Geotechnical test embankment on soft marine clay in Newington-Dover, New Hampshire

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Geotechnical test embankment on soft marine clay in Newington-Dover, New Hampshire

Abstract
During the fall of 2012, the New Hampshire Department of Transportation built a test embankment for the new alignment segment of the proposed Exit 6 Southbound On-Ramp site off Route 4 in Newington-Dover, New Hampshire. The purpose of the test embankment is to determine effective and efficient treatment for the long-term ground settlement of the local soft marine clay native to the site. In addition to the test embankment, prefabricated vertical drains were installed to allow pore water from the soil to seep out and accelerate time-rate consolidation. Various in situ field testing methods were performed including: piezocone penetration, flat plate dilatometer and field shear vane. Additionally, consolidation testing was performed in the laboratory with undisturbed samples of the clay. Settlement calculations were performed to compare with field values. This research will benefit construction of the site, in addition to many other upcoming sites of construction in the local area.

Keywords
Engineering, Civil, Geotechnology

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GEOTECHNICAL TEST EMBANKMENT ON SOFT MARINE CLAY IN NEWINGTON-DOVER, NEW HAMPSHIRE

BY

AMY GETCHELL

B.S. Civil Engineering, University of Maine, 2011

THESIS

Submitted to the University of New Hampshire

in Partial Fulfillment of

the Requirements for the Degree of

Master of Science

in

Civil Engineering

September, 2013
This thesis has been examined and approved.

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Date
ACKNOWLEDGEMENTS

First, I would like to extend my sincerest thankfulness to my advisor, Professor Benoît, for giving me all of the opportunities and experiences graduate school has offered to me. I would also like to thank him for his unending support and encouragement.

I would like to thank the NHDOT, specifically Joe Blair and Chuck Dusseault, for giving me the opportunity to be a part of this project, it has been a very exciting experience. I would also like to thank the field personnel for their assistance in the field: Doug, Scott, John "Woody", PJ, Jimmy and Bob.

I would like to thanks Professors Majid Ghayoomi and Igor Tsukrov for being a part of my thesis committee and taking time out of their busy schedules.

I would like to thank the undergraduate students who assisted me in the field and lab: Nate Bradley and Sonja Pape.

Lastly, I would like to thank my family for their never-ending support throughout my academic career.
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ABSTRACT

NHDOT GEOTECHNICAL TEST EMBANKMENT ON SOFT MARINE CLAY

By

Amy Getchell

University of New Hampshire, September, 2013

During the fall of 2012, the New Hampshire Department of Transportation built a test embankment for the new alignment segment of the proposed Exit 6 Southbound On-Ramp site off Route 4 in Newington-Dover, New Hampshire. The purpose of the test embankment is to determine effective and efficient treatment for the long-term ground settlement of the local soft marine clay native to the site. In addition to the test embankment, prefabricated vertical drains were installed to allow pore water from the soil to seep out and accelerate time-rate consolidation. Various in situ field testing methods were performed including: piezocone penetration, flat plate dilatometer and field shear vane. Additionally, consolidation testing was performed in the laboratory with undisturbed samples of the clay. Settlement calculations were performed to compare with field values. This research will benefit construction of the site, in addition to many other upcoming sites of construction in the local area.
Chapter 1 - INTRODUCTION

In the spring of 2012, the New Hampshire Department of Transportation (NHDOT) approached the University of New Hampshire (UNH) about a proposed test embankment that would be constructed between Station 600+60 and Station 613+50 of the future Exit 6 SB On-Ramp in Newington-Dover, NH on top of an extensive compressible marine deposit. The embankment would be divided into 7 segments, 5 of which would be instrumented with stable benchmarks, settlement monitoring points, piezometers and inclinometers to monitor the settlement and generated pore pressures. The embankment would be incorporated into the final design of the on-ramp and therefore, prefabricated vertical (PV) drains would be installed throughout the 7 sections, using various spacings in a triangular pattern, to accelerate the time of consolidation. Additionally, a sand drainage blanket would be installed below the embankment. Table 1-1 summarizes the various spacings and height of fill used in each segment of the embankment.

Table 1-1: Summary of PV spacing and Fill Height for Embankment

<table>
<thead>
<tr>
<th>Segment</th>
<th>Location</th>
<th>Instrumentation</th>
<th>PV Spacing (ft)</th>
<th>Fill Height (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial</td>
<td>Station 600+60 to 602</td>
<td>No</td>
<td>8</td>
<td>Transitional</td>
</tr>
<tr>
<td>1</td>
<td>Station 602 to 604</td>
<td>Yes</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>2</td>
<td>Station 604 to 606</td>
<td>Yes</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>3</td>
<td>Station 606 to 608</td>
<td>Yes</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>4</td>
<td>Station 608 to 610</td>
<td>Yes</td>
<td>10</td>
<td>18</td>
</tr>
<tr>
<td>5</td>
<td>Station 610 to 612</td>
<td>Yes</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td>Final</td>
<td>Station 612 to 613+50</td>
<td>No</td>
<td>8</td>
<td>Transitional</td>
</tr>
</tbody>
</table>
Due to the significant compressibility and the areal extent of the underlying marine deposit, the NHDOT requested that UNH perform various in situ tests to assist in the determination of effective and efficient treatment for the long-term settlement of the marine clay native to the site and for other deposits in the general Seacoast area. The following phases of testing were planned: prior to construction of the embankment (Phase 1), after construction of the embankment (Phase 2) and long-term (Phase 3).

Prior to construction and removal of trees (Phase 1), piston sampling was used to obtain samples of the marine deposit for laboratory consolidation testing. Three separate flat plate dilatometer profiles were performed, including two profiles with dissipation tests. One field vane profile was conducted with testing every 3.28 ft (1 m) throughout the marine deposit. Additionally, nine piezocone profiles were advanced throughout the site to assess variability of subsurface conditions.

Using the data collected during Phase 1, settlement predictions were made using a geotechnical finite element software package titled “PLAXIS 2D 2011,” a 3-dimensional vertical consolidation and settlement software package by Rocscience, and hand calculations using the dilatometer constrained modulus and Boussinesq method.

Approximately four months after construction of the embankment was completed, additional dilatometer and piezocone testing was performed to determine the
change in strength and consolidation properties of the marine deposit (Phase 2). Phase 3 is not within the scope of this thesis.

The objectives of this research are as follows:

- Collect samples of the marine deposit via piston sampling to use in laboratory consolidation testing for the evaluation of soil parameters required in settlement analysis.

- Perform dilatometer, field vane and piezocone testing at various segments of the embankment both before and after construction. This will provide stratigraphic data of the area and allow comparison between testing methods. This data will also be compared to data collected previously in Portsmouth and Dover, NH on the same clay deposit.

- Predict the total and rate of settlement of the marine deposit after the embankment loading using the data collected in the field and laboratory. Various analytical methods will be used including finite element analyses, settlement software packages and hand calculations.

- Provide the NHDOT with guidelines on the prediction of settlement and rate of settlement of embankment loads on soft compressible marine clays in the Seacoast area.
Chapter 2 of this thesis reviews several case histories in the use of preloads and PV drains to accelerate the rate of consolidation of soft clays. Additionally, this chapter defines the different soil models considered in modeling the marine deposit for finite element analyses used to predict the settlement and rate of settlement. Chapter 3 describes the testing methods used to evaluate the geotechnical soil parameters of the marine deposit. Chapter 4 describes the embankment site and subsurface conditions. Chapter 5 discusses the three phases of testing performed to evaluate the behavior of the marine deposit. Chapter 6 compares Phase 1 data to past research and amongst the other testing methods performed at the Newington-Dover site. Chapter 7 uses the data collected during Phase 1 to predict the settlement of the marine deposit due to the embankment loading and the effect of the PV drains spacings. The settlement rate, especially the time to reach 90% consolidation, is evaluated for each PV spacing. Chapter 8 compares the data collected during Phase 1 and Phase 2. Chapter 9 summarizes the results of the testing program and provides conclusions and recommendations. Lastly, potential future research is addressed.
Chapter 2 - BACKGROUND

2.1 - Introduction

Constructing embankments on soft compressible soils often requires preloading the soils to minimize post construction settlements and to increase stability. Preloading or precompression is the process of consolidating foundation soils under an applied vertical stress prior to placement or completion of the final permanent construction load. Typically a temporary embankment is used to provide an initial load to the site. The settlement from the precompression load is a result of the processes of immediate deformation, primary consolidation and secondary compression. If the foundation soils are weak, overall stability of the embankment and foundation soils must be considered in design (FHWA, 1986).

In addition to precompression, prefabricated vertical (PV) drains can be used to assist in accelerating the rate of primary consolidation settlement. PV or “wick” drains are artificially created drainage paths that shorten the natural drainage path of the pore water and accelerate the rate of primary consolidation when combined with precompression. PV drains most commonly have a rectangular cross section consisting of a synthetic geotextile “jacket” surrounding a plastic core as seen in Figure 2-1.
PV drains are installed vertically into compressible subsurface soils to transmit the porewater up and down the length of the drains accelerating the dissipation of porewater pressures produced by the embankment load. According to FHWA, the benefits of using PV drains combined with precompression include: a decrease in the overall time required for completion of primary consolidation due to preloading, a decrease in the amount of surcharge required to achieve the desired amount of precompression and an increase in the rate of strength gain due to consolidation of soft soils (FHWA, 1986).

Figure 2-2 is a general schematic of a temporary or permanent embankment with PV drains. The drains are typically installed through low permeability soils. Various patterns can be used, although, a triangular spacing pattern is preferred over square spacing because it tends to generate a more uniform consolidation between drains. The water in the drains moves upward into the drainage blanket as well as down to underlying permeable layers if present (FHWA, 1986).
Figure 2-2: Typical Vertical Drain Installation for a Highway Embankment (FHWA, 1986)

The following sections will discuss the modeling of embankments on soft clays using finite element analysis and case histories where PV drains were used to expedite the settlement of soft compressible soils under embankments.

