D = 3, Loaded 7 days after installation)
Figure G-5d: Load-Settlement Curves from Load Test on Pile TP4
Figure G-5e: Load-Settlement Curves from Load Test on Pile TP5 (Pile TP5: S/D = 3, Loaded 1 day after installation)
Figure G-5f: Load-Settlement Curves from Load Test on Pile TP6
Figure G-6a: Settlement-Time Curves from Load Test on Pile TP1
Figure G-6b: Settlement-Time Curves from Load Test on Pile TP2
Figure G-6c: Settlement-Time Curves from Load Test on Pile TP3
Figure G-6d: Settlement-Time Curves from Load Test on Pile TP4
Figure G-6e: Settlement-Time Curves from Load Test on Pile TP5
Figure G-6f: Settlement-Time Curves from Load Test on Pile TP6
Figure G-7: Load-Settlement Curves for Bottom of Grout Column (SG3 to SG4)
Figure G-8: Excess Pore Pressure Generation below Bottom Helix during Load Test

Figure G-9: Load-Settlement Response of Bottom Helix Inferred from Pore Pressures