2.2 - Finite Element Analysis Soil Models

Finite element analysis (FEA) is a numerical technique for approximating solutions to boundary condition problems by dividing the model into small elements minimizing the error function to produce an accurate solution. Using FEA, geotechnical engineers can better analyze the deformation, stability and flow of water in complex geotechnical engineering problems where hand calculations cannot.
Geotechnical applications of FEA require advanced models for the simulation of the behavior of soils and rock. PLAXIS 2D, a finite element software for two-dimensional analysis in geotechnical engineering problems, is capable of modeling soil as a multi-phase material (air, water and solids), taking into account the change in pore water pressures throughout the study. Due to the complexity of soft clays, being able to closely approximate the change in pore water pressures and deformation due to embankment loading is valuable knowledge in design.

Various models can be used when evaluating the behavior of soft clays under embankment loading. Some of the models available within PLAXIS include: Mohr-Coulomb, Soft Soil, Soft Soil Creep and Modified Cam-Clay. Those models have been used extensively in numerous embankment loading studies.

The Mohr-Coulomb (MC) model estimates the soil as a linear elastic perfectly-plastic material where the user provides: Young’s modulus (E), Poisson’s ratio (ν), friction angle (φ), cohesion (c) and angle of dilatancy (Ψ). The MC model does not account for the increase in shear strength with consolidation. The Soft Soil Creep (SCC) model is used in the application of settlement and embankment problems for soft soils based on cohesion (c), friction angle (φ), angle of dilatancy (Ψ), modified swelling index (k*), modified compression index (λ*) and modified creep index (μ*). The SCC model was not considered due to a lack in information with respect to the creep characteristics of the deposit. The Modified Cam-Clay (MCC) model is meant for modeling of near normally-consolidated
clays and based on Poisson's ratio ($v$), Cam-Clay swelling index ($\kappa$), Cam-Clay compression index ($\lambda$), tangent of the critical state line ($M$) and initial void ratio ($e_{init}$).

The Soft Soil (SS) model is a combination of the Cam-Clay and Mohr-Coulomb models and is ideal in modeling primary compression of near normally-consolidated clays, clayey silts and peat (PLAXIS Manual, 2011). Compared to other models, the SS model uses material parameters commonly found in conventional lab or field testing: Modified compression index ($\lambda^*$), Modified swelling index ($\kappa^*$), cohesion ($c$), friction angle ($\varphi$) and dilatancy angle ($\psi$). As a result, the SS model was selected to represent the marine deposit in analyses for this research.

2.2.1 - Soft Soil (SS) Model

In the SS model, the soil response due to a change in stress is modeled by a semi-logarithmic relationship between the volumetric strain ($\varepsilon_v$) and mean effective stress ($p'$) as seen in Figure 2-3. This relationship can be evaluated using equations [2-1] for virgin compression and [2-2] for unloading and reloading as defined by PLAXIS (2011).

\[ \varepsilon_v - \varepsilon_v^0 = -\lambda^* \ln \left( \frac{p'}{p_0^*} \right) \]  

[2-1]

Where $p'$ is the current mean effective stress and $p_0^*$ is the initial mean effective stress.
\[ \varepsilon_{v}^{e} - \varepsilon_{v}^{e0} = -\kappa^{*} \ln\left(\frac{p'}{p_{0}}\right) \] \[ [2-2] \]

Where \( \varepsilon_{v}^{e} \) is the elastic volumetric strain.

\[ p' = \frac{1}{3} (\sigma_{1}' + \sigma_{2}' + \sigma_{3}') \] \[ [2-3] \]

Where \( \sigma_{1}', \sigma_{2}' \) and \( \sigma_{3}' \) are the principal effective stresses.

\[ q = |\sigma_{1}' - \sigma_{2}'| \] \[ [2-4] \]

Figure 2-3: Logarithmic Relation Between Volumetric Strain and Mean Stress for PLAXIS SS Model (PLAXIS Material Models Manual, 2011)
The mean effective stress and equivalent shear stress (q), which can be evaluated based on equations [2-3] and [2-4], define the stress state of the soil. The dilatancy angle controls the amount of plastic volumetric strain developed during shearing and generally can be neglected and left as 0° for clays.

The modified compression index determines the compressibility of the material during primary loading, whereas, the modified swelling index determines the compressibility of the material during unloading and reloading. Both parameters can be obtained by performing one-dimensional compression tests in the laboratory. PLAXIS (2011) allows the user to enter the compression index (Cc) and swelling index (Cs) as an alternate to λ* and κ* based on equations [2-5] and [2-6].

\[ \lambda^* = \frac{C_c}{2.3(1 + e)} \]  

[2-5]

Where e is the initial void ratio.

\[ \kappa^* = \frac{2C_s}{2.3(1 + e)} \]  

[2-6]

Additionally, a yield function (f) is used to describe the boundary of the region between elastic (reversible) and inelastic (irreversible) volumetric strains. The yield function for the triaxial stress state (σ'2 = σ'3) is defined by equation [2-7]. Initially, the soil is assumed to be elastic during the unload and reload cycles,
however, once the stresses during primary compression reach the limit of the yield function, the strains are plastic.

\[ f = \bar{f} - p_p \]  

[2-7]

Where \( \bar{f} \) is a function of the stress state \((p',q)\) and \( p_p \) is the preconsolidation stress (PLAXIS, 2011).

As seen in Figure 2-4, the yield function can be represented as an ellipse where the parameter \( M \) determines the height of the ellipse. The height of the ellipse is responsible for the ratio of the horizontal and vertical stresses in primary one-dimensional compression, and therefore, parameter \( M \) determines the coefficient of lateral earth pressure. The parameter \( M \) can be found using equation [2-8].
The M-line is referred to as the critical state line (CSL) and represents the stress states at post peak failure. The preconsolidation stress represents the extent of the ellipse along the \( p' \)-axis and can be calculated using the hardening relationship represented by equation [2-9]. During primary compression, the ellipse grows, increasing the boundary between elastic and inelastic strains.

\[
\bar{f} = \frac{q^2}{M^2(p' + c \cot \phi)} + p'
\]  

[2-8]

\[
p_p = p_p^0 \exp\left(\frac{-\varepsilon_p^p}{\lambda - \kappa} \right)
\]  

[2-9]

Where \( p_p^0 \) is the initial value of the preconsolidation stress (PLAXIS, 2011).

The combination of the compression yield functions, represented by the ellipses in Figure 2-4, and the perfectly-plastic Mohr-Coulomb yield functions define the SS model in the three-dimensional effective principal stress space. The Mohr-Coulomb yield function is represented by the failure line in Figure 2-4. Combinations of the effective principal stresses that fall along the cap found in Figure 2-5 represent states of failure (PLAXIS, 2011).

The SS model will be used in Chapter 7 to model the soft marine deposit under the embankment loading and provide estimates of settlement.
2.3 - Case Histories

Three case histories were reviewed to better understand the behavior of a soft soil experiencing embankment loading with or without PV or sand drains. The first case history summarizes the field findings of a test embankment with sand drains at the Pease Air Force Base in Portsmouth, NH, less than 5 miles from the Newington-Dover site. The results at this location will be referenced throughout the thesis due to the proximity of the site. The second case history reviews two separate sites in Sweden with embankments on a soft soil both with and without PV drains. The author compares the field data with that found using PLAXIS to
model the embankments. The last case history looks at an embankment built on a soft clay deposit with PV drains in Malaysia. Using a FEA software titled CRISP, the field measurements were compared to the FEA results.

2.3.1 - Case History 1: Pease Air Force Base Portsmouth, NH

In 1967, the New Hampshire Department of Public Works and Highways (NHDPWH) began a program of improvements for Interstate 95 (I-95) in Portsmouth, NH; specifically the area bounded by I-95, US Route 4 and the Pease Air Force Base. According to Ladd (1972), the site contained significant deposits of soft very sensitive marine clay which presented concern in design. After ample field exploration and laboratory testing, there were still uncertainties in regards to the engineering properties of the clay. In order to better define the engineering properties of the clay, a test embankment was constructed to failure in 1968. It was found that large excess pore pressures developed under the marine layer causing large lateral deformations (Ladd, 1972).

Due to the poor conditions of the site, when it came time to build the embankments for roadways and construction, the design included a combination of surcharge fills, stabilizing berms, staged construction and the installation of vertical sand drains. Vertical sand drains work similarly to PV drains. The vertical sand drains worked very well in assisting the clay to reach primary consolidation within one year and provided additional stability. After a detailed analysis of the field data, it was determined that the field settlements exceeded the predicted settlements by an average of 27 ± 16%. The clay proved to be
more compressible and susceptible to undrained creep than was initially predicted (Ladd et al., 1972).

For the current project, the NHDOT followed Ladd’s work closely, due to the similarities of the project and marine deposit. The marine clay found in Newington-Dover was thought to have comparable geotechnical properties of those found in Portsmouth and initial calculations were based on Ladd’s findings.

2.3.2 - Case History 2: Lilla Mellösa and Skå-Edeby, Sweden

In 1944, the Swedish Geotechnical Institute (SGI) was involved in the search of a place for a new international airport outside of Stockholm. The primary site in the investigation was Lilla Mellösa, however, the soil was very compressible with large thicknesses of soft clay. SGI decided to build two test embankments to consolidate the soil in advance and study the consolidation behavior. The first embankment was installed with vertical drains and the second without. The site was dismissed with time, but left for further geotechnical investigation. In 2008, Gündüz compared the field data collected with FEA using PLAXIS.

The soil profile at Lilla Mellösa consisted of an organic subsoil at the top 0.9 ft (0.3 m), which was removed prior to construction of the embankments, followed by a 1.6 ft (0.5 m) thick dry clay crust. Below the dry crust were several layers of soft soil. Figure 2-6 summarizes the water content, density, organic content, undrained shear strength and effective vertical stress of the subsurface with depth at the Lilla Mellösa site (Gündüz, 2008).
After about 200 days, the embankment with the drains at Lilia Mellösa experienced approximately 2.3 ft (0.7 m) of settlement. At this point, fill was removed from the embankment; however, the settlements did not stop. The majority of the settlement before removal of the surcharge took place in the upper 16.4 ft (5 m) of the soil profile where the vertical drains were installed. In 2002, the total settlement was about 5.2 ft (1.6 m). Today, the upper layer of the deposit has stopped settling, while the lower layer continues to settle (Gündüz, 2008).

By 1966, the second embankment without the drains at Lilia Mellösa had settled about 4.6 ft (1.4 m) while experiencing excess pore pressures of 4.4 psi (30 kPa). In 1979, the total settlement increased to 5.4 ft (1.65 m) with remaining excess pore pressures of 2.9 psi (20 kPa). By 2002, the settlement was over 6.6 ft (2 m) while there were still excess pore pressures in the order of 1.7 psi (12 kPa). The current settlement rate is about 0.4 in. (10 mm) per year (Gündüz, 2008).
In 1956, the need for a new airport became more urgent. Skå-Edeby became a possible option due to the proximity to Stockholm; however, the soil conditions were unknown. In 1957, the SGI performed field tests to investigate the possibility of building a new airport and found a soft clay deposit similar to that found in Lilla Mellösa. Four circle shaped test fills or embankments were constructed to study the consolidation of the soft soil. Due to economic reasons, the site was abandoned but research continued. Two additional test embankments were added later on (Gündüz, 2008).

Four test fills, with diameters ranging between 115 and 230 ft (35 and 70 m), were constructed in 1957. Three of the four test fills contained vertical sand drains, similar to PV drains. A fifth test fill was constructed later on with fabricated vertical band (PV) drains. Figure 2-7 shows the relative location of each test with respect to the others.

The top 1.6 ft (0.5 m) of soil at the Skå-Edeby site is an overconsolidated dry crust; similar to what was found at Lilla Mellösa. Beneath the crust layer, there is a layer of high-plastic organic clay followed by postglacial clay and organic clay. Figure 2-8 summarizes the water content, undrained shear strength and effective vertical stress of the subsurface with depth at Skå-Edeby.
Figure 2-7: Test Areas at Skå-Edeby (Gündüz, 2008)
The test embankment at area 3 of Figure 2-7 experienced a settlement of about 5 ft (1.55 m) after 4 years. At that time, the upper 2.3 ft (0.7 m) was removed and continued to settle linearly with time. In 2002, the total settlement of area 3 was about 5.2 ft (1.6 m). The embankments in area 1, 2 and 5 had settled 4.1 ft (1.25 m), 3.9 ft (1.2 m) and 2.5 ft (0.9 m), respectively.

The embankment in area 4 was the only one on site without drains. The initial settlement of area 4 during construction was 0.2 ft (0.06 m). In 1972, the settlement was about 2.5 ft (0.75 m) with excess pore pressures of 2.9 psi (20 kPa). In 1982, the total settlement was at 3.1 ft (0.95 m) with excess pore pressures of 1.7 psi (12 kPa). In 2002, the settlement was 3.6 ft (1.1 m) while still experiencing excess pore pressures of 1.2 psi (8 kPa).

Using the data collected at both sites, Gündüz modeled the embankments using the FEA software PLAXIS. Gündüz determined the plane strain analysis with the
Soft Soil Creep (SSC) model representing the clay layers estimated the settlement behavior at both sites best.

The settlement estimated by PLAXIS and the observed field data for the vertical drained test embankment at Lilla Mellösa is graphed in Figure 2-9. The drains in the Lilla Mellösa site were only installed in the top layer of the deposit, making the consolidation process in the lower layer much slower and still ongoing today although the top layer is no longer settling. PLAXIS appears to be underestimating the field data between 100 and 1000 days by approximately 0.25 m (0.8 ft). In general, PLAXIS appears to be demonstrating the same overall trend seen in the field (Gündüz, 2008).

![Figure 2-9: Comparison between Settlements Obtained by PLAXIS and Field Data for the Drained test fill at Lilla Mellösa (Gündüz, 2008)](image-url)
Figure 2-10 shows the field settlement data for the undrained embankment at Lilla Mellösa, along with the data calculated by PLAXIS. The PLAXIS results appear to overestimate the amount of settlement (approximately 1 ft (0.3 m)) experienced between 1946 and 1972, however, still demonstrate the same general trend.

![Figure 2-10: Comparison between Settlements Obtained by PLAXIS and Field Data for the Undrained test fill at Lilla Mellösa (Gündüz, 2008)](image)

The excess pore pressure data obtained in the undrained embankment at Lilla Mellösa is plotted in Figure 2-11 with the values determined by PLAXIS. Due to the limited field data for the excess pore pressures, it is unknown how accurate PLAXIS modeled the data prior to around 1956. Between 1956 and 2000, PLAXIS appears to underestimate the excess pore pressures greatly.
Similar trends were found for the embankments at Skå-Edeby. Gündüz concluded the excess pore pressures obtained by PLAXIS are much different from those obtained in the field. The vertical settlement calculated in PLAXIS appears to match the general trends found in the field (Gündüz, 2008).

2.3.3 - Case History 3: Malaysia

Similar research was also performed by the Malaysian Highway Authority where 14 embankments were built on soft clay with different ground-improvement techniques on the Muar coastal plain, slightly north of the Malaysian North-South Expressway. One embankment was constructed to failure without any soil
improvement methods for the purpose of comparison. Another embankment was constructed with vertical band (PV) drains penetrating completely through the soft clay. The purpose of the study was to evaluate the effectiveness of vertical band drains in improving soft soil foundations subjected to embankment loading. The embankment was modeled using CRISP, a finite element analysis software package, to compare with the field results (Indraratna et al., 1994).

The embankment with the vertical band drains was constructed over a 4 month period with a maximum fill height of 15.6 ft (4.75 m). The fill used to construct the embankment was compacted to a unit weight of 159.15 pcf (25 kN/m³). The drains were spaced 4.3 ft (1.3 m) apart in a triangular pattern with a maximum drain length of 59 ft (18 m) (Indraratna et al., 1994).

The subsurface geology data at the site showed a 6.6 ft (2 m) layer of weathered crust above a 54 ft (16.5 m) layer of soft silty clay. The soft silty clay was divided into an upper very soft layer and lower soft silty clay. Below the clay layer was a peaty soil followed by a stiff sandy clay. It was determined that the unit weight was fairly consistent between 95 and 102 pcf (15 and 16 kN/m³) except at the topmost crust where the unit weight was closer to 108 pcf (17 kN/m³). The variations in water content, liquid and plastic limits, and consolidation parameters with depth are summarized in Figure 2-12. In addition to triaxial and oedometer consolidation laboratory testing, in situ field vane testing was performed to determine the undrained shear strength. An undrained shear strength of 167 psf (8 kPa) was found 9.8 ft (3 m) below the ground surface and continued to
increase linearly with depth. A dense sand layer was located below the clay (Indraratna et al., 1994).

Figure 2-12: Geotechnical Properties of Muar Clay (Indraratna et al., 1994)

The soft clay was extensively instrumented to monitor the lateral and vertical movements, along with the excess pore water pressures. Extensometers and settlement gages were used to measure the vertical settlements. Inclinometers were used to measure the lateral movements. Pneumatic piezometers were used to measure the pore pressures below the embankment (Indraratna et al., 1994).

The finite element program CRISP was developed at Cambridge University, incorporating the soil model and consolidation behavior of the soil to be carried
out in an undrained or drained analysis. Based on the available data collected, the modified Cam-Clay model was selected for the FEA with the parameters summarized in Table 2-1. The modified Cam-Clay model was previously proven accurate in predicting the deformation response of normally consolidated soils. Table 2-2 summarizes the construction phases modeled in CRISP (Indraratna et al., 1994).

Table 2-1: Modified Cam-Clay Parameters Used in CRISP (Indraratna et al., 1994)

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>k</th>
<th>λ</th>
<th>ε_movie</th>
<th>M</th>
<th>ν</th>
<th>Kp (× 10⁹ kN/m)</th>
<th>ηv (m/s)</th>
<th>kn (m/s)</th>
<th>ku (m/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-1.75</td>
<td>0.06</td>
<td>0.16</td>
<td>3.10</td>
<td>1.19</td>
<td>0.29</td>
<td>4.4</td>
<td>16.5</td>
<td>6.4 x 10⁻⁹</td>
<td>3.0 x 10⁻⁹</td>
</tr>
<tr>
<td>1.75-5.50</td>
<td>0.06</td>
<td>0.16</td>
<td>3.10</td>
<td>1.19</td>
<td>0.31</td>
<td>1.1</td>
<td>15.0</td>
<td>5.2 x 10⁻⁹</td>
<td>2.7 x 10⁻⁹</td>
</tr>
<tr>
<td>5.50-8.0</td>
<td>0.05</td>
<td>0.13</td>
<td>3.06</td>
<td>1.12</td>
<td>0.29</td>
<td>2.4</td>
<td>15.5</td>
<td>3.1 x 10⁻⁹</td>
<td>1.4 x 10⁻⁹</td>
</tr>
<tr>
<td>8.0-18.0</td>
<td>0.035</td>
<td>0.09</td>
<td>1.61</td>
<td>1.07</td>
<td>0.26</td>
<td>22.7</td>
<td>16.0</td>
<td>1.3 x 10⁻⁹</td>
<td>0.6 x 10⁻⁹</td>
</tr>
</tbody>
</table>

Table 2-2: Construction History of Embankment Used in CRISP (Indraratna et al., 1994)

<table>
<thead>
<tr>
<th>Stage (1)</th>
<th>Fill period (days) (2)</th>
<th>Fill thickness (m) (3)</th>
<th>Rate of filling (m/day) (4)</th>
<th>Rest period (days) (5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>1-14</td>
<td>0.0-2.57</td>
<td>0.18</td>
<td>14-105</td>
</tr>
<tr>
<td>2</td>
<td>105-129</td>
<td>2.57-4.74</td>
<td>0.09</td>
<td>129-present</td>
</tr>
</tbody>
</table>

Prior to executing the FEA, the following limitations were made: plane strain conditions were assumed and the stiffness of the drain was considered to be the same as that of the adjacent soil element. The effect of smearing on the drains could not be modeled, and therefore two extreme cases were performed: with perfect drains (no smearing, zero pore pressures along the drains) and without drains. After performing both FEA cases, an additional case was performed
where the excess pore pressures remaining at the drain boundaries were incorporated into the analysis (Indraratna et al., 1994).

Figure 2-13 presents the FEA results with the actual vertical settlement field measurements along the embankment centerline at various depths. The solid line represents the field measurements, the dashed line represents the FEA results with perfect drains and the dotted line represents the FEA results with the excess pore pressures along the drains incorporated in calculations. As seen in Figure 2-13, the drains with the excess pore pressures incorporated into calculations matches the field measurements very closely.

Figure 2-14 shows the lateral movements predicted by the FEA and field measurements. The solid line represents the field measurements made by the inclinometer, the dashed line represents the FEA results with perfect drains and the dotted line represents the FEA results with the excess pore pressures along the drains incorporated in calculations. The field measurements are considerably greater than the PLAXIS predictions.

In the short term, the assumption of perfect drains overestimates the vertical settlements but underestimates the lateral movement. Although the perfect drain model provides conservative results for the vertical settlements, the lateral movement is under-predicted which is of great concern. This study showed that unless the excess pore pressures along the drain boundaries are correctly accounted for, the vertical settlements and lateral displacements cannot be predicted accurately. If the excess pore pressures along the drains are
accounted for, the 2D FEA can provide acceptable predictions for vertical and lateral movements in the soft clay due to embankment loading.

Figure 2-13: Horizontal Deformation Comparison of FEA with Field Measurements along embankment centerline (Indraratna et al., 1994)
2.3.4 - Conclusions

Previous research has been performed on the preloading effects on soft clays. Vertical drains are commonly used to accelerate the rate of consolidation when combined with embankment loading. All three case histories showed initial
development of excess pore pressures in the soft soil deposit from the embankment. With the assistance of drains, the pore pressures were able to dissipate faster than those without. The excess pore pressures cause lateral movement within the soil.

Gündüz (2008) and Indraratna et al. (1994) modeled the embankment loadings using different FEA software packages with a plane strain analysis. The specific soil model used to represent the soft clay deposit was selected based on the known available geotechnical parameters. According to Indraratna et al. (1994), unless the drains are modeled with non-zero excess pore pressures, the lateral and horizontal displacements cannot accurately be predicted and will fall between the extreme cases (with and without drains).
Chapter 3 - LABORATORY AND IN SITU TESTING SYSTEMS AND PROCEDURES

3.1 - Introduction

Various testing was performed in and on the marine deposit before and after construction of the embankment. Prior to construction, laboratory consolidation testing, standard penetration testing, dilatometer testing, field vane testing and piezocone testing were performed throughout the proposed footprint of the embankment. After construction, dilatometer and piezocone testing were performed. This chapter will discuss the different methods of laboratory and in situ testing implemented at the site and how they were performed.

3.2 - Consolidation Laboratory Testing

3.2.1 - Piston Sampling

Prior to the UNH in situ testing, samples of the marine clay were obtained for laboratory consolidation testing using a stationary piston sampler and Shelby tube assembly. A piston sampler was used over the conventional push sampler because it prevents debris from entering the tube during lowering of the sampler to the sampling point, controls the entry of the soil during collection and holds on to the sample via suction during tube retrieval (DeGroot and Ladd, 2012).
Figure 3-1 shows the piston sampler with Shelby tube assembly used in this project. After casing the borehole down to the top of the clay layer, the sampler was slowly lowered into place. Next, the Shelby tube was pushed ahead of the piston, which remained stationary, creating suction on the sample. It is important to note that although the Shelby tube is 30 in. (0.76 m) in length, it can only be pushed 24 in. (0.6 m) because of the top 6 in. (0.15 m) piston. After waiting approximately 15 minutes, the sampler was rotated 2 revolutions and then lifted slowly out of the ground. The piston was carefully removed from the tube and the top and bottom of the tube were sealed with paraffin. Leaves were used to fill any gap at the top of the tube. The samples were carefully transported back to the laboratory in a wooden stand. The samples were stored upright in the laboratory at constant room temperature until testing.
3.2.2 - **Consolidation Testing**

Consolidation testing was performed using a Geocomp LoadTrac-II system. The Geocomp ICONP software was used to control, monitor and record the testing. Sample preparation was performed following ASTM D2435 One-Dimensional Consolidation Properties of Soils Using Incremental Loading standard and the procedures outlined in the Geocomp manual.

Each specimen was cut from the Shelby tube using a pipe cutter as seen in Figure 3-2. The specimen was cut 3 in. (0.08 m) in thickness. Immediately after cutting, the specimen was extruded directly from the Shelby tube using the method of DeGroot and Ladd (2004) for debonding the sample from the tube to avoid significant disturbance during extrusion. They recommend using a thin piano wire to remove the bond between the sample and tube. A method similar to that shown in Figure 3-3 was followed for this testing. They also recommend not to take the specimen from the top and bottom 1 to 1.5 times the tube diameter due to the higher potential for greater degree of disturbance that usually occurs at the ends (DeGroot and Ladd, 2004).
After the tube was cut, it was placed on its side and secured by a chain clamp as seen in Figure 3-4. A hypodermic needle was threaded through the edge with the wire saw attached. Next, the wire of the saw was fastened tightly to the other side and rotated around the specimen until it appeared to be detached from the tube. The specimen was then extruded carefully with minimal disturbance.
(1) Cut tube and soil with horizontal band saw

(2) \( w_s \) and 2 +/- Torvane tests, remove clay, seal with wax (50/50 paraffin and petroleum jelly: LaRochelle et al. 1988) and caps

(3) Debond with piano wire, square ends and extrude

(4) Trim specimen, \( w_s \) on mixed soil for Atterberg Limits, etc.

Figure 3-3: Procedure for Obtaining Test Specimen from Tube Sample
(DeGroot and Ladd, 2004)

Figure 3-4: Cut Section of Tube Sample in Chain Clamp
A consolidation ring was pushed through the sample so that there was excess clay on both sides. It should be noted that vacuum grease was applied to the inside of the consolidation ring to assist in pushing the sample out of the ring after testing, as recommended by the Geocomp ICONP User's Manual (2008). Using the wire saw, the excess clay was trimmed off the ring. These trimmings were used to determine an initial water content of the specimen. The various components for the consolidation testing are shown in Figure 3-5. After the specimen was prepared, the large saturated porous stone (1) was placed at the base of the consolidometer (2), followed by a filter paper (3), sample (4), second piece of filter paper (5) and a small saturated porous stone (6). The consolidometer was placed on the platen with the loading plate and ball (7) on top.

Using the Geocomp software, the initial mass and dimensions of the sample were entered along with the loading steps. Each loading step was set to run for a minimum of 2 hr and maximum of 24 hr. The initial load of the combined loading plate and ball were also entered in the software to allow adjustments to be made to the applied stress. Once the data was entered, the test immediately began. While the seating load was applied, the consolidometer was filled with water to maintain full saturation. Complete testing took approximately 32 hours. After testing, a final moisture content of the sample was determined using the entire specimen. The initial and final water contents were entered into the software and consolidation calculations were performed by the software.
A total of 16 tests were performed at various elevations ranging between 0.5 and -53.6 ft (0.15 and -16.34 m) using the samples taken from boring Q-B212 during Phase 1. In Appendix A, Table A-1 summarizes the results found during testing.

3.3 - Standard Penetration Testing

In addition to the in situ and laboratory testing performed by UNH, the NHDOT completed 13 boreholes with continuous Standard Penetration Tests (SPT) throughout the site. The boreholes were advanced using case and wash drilling techniques. The SPT were carried out using an automatic hammer, a 24 inch (0.6 m) split-spoon sampler driven in 6 inch (0.15 m) increments with a 140 lb hammer dropped from 30 in. (0.76 m). The tests were performed beginning at the ground surface until refusal at the dense glacial till layer or bedrock.
these tests the NHDOT was able to determine a general profile of the subsurface conditions which will be discussed in Chapter 4.

3.4 - Dilatometer Testing

3.4.1 - Background

The dilatometer test (DMT), originally developed in Italy by Silvano Marchetti and introduced to North America and Europe in 1980, was used to evaluate the soils at the site in terms of strength parameters, soil behavior, stress history and soil stratigraphy. Figure 3-6 is a photograph of the flat dilatometer blade and membrane (Marchetti et al., 2001).

Figure 3-6: Flat Dilatometer Blade (Marchetti et al., 2001)
3.4.2 - Procedure

The dilatometer consists of a stainless steel blade with an 18° wedge tip and flexible steel membrane of 2.54 in. (60 mm) diameter located on one side. The blade is about 3.74 in. (95 mm) wide and 0.59 in. (15 mm) thick. The blade is connected to a cable (pre-threaded through the rods) which is attached to a pressure gauge and control unit for testing. The control unit is shown in Figure 3-7. The dilatometer testing was performed following ASTM D6635 Standard Test Method for Performing the Flat Plate Dilatometer. The test is both quick and economical.

Typical testing procedures consist in initial calibrations to correct for membrane stiffness in air, \( \Delta A \) and \( \Delta B \). \( \Delta A \) is determined by applying suction to the membrane and \( \Delta B \) by pressurizing the membrane in air until a displacement of 0.04 in. (1.1 mm) is reached. Additionally, the gauge offset \( (Z_M) \) for both the low
and high gauges are recorded representing the gauge pressure deviation from zero when vented to atmospheric pressure. The test begins by pushing the dilatometer blade into the ground using a drill rig at a rate of approximately 0.8 in./s (2 cm/s). For the testing at the site, the push stopped at 0.5 ft (0.15 m) intervals to record the A, B and C readings.

The A reading refers to the point when the membrane is inflated pneumatically with nitrogen gas until it becomes flush with the face of the blade and begins to overcome the horizontal stress in the ground. The B reading refers to the point when the membrane has moved 0.04 in. (1.1 mm) outward from the center. The C reading refers to the point when the membrane returns back to the initial position after controlled deflation. Figure 3-8 and 3-9 depict the working principle of the system and test layout, respectively.

After each test, the dilatometer was pushed to the next depth and the test procedure was repeated. During Phase 1, one continuous profile was performed at borehole Q-B213 between the elevations of 11.42 and -63.4 ft (3.48 and -19.33 m), every 6 in. (0.15 m). During Phase 2, 4 continuous profiles were performed every 6 in. (0.15 m).

Additionally, at depths where the coefficient of consolidation was being evaluated, A-method dissipation tests were performed by taking a series of A readings immediately and repeatedly for up to 24 hr (Marchetti et al., 2001). The dissipation data was analyzed using the Marchetti DMT Dissip software.
WORKING PRINCIPLE

Figure 3-8: DMT Working Principle (Marchetti et al., 2001)
During Phase 1, 15 dissipation tests were performed every 3.28 ft (1 m) between the elevations of 0 and -60.5 ft (0 and -18.45 m) at borehole Q-B215 (Segment 1). Four dissipation tests were performed every 10 ft (3.05 m) between elevations 0.3 and -29.7 ft (0.09 and -9.05 m) at borehole Q-B218 (Segment 1). In between dissipation tests, DMT tests were performed every 6 in. (0.15 m) to use in comparison with the readings recorded in borehole Q-B213 (Segment 1).
During Phase 2, 10 dissipation tests were performed every 5 ft (1.5 m) between the elevations of -4.3 and -51.3 ft (-1.3 and -15.6 m) at sta. 603+40 RT 5 (Segment 1).

Repeatability of the DMT was tested in Phase 1 by performing tests closely spaced together, and producing similar trends and values. The results shown in Figure 3-10 confirm the data is reliable as indicated by the nearly identical curves for $p_0$, $p_1$ and $p_2$ within the marine deposit for Q-B213 and Q-B215 (approximately 16 ft (4.9 m) apart). The marine deposit begins around 20 ft (6.1 m) below the surface and ends around 62 ft (18.9 m) and appears quite homogenous with a few interspersed sand layers.
Figure 3-10: Segment 1 DMT Repeatability for Q-B213 and Q-B215
3.4.3 - Field Data Reduction

Using Marchetti’s SDMT Elab, the A, B and C readings, and calibration information was inputted into the software which then produced graphs and tables of various empirically derived geotechnical parameters with respect to depth.

The software begins by correcting A, B and C for membrane stiffness and zero offset which correspond to $p_0$, $p_1$ and $p_2$, respectively (Marchetti et al., 2001).

\[
\begin{align*}
    p_0 &= 1.05(A - Z_M + A) - 0.05(B - Z_M - B) \quad [B-1] \\
    p_1 &= B - Z_M - B \quad [B-2] \\
    p_2 &= C - Z_M + A \quad [B-3]
\end{align*}
\]

After $p_0$, $p_1$ and $p_2$ are calculated, the DMT index parameters are evaluated along with other geotechnical soil parameters. The index parameters include: the material index ($I_D$), horizontal stress index ($K_D$) and dilatometer modulus ($E_D$). The soil type can be identified based on the material index. It is important to note that $I_D$ is a reflection of mechanical behavior and may not accurately identify the actual soil type (Marchetti, 1980).

\[
I_D = (p_1 - p_0)/(p_0 - u_0) \quad [B-4]
\]
The horizontal stress index provides the basis for soil parameter correlations. When plotted with depth, the horizontal stress index provides a similar shape to the overconsolidation ratio (Marchetti, 1980).

\[ K_D = \frac{(p_0 - u_0)}{\sigma'_{v0}} \]  \[ \text{[B-5]} \]

The dilatometer modulus is based on the theory of elasticity and should not be used on its own in analysis due to the lack of information on stress history (Marchetti, 1980).

\[ E_D = 34.7(p_1 - p_0) \]  \[ \text{[B-6]} \]

Figure 3-11 shows an example of the dilatometer readings, along with the corresponding material index, horizontal stress index and dilatometer modulus.

DMT dissipation tests were analyzed using the Marchetti DMT Dissip software. The software plots the A reading against the logarithm of time. The resulting curve should take on an “S”-shape and approach hydrostatic pressure at the end of the test. Next the contraflexure point and corresponding time \( t_{\text{flex}} \) are automatically determined by the software as illustrated in Figure 3-12. Using the \( t_{\text{flex}} \), the horizontal coefficient of consolidation can be estimated.
Figure 3-11: Segment 1 Q-B213 (Sta. 603 + 28, LT 14) DMT Readings, Material Index Classification ($I_D$), Horizontal Stress Index ($K_D$) and Dilatometer Modulus ($E_D$)
DISSIP DEPTH = 6.098 m

Uo,equil = 45.713 kPa
Tflex = 7.6 min

Figure 3-12: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation 0 ft

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{T_{flex}} = 0.015 \text{ cm}^2/\text{sec} \]
3.5 - Field Vane Testing

3.5.1 - Background

The Field Vane Test (FVT) is used to determine the undrained shear strength ($s_u$) of saturated clays and can be performed by following ASTM D2573 Standard Test Method for Field Vane Shear Test in Cohesive Soil. The field vane test has been used since the 1950's when Cadling and Odenstad introduced it to measure the in situ undrained shear strength of fine grained soils for slope stability evaluation. The test consists in pushing a thin bladed vane into the soil, rotating the vane and measuring the torque (Aas et al., 1986). Vanes come in various sizes, shapes and configuration for different soil consistencies and strength.

A Geonor H-10 Vane Borer was used in this study, featuring a vane blade 2.6 in. (6.5 cm) in diameter, 5.1 in. (13 cm) in height and a blade thickness ($e$) of 0.08 in. (0.2 cm). ASTM requires the vane to have a height to diameter ratio between 1 and 2.5 (13/6.5 = 2). This specific vane slides in and out of a housing unit, minimizing any excess friction and allowing a thinner vane to be used for testing. The equipment used during this test can be seen in Figure 3-13. The vane borer instrument in the top left corner of the photograph was used to apply the torque to the system.
The test was performed after the borehole was cleaned to the desired testing depth. The vane borer assembly was then placed into the borehole and pushed slightly above the test depth. It is recommended that testing be carried out at least four borehole diameters below the base of the borehole to avoid potential disturbance from drilling. Next, the vane was pushed out of the housing unit into the marine clay a distance of 1.6 ft (50 cm). After the vane is pushed into the soil the test must begin within 1 to 5 minutes to avoid significant dissipation of excess pore pressures which could lead to strength increases of up to 50% (Whittle et al., 1990). The vane borer instrument was then attached to the rods and secured in place. The test proceeds by rotating the vane at a rate of 0.1°/second. Every 15 seconds the torque reading was recorded. This procedure was followed until the soil failed to evaluate the undisturbed undrained shear strength. To assess
the clay sensitivity, which is the ratio of undisturbed to remolded strength, the soil was remolded by rotating the vane clockwise 10 revolutions. The test was performed again until a maximum value was reached; this represented the remolded strength value.

3.5.2 - Field Data Reduction

A typical test is shown in Figure 3-14 where the shear stress increases with angular rotation until a maximum point where it suddenly drops and levels off.

Figure 3-14: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -15.94 ft
The undrained shear strength, remolded strength and sensitivity are calculated as follows for systems with a height to diameter ratio of 2 (NHI, 2002):

\[
S_u = \frac{6T_{\text{max}}}{7\pi D^3}
\]  

[B-7]

Where \( T_{\text{max}} \) is the maximum recorded torque and \( D \) is the diameter of the vane.

\[
S_t = \frac{S_{u,\text{peak}}}{S_{u,\text{remolded}}}
\]  

[B-8]

Results for Phase 1 can be found in Appendix D.

### 3.6 - Piezocone Testing

#### 3.6.1 - Background

The piezocone (CPTu) can be used in soil profiling and for predicting numerous geotechnical parameters for a wide range of soils. The piezocone test consists of a cylindrical device, 1.44 in. (3.66 cm) diameter with a 60 degree conical tip, pushed into the ground at a standard rate of 0.8 in./s (2 cm/s). The probe is instrumented and allows the measurement of the resistance to penetration, friction on the sleeve and penetration pore pressure during the continuous push. Using ASTM D5778 Standard Test Method for Electronic Friction Cone and Piezocone Penetration Testing of Soils, the collected data can be used to estimate various geotechnical soil parameters.
3.6.2 - Procedure

For this research, a Hogentogler (HT) 10 Ton Cone Penetrometer was used. Since the purchase of the cone, Applied Research Associates (ARA)/VERTEK have acquired the Cone Penetrometer Testing (CPT) division of Hogentogler & Co., Inc. ARA/VERTEK was contacted before testing to upgrade to a digital cone penetrometer providing immediate results in the field. The probe consists of a dual element strain gauge transducer, with the cone wiring packaged behind the transducers. Six channels are available for reading: tip resistance, sleeve resistance, pore water pressure, inclination, temperature, and seismic. The temperature and seismic channels were not used during testing. Pore water pressure was measured in the U2 position behind the tip as seen in Figure 3-15.

Before testing, the cone cable was strung through the cone rods based on the anticipated final depth of penetration. The digital system was set up as seen in Figure 3-16. Figure 3-17 show the depth transducer mounted to the drill rig which monitors the actual depth and penetration rate.

Prior to testing, the piezocone porous element and fluid cavity need to be fully saturated. Poor saturation can result in inaccuracy and lag response in pore water pressure measurements and therefore, saturation is an important step in the pre-testing process. Using a funnel, the cone was threaded through, tip upward, and de-aired water was poured into the funnel until it entirely covered the cone. Next the tip was detached and a syringe was used to evacuate air bubbles from the inside. When the cone appears to be fully saturated, the
A porous filter was attached to the cone and the tip screwed back in place. A prophylactic membrane was used to cover the cone to help maintain saturation prior to inserting the probe into the ground. The prophylactic breaks once the cone is pushed into the ground and does not interfere with testing.

Figure 3-15: Terminology for cone penetrometers (Robertson and Gregg Drilling, 2006)

For testing, the cone was placed in a predrilled hole through the upper fill and below the water table. With the tip above the base of the hole, initial baseline readings were recorded. The test began following this procedure and the data was recorded using the computer cone software (Vertek's Digital Cone software version 2011.10.28.1).

After testing is completed, a final baseline should be recorded. The baseline should not be recorded while the cone is in contact with soil, instead while it is below the water table within the casing. If the test will be performed again at the same site, it is important to keep the cone saturated. Ten profiles were performed
during Phase 1; however, the last profile did not result in workable data and will
not be discussed in this thesis. Table 3-1 summarizes the location and total run
length of each profile tested during Phase 1.

Figure 3-16: Hogentogler (HT) 10 Ton Cone Penetrometer Setup
Figure 3-17: Depth transducer setup

Table 3-1: Summary of Phase 1 CPTu Testing

<table>
<thead>
<tr>
<th>Borehole</th>
<th>Location</th>
<th>Segment</th>
<th>Total Run Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q-B220</td>
<td>Sta. 605+37, LT 25</td>
<td>1</td>
<td>70.9</td>
</tr>
<tr>
<td>Q-B221</td>
<td>Sta. 603+47, RT 11</td>
<td>1</td>
<td>69.2</td>
</tr>
<tr>
<td>Q-B222</td>
<td>Sta. 604+32, LT 6</td>
<td>2</td>
<td>73.8</td>
</tr>
<tr>
<td>Q-B223</td>
<td>Sta. 605+04, LT 6</td>
<td>2</td>
<td>66.3</td>
</tr>
<tr>
<td>Q-B224</td>
<td>Sta. 606+75, CL</td>
<td>3</td>
<td>74.0</td>
</tr>
<tr>
<td>Q-B225</td>
<td>Sta. 606+86, CL</td>
<td>3</td>
<td>76.8</td>
</tr>
<tr>
<td>Q-B226</td>
<td>Sta. 609+03, LT 10</td>
<td>4</td>
<td>77.0</td>
</tr>
<tr>
<td>Q-B227</td>
<td>Sta. 609+09, CL</td>
<td>4</td>
<td>80.4</td>
</tr>
<tr>
<td>Q-B228</td>
<td>Sta. 611+00, LT 6</td>
<td>5</td>
<td>77.0</td>
</tr>
</tbody>
</table>
Five profiles were performed during Phase 2, one at each of the five segments tested during Phase 1. Table 3-2 summarizes the location and total run length of each profile tested during Phase 2.

### Table 3-2: Summary of Phase 2 CPTu Testing

<table>
<thead>
<tr>
<th>Location</th>
<th>Segment</th>
<th>Total Run Length (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sta. 600+45 CL</td>
<td>1</td>
<td>65.1</td>
</tr>
<tr>
<td>604+90 CL</td>
<td>2</td>
<td>67.2</td>
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<tr>
<td>Sta. 605+30 CL</td>
<td>3</td>
<td>66.2</td>
</tr>
<tr>
<td>Sta. 609+10 RT</td>
<td>4</td>
<td>65.8</td>
</tr>
<tr>
<td>Sta. 611+65 CL</td>
<td>5</td>
<td>75.8</td>
</tr>
</tbody>
</table>

During Phase 1, two CPTu profiles were performed in each segment. Figure 3-18 exhibits the repeatability which was found between each pair of CPTu profiles within each segment of the proposed embankment. Q-B220 and Q-B221 were performed approximately 11 ft (3.4 m) apart in Segment 1 and show similar trends and values as expected and observed with the DMT.

Results for Phase 1 can be found in Appendix F. Results for Phase 2 can be found in Appendix J.
3.6.3 - Field Data Reduction

Before analysis, the data recorded by the Digital Cone software (tip resistance ($q_c$), pore water pressure ($u_2$) and sleeve friction ($f_s$)) was opened in Vertek's
Digital Cleanup software. Digital Cleanup was used to adjust the data based on the initial and final baselines recorded. After cleanup, the file was opened in Vertek's Coneplot to convert the CPT file to a text file. The text file was copied into Excel where the depth measurements were corrected due to the slight angle of the depth transducer on the rig. Adjustments were made based on the manual depth measurements recorded in the field.

Lastly, a software package titled CPeT-IT by GeoLogismiki was used to analyze the corrected data and determine soil parameters. It is important to note that the CPTu cannot be expected to provide accurate predictions of soil type, but provide a guide to the strength and stiffness of the soil, or the soil behavior type (SBT). Detailed steps in the software analysis can be found in Appendix E.
Chapter 4 - SITE CHARACTERIZATION

4.1 - Introduction

The location of the proposed test embankment site is circled in Figure 4-1 at the intersection of Route 4 and the Spaulding Turnpike on the eastern side of the existing Exit 6 SB On-Ramp to the Spaulding Turnpike in Dover, NH. The embankment is approximately represented by the yellow rectangle parallel to the existing On-Ramp. To the west of the site is the Little Bay and to the east is the Piscataqua River, both influenced by tidal bodies of water. In addition to the tidal influences, the site is a designated wetland preventing the site from draining quickly. According to the NHDOT, prior to Phase 1 the groundwater levels varied between elevations 6.2 and -3.5 ft (1.9 and -1.1 m) (Geotechnical Test Embankment Recommendations Report, 2012). The site is generally flat and wooded.

As mentioned previously, the marine deposit is found throughout the general Seacoast area. Table 4-1 summarizes previous research performed on the deposit that will be referenced throughout the remainder of this thesis. Figure 4-2 highlights the general location of the test sites listed in Table 4-1.
Figure 4-1: NHDOT Newington-Dover Site

Table 4-1: Summary of Previous Testing Performed

<table>
<thead>
<tr>
<th>Reference</th>
<th>Location</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ladd et al. (1972)</td>
<td>Portsmouth, NH</td>
</tr>
<tr>
<td>NeJame (1991)</td>
<td>Pease Air Force Base (PAFB) Portsmouth, NH</td>
</tr>
<tr>
<td>UNH CIE 961 (1997)</td>
<td>Scammell Bridge Dover, NH</td>
</tr>
</tbody>
</table>
As mentioned in Chapter 2, Ladd (1972) performed research on the embankments built on the marine deposit in Portsmouth. Ladd (1972) found a 30 to 35 ft (9.1 to 10.7 m) thick layer of gray silty clay directly below a thin organic layer. The top 5 to 8 ft (1.5 to 2.4 m) of the clay appeared weathered and firm while the lower layer was weak and extremely sensitive. Next, a 5 ft (1.5 m) layer of loose silt and sand followed by a 5 ft (1.5 m) layer of soft silty clay was encountered. Lastly, a layer of till was found.
Findlay (1991), NeJame (1991) and Murray (1995) found similar stratigraphy for PAFB. Findlay (1991) performed self-boring pressuremeter tests and NeJame (1991) performed dilatometer tests at PAFB. Murray (1995) performed Wissa piezocone, dilatometer and field vane testing in two adjacent locations, approximately 328 ft (100 m) apart. The first location was in the middle of a swamp, designated as the K_0 area. The second location was at the toe of the embankment and designated as the non-K_0 area. The K_0 area consisted of approximately 1 to 2 ft (30 to 60 cm) of surface water over a layer of organic muck. Below the 1 ft (30 cm) layer of organic muck, a thick deposit of gray marine illitic silty clay was found extending up to 26.2 ft (8 m), followed by glacial till and bedrock. The top 5 to 8 ft (1.5 to 2.5 m) of the clay was weathered and stiff, while the lower portion was soft and highly sensitive.

In 1997, dilatometer and field vane testing was performed by the UNH CIE 961-In Situ Geotechnical Testing class in Dover, NH for the Scammell Bridge which crosses the Bellamy River. The soil profile from a nearby boring consisted of estuarine silt and sand for the top 15.1 ft (4.6 m), followed by 50 ft (15.2 m) of glaciomarine silty clay. The clay overlay 30 ft (9.1 m) of glacial till. The stratigraphy was confirmed after testing was performed.

4.2 - Subsurface Characterization

As mentioned previously, the NHDOT performed SPT at the site to help define the stratigraphy. The top layer consists of a fill which is a mixture of loamy topsoil, fine sand, medium to fine sand and sandy silt followed by an alluvium
layer with mainly fine sand with variable amounts of silt. Below the alluvium is the marine deposit with a stiff top layer followed by a thick very soft sensitive clay. The marine deposit contains variable amounts of silt and clay with scattered fine sand layers. Using samples recovered from the SPT tests, the NHDOT performed Atterberg Limit tests and water content determinations for the marine deposit. A total of 48 tests were performed from various borings and depths. These values were considered similar to that of a marine illitic clay.

Below the marine deposit, a glacial outwash was found that generally consisted of fine sand with silt, sand and gravel particles overlying a dense glacial till layer generally comprised of silty fine sand or fine sandy silt with medium-coarse sand and gravel particles. Below the till is a schist bedrock. Figure 4-3 shows the general cross-section of the subsurface conditions. The marine deposit is generally thicker than that found in Portsmouth, PAFB and Dover, NH.

The site is located within the Eliot formation. According to Novotny (1969), metamorphism converted the original sedimentary materials to slate, phyllite, micaceous schists and poorly recrystallized pyritic quartzite. The original sedimentary materials consisted of thin-bedded deposits of clays, silts and fine sands with some calcium carbonate.
4.3 - Soil Properties

Both Ladd (1972) and Findlay (1991) performed Atterberg Limit and consolidation testing on the marine deposit found in Portsmouth. Table 4-2 summarizes the soil properties of the marine deposit found by Ladd (1972) and Findlay (1991) for the Atterberg Limit testing.
Previous to the in situ and laboratory testing performed by UNH, the NHDOT had performed Atterberg Limit tests from the samples obtained by the Standard Penetration Testing. The samples used were obtained at various depths and in different borehole locations at the site. A summary of the average values at each test depth are given in Table 4-3. Figure 4-4 graphically displays the Atterberg Limit results along with the moisture contents obtained from the UNH consolidation testing. The samples obtained for testing were collected during Phase 1 at borehole Q-B212.

The water contents determined by UNH appear to be slightly less than those found by the NHDOT. Findlay's (1991) water content results appear to be representative of the overall average of UNH and NHDOT's findings. In general, the water content is greater than the LL, a further indication of the sensitivity of the clay. Both the Liquid Limit and Plastic Limit appear to be within similar ranges of Ladd (1972) and Findlay's (1991) findings.
Table 4-3: NHDOT Atterberg Limit Summary

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>LL_{avg}</th>
<th>PL_{avg}</th>
<th>PI_{avg}</th>
<th>W_{avg} (%)</th>
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Figure 4-4: NHDOT Atterberg Limit Results, Q-B212 (Sta. 603 + 29, LT 1) UNH Laboratory Results and Q-B214 (Sta. 603 + 15, CL) FVT Results
Chapter 5 - TESTING PHASES

5.1 - Introduction

As mentioned previously, the testing was divided into three separate phases. The data collected during Phase 1, prior to construction of the embankment, was used in the settlement analyses. The data collected during Phase 2, approximately 4 months after construction was completed, was used to monitor strength changes of the marine deposit with time. Phase 3 will be performed prior to the use of the embankment in the construction of the future Exit 6 SB On-Ramp new alignment.

5.2 - Phase 1

During the summer of 2012, sampling and in situ testing were performed on the proposed highway embankment location. Undisturbed sampling, DMT and FVT testing were strictly performed in the main segment (Segment 1) of the embankment; CPTu testing was performed throughout Segments 1 through 5. Table 5-1 summarizes the testing performed during Phase 1. The borehole used in performing the FVT (Q-B214) remained open allowing water table measurements to be recorded during testing.

Table 5-2 summarizes the water table readings recorded during Phase 1 in Segment 1.
Table 5-1: Summary of Sampling and In Situ Testing During Phase 1

<table>
<thead>
<tr>
<th>Date</th>
<th>Borehole</th>
<th>Test Embankment Segment</th>
<th>Station</th>
<th>Surface Elevation (ft)</th>
<th>End Elevation (ft)</th>
<th>Field Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sampling:</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6/26/12 to</td>
<td>Q-B212</td>
<td>1</td>
<td>Sta. 603+ 29, LT 1</td>
<td>12.2</td>
<td>-57.7</td>
<td>Piston Sampling for Laboratory Consolidation Testing</td>
</tr>
<tr>
<td>7/2/12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Laboratory Testing:</td>
<td>11/8/12 to</td>
<td>Q-B213</td>
<td>1</td>
<td>Sta. 603 + 28, LT 14</td>
<td>12.4</td>
<td>-63.4</td>
</tr>
<tr>
<td>2/6/13</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/5/12 to</td>
<td>Q-B214</td>
<td>1</td>
<td>Sta. 603 + 15, CL</td>
<td>12.3</td>
<td>-52.06</td>
<td>FVT</td>
</tr>
<tr>
<td>7/6/12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/11/12 to</td>
<td>Q-B215</td>
<td>1</td>
<td>Sta. 603 + 40, LT 4</td>
<td>12.0</td>
<td>-60.9</td>
<td>DMT with Dissipation Tests every 3.28 ft (1 meter)</td>
</tr>
<tr>
<td>7/17/12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>7/12/12 to</td>
<td>Q-B218</td>
<td>1</td>
<td>Sta. 603 + 35, LT 25</td>
<td>12.3</td>
<td>-29.7</td>
<td>DMT with Dissipation Tests every 10 ft</td>
</tr>
<tr>
<td>7/31/12</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>8/3/12</td>
<td>Q-B220</td>
<td>1</td>
<td>Sta. 603 + 37, RT 16</td>
<td>11.9</td>
<td>-59</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/6/12</td>
<td>Q-B221</td>
<td>1</td>
<td>Sta. 603 + 47, RT 11</td>
<td>12.4</td>
<td>-56.8</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/7/12</td>
<td>Q-B222</td>
<td>2</td>
<td>Sta. 604 + 92, LT 6</td>
<td>11.9</td>
<td>-61.9</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/7/12</td>
<td>Q-B223</td>
<td>2</td>
<td>Sta. 605 + 04, LT 6</td>
<td>11.9</td>
<td>-54.4</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/7/12</td>
<td>Q-B224</td>
<td>3</td>
<td>Sta. 606 + 76, CL</td>
<td>10.7</td>
<td>-64.2</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/8/12</td>
<td>Q-B225</td>
<td>3</td>
<td>Sta. 606 + 86, CL</td>
<td>10.5</td>
<td>-66.3</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/9/12</td>
<td>Q-B226</td>
<td>4</td>
<td>Sta. 609 + 05, LT 10</td>
<td>10.6</td>
<td>-66.4</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/8/12</td>
<td>Q-B227</td>
<td>4</td>
<td>Sta. 609 + 09, CL</td>
<td>10.4</td>
<td>-70</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/9/12</td>
<td>Q-B228</td>
<td>5</td>
<td>Sta. 611 + 00, LT 6</td>
<td>9.9</td>
<td>-67.1</td>
<td>CPTu</td>
</tr>
</tbody>
</table>

70
Table 5-2: Water Table Readings during Phase 1

<table>
<thead>
<tr>
<th>Date</th>
<th>Depth to Water (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7/12/2012</td>
<td>1.4</td>
</tr>
<tr>
<td>7/20/2012</td>
<td>3.8</td>
</tr>
</tbody>
</table>

Location: 603 + 15 CL El. 12.3 ft

5.3 - Phase 2

Prior to construction, all vegetation in the footprint of the embankment was removed, followed by the installation of the sand drainage blanket 1 ft (0.3 m) thick. The PV drains were installed in a triangular spacing pattern as shown in Figure 5-1 and summarized in Table 5-3. All PV drains penetrated completely through the marine clay layer except those in Segment 3 which stopped short of the glacial till by about 10 ft (3 m). Next, the NHDOT instrumentation was installed, including: settlement platforms, inclinometers, vibrating wire piezometers and stable benchmarks. Lastly, the embankment was constructed to the appropriate fill height and grade.

Table 5-3: Summary of Construction Segments

<table>
<thead>
<tr>
<th>Station</th>
<th>PV-Drain Spacing (ft)</th>
<th>Fill Height (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Segment 600+00 to 602+00</td>
<td>8</td>
<td>-</td>
</tr>
<tr>
<td>Segment 1 602+00 to 604+00</td>
<td>6</td>
<td>12</td>
</tr>
<tr>
<td>Segment 2 604+00 to 606+00</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>Segment 3 606+00 to 608+00</td>
<td>10</td>
<td>12</td>
</tr>
<tr>
<td>Segment 4 608+00 to 610+00</td>
<td>10</td>
<td>18</td>
</tr>
<tr>
<td>Segment 5 610+00 to 612+00</td>
<td>14</td>
<td>12</td>
</tr>
<tr>
<td>Final Segment 612+00 to 613+50</td>
<td>8</td>
<td>-</td>
</tr>
</tbody>
</table>
After construction, the NHDOT began taking readings of the installed instrumentation on a weekly basis. Phase 2 of the UNH testing was performed during the spring of 2013, approximately 16 weeks after completion of the embankment fill. DMT and CPTu testing were performed at various locations throughout the site. Originally, two DMT profiles and a dissipation test were to be performed, however, due to a leak in the system, additional testing was performed to assess the effect of the leak. After comparing the profiles before and after the leak was fixed, it was confirmed the small leak did not affect the testing results. Chapter 8 will further discuss the DMT leak. Table 5-4 summarizes the location of each completed profile.
A well was installed in the piezocone hole from Segment 5 to measure the water table during the remainder of testing. The results are shown in Table 5-5 and indicate the water table to be at a depth near the original ground surface.

Table 5-4: Summary of In Situ Testing During Phase 2

<table>
<thead>
<tr>
<th>Date</th>
<th>Embankment Segment</th>
<th>Station</th>
<th>Surface Elevation (ft)</th>
<th>End Elevation (ft)</th>
<th>Field Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>4/1/13</td>
<td>1</td>
<td>603 + 20, LT 5</td>
<td>23.4</td>
<td>-56.6</td>
<td>DMT</td>
</tr>
<tr>
<td>4/3/13 to 4/26/13</td>
<td>1</td>
<td>603 + 40, RT 5</td>
<td>23.7</td>
<td>-51.3</td>
<td>DMT with Dissipation Tests every 5 feet</td>
</tr>
<tr>
<td>5/7/13</td>
<td>4</td>
<td>609 + 00, RT 5</td>
<td>27.74</td>
<td>-59.76</td>
<td>DMT</td>
</tr>
<tr>
<td>5/22/13</td>
<td>1</td>
<td>603 + 24, LT 5</td>
<td>23.4</td>
<td>-60.6</td>
<td>DMT</td>
</tr>
<tr>
<td>5/23/13</td>
<td>1</td>
<td>603 + 25, LT 10</td>
<td>23.3</td>
<td>-59.7</td>
<td>DMT</td>
</tr>
<tr>
<td>5/8/13</td>
<td>4</td>
<td>609 + 10, RT 3</td>
<td>27.7</td>
<td>-62.4</td>
<td>CPTu</td>
</tr>
<tr>
<td>5/8/13</td>
<td>5</td>
<td>611 + 05, CL</td>
<td>21.71</td>
<td>-68.5</td>
<td>CPTu</td>
</tr>
<tr>
<td>5/10/13</td>
<td>3</td>
<td>606 + 80, CL</td>
<td>23.4</td>
<td>-62.1</td>
<td>CPTu</td>
</tr>
<tr>
<td>5/10/13</td>
<td>2</td>
<td>604 + 90, CL</td>
<td>24.06</td>
<td>-62.1</td>
<td>CPTu</td>
</tr>
<tr>
<td>5/20/13</td>
<td>1</td>
<td>603 + 45, CL</td>
<td>23.5</td>
<td>-60.2</td>
<td>CPTu</td>
</tr>
</tbody>
</table>
Table 5-5: Water Table Readings during Phase 2

<table>
<thead>
<tr>
<th>Date</th>
<th>Depth to Water (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5/17/2013</td>
<td>12.5</td>
</tr>
<tr>
<td>5/20/2013</td>
<td>12.4</td>
</tr>
<tr>
<td>5/22/2013</td>
<td>12.2</td>
</tr>
</tbody>
</table>

Location: 611 + 05 CL El. 21.71ft

5.4 - Phase 3

Future testing is planned to assess long-term strength change of the marine deposit with time.
Chapter 6 - INTERPRETATION OF TESTING RESULTS

6.1 - Introduction

After testing was completed, the data collected during Phase 1 was used to calculate the geotechnical parameters associated with the marine deposit. The test specific parameters were compared with other tests of the same type (i.e. DMT or CPTu) to determine the repeatability of the data. Next, the data was compared with previous research findings in the marine deposit found throughout the Seacoast area, specifically Portsmouth and Dover, NH. After confirming the repeatability of the Phase 1 data, each testing method was plotted for interpretation of the marine deposit behavior. Using the findings, settlement analyses were performed. This chapter will focus solely on the interpretation of Phase 1 data.

6.2 - Previous Work

Table 4-1 in Chapter 4 summarized the locations of previous testing performed in the Seacoast area that will be used to compare with the data collected at the Newington-Dover site.

NeJame (1991) had performed dilatometer tests at the Pease Air Force Base in Portsmouth, NH. Figure 6-1 presents his findings for one of the soundings while Figure 6-2 presents those same results but at the Newington-Dover site. Although the clay deposit at the PAFB site is not as thick as that found in the
Newington-Dover site, it is apparent that both sites demonstrate similar trends with respect to the material index ($I_D$), horizontal stress index ($K_D$) and dilatometer modulus ($E_D$). The Newington-Dover site shows a slightly lower $I_D$ in the clay layer than the PAFB indicating a behavior a bit closer to mud than soft clay. The $K_D$ appears to remain constant around a value of 5 at Newington-Dover which is in agreement with the PAFB data within the near normally consolidated portion of the clay layer. The $E_D$ appears to be about 30 ksf in the clay layer at both Newington-Dover and PAFB.

![Dilatometer Testing - Pease Air Force Base](image)

Figure 6-1: Typical DMT Indices at Pease Air Force Base for sounding PD6 (NeJame, 1991)
Figure 6-2: Typical DMT Indices at Newington-Dover Site
Murray (1995) performed piezocone testing at the Pease Air Force Base using a Wissa cone with the pore pressure element in the \(u_1\) position. His results in Figure 6-3 are compared with those found at the Newington-Dover site in Figure 6-4. Similar to NeJame's work, the results appear to demonstrate similar trends, most apparent in the corrected sleeve friction \((f_t)\) and corrected tip resistance \((q_t)\) plots. The friction sleeve value in the clay appears to be around 0.025 MPa at PAFB, slightly greater than the 0.01 MPa at Newington-Dover. The tip resistance appears to about 0.25 MPa at the beginning of the clay layer at both sites and slowly increases with depth at a rate of approximately 0.09 MPa/m.

As mentioned previously, the pore pressure element in the Wissa cone is in the \(u_1\) position and the pore pressure element in the subtraction piezocone is in the \(u_2\) position. Excess pore pressures at \(u_1\) are generally 10-20% greater than those in the \(u_2\) position. The pore pressures measured by the Wissa cone in the clay layer appear to fluctuate around 0.3 MPa. The pore pressures experienced by the subtraction piezocone in the same depth range appear to be around 0.35 MPa, which is not 10-20% less than the Wissa cone values. The excess pore pressures may not follow the expected trend because of the variation in the marine deposit between sites.
Figure 6-3: Piezocone Profile CPTu_M2 at Pease Air Force Base (Murray, 1995)

(1 m = 3.28 ft)
In 1997, the UNH CIE 961-In Situ Geotechnical Testing class performed dilatometer and field vane testing at the Scammell Bridge site in Dover, NH. Figure 6-5 and 6-6 also appear to demonstrate similar trends within the marine clay deposit. It is important to note the Scammell Bridge profile was performed below the mudline with water above. The clay layer in Newington-Dover and the
Scammell Bridge appear to have similar values (250 to 400 kPa in the beginning of the marine deposit), however, the Newington-Dover values reach greater values (700 kPa) in the marine deposit due to the greater depth.

Figure 6-5: Corrected Pressures versus Depth from Dilatometer Testing at Scammell Bridge (CIE 961, 1997)
Figure 6-6: Segment 1 Q-B213 (Sta. 603 + 28, LT 14) DMT Readings in Metric
6.3 - Total Unit Weight ($\gamma_t$)

The unit weight of the marine deposit was calculated as part of the consolidation testing, in addition to the estimates evaluated by the dilatometer and piezocone test results. Equation [6-1] was used to calculate the initial unit weight of the soil based on the DMT data. Table E-1 in Appendix E was used to estimate the unit weight of the soil based on the CPTu Soil Behavior Type. The piezocone, dilatometer and laboratory tests appear to give similar trends as shown in Figure 6-7 but shifted along the x-axis at different locations.

\[
\gamma_{T\,DMT} = 1.12 \gamma_w \left( \frac{E_D}{\sigma_{atm}} \right)^{0.1} (I_D)^{-0.05}
\]  

[6-1]

Where $\sigma_{atm}$ is atmospheric pressure, $\gamma_w$ is the unit weight of water, $E_D$ is the dilatometer modulus and $I_D$ is the material index (NHI, 2002).

Due to the direct testing of the unit weight during consolidation testing, the DMT and CPTu data was shifted to match the consolidation data as shown in Figure 6-8. The DMT data was shifted to the right by 10 pcf. The CPTu data was shifted to the right by 20 pcf. Equation [6-2] and [6-3] represent the site specific relationships for the density based on the initial DMT and CPTu equations.

\[
\gamma_{T\,Newington-Dover\,DMT} = \gamma_{T\,DMT} + 10\text{pcf}
\]  

[6-2]

\[
\gamma_{T\,Newington-Dover\,CPTu} = \gamma_{T\,CPTu} + 20\text{pcf}
\]  

[6-3]
The resulting total unit weights plotted in Figure 6-8 are within the range Findlay (1991) found as listed in Table 4-2 (110.1 ± 20.4 pcf). More narrowly, the total unit weight range found in Newington-Dover is between 107 and 120 pcf.

Figure 6-7: Total Unit Weight Found in Segment 1
Phase 1 Shifted

- Segment 1 Q-B212 (Sta. 603 + 29, LT 1) Consolidation Testing
- Segment 1 Q-B213 (Sta. 603 + 28, LT 14) DMT Shifted +10 pcf
- Segment 1 Q-B220 (Sta. 603 + 37, RT 16) CPTu Shifted +20 pcf

Figure 6-8: Shifted Total Unit Weight Found in Segment 1
6.4 - Undrained Shear Strength ($s_u$)

The field vane measures the undrained shear strength directly, whereas the dilatometer and piezocone can each estimate the undrained shear strength of the marine clay using various empirical correlations developed from a large database of clay deposits worldwide.

According to Chandler (1988), it is important to correct the vane strength prior to use in stability analysis involving embankments on soft ground, which is relevant to the Q-project. The following relationships were used based on the NHDOT Atterberg Limit data and the confirmation that the plasticity index (PI) was greater than 5%.

$$ S_u = \mu_R S_{u\,uncorrected} $$  \hspace{1cm} [6-4]

$$ \mu_R = 1.05 - b(PI)^{0.5} $$  \hspace{1cm} [6-5]

$$ b = 0.015 + 0.0075\log(t_f) $$  \hspace{1cm} [6-6]

Where $b$ is a rate factor and $t_f$ is the time to failure (Chandler, 1988).

Aas et al. (1986) recommends correcting the data for plasticity and the overconsolidation ratio. The soil was assumed to be normally consolidated, based on the data presented in Chapter 4, and the effective overburden pressure was based on the DMT results. Figure 6-9 was used to determine the correction value, $\mu_R$. If $\mu_R$ is greater than 1, 1 should be used as the correction value (Aas et al., 1986).
\[ S_u = \mu R S_{u \text{uncorrected}} \] [6-7]

Figure 6-9: Diagrams for determination of stress history and field vane correction factor, with some data from well documented case histories (Aas et al., 1986)

1 Bangkok (Eide & Holmberg, 1972)
2 Fiumicino (Calabresi & Burghignoli, 1977)
3 San Francisco Bay (Duncan and Buchignani, 1973)
4 Onsøy (Berre, 1973)
5 Kimola (Kankare, 1969)
6 Postgirò (Aas, 1979)
7 Malmo (Pusch, 1968)
8 Ellingsrud (Aas, 1979)
9-17 MIT cases (Lacasse et al., 1978)
15,16,17 no failure
Figure 6-10 shows the undisturbed, corrected and remolded shear strengths, in addition to the estimates generated by the DMT and CPTu. Equation [6-8] was used to estimate the shear strength based on the DMT data (Marchetti, 1980). Equation [6-9] was used to estimate the shear strength based on the CPTu data (Lunne et al., 1997). Further discussion on the determination of the undrained shear strength can be found in Appendix B and E. The undisturbed uncorrected shear strength is very similar to the corrected undisturbed shear strength using Chandler’s (1988) method. The DMT and CPTu appear to overestimate the strength determined by the vane, but seem to follow the same general trend.

\[
S_{udMT} = 0.22\sigma'_{v0}(0.5K_p)^{1.25} \times \text{for } I_D < 1.2 \\
S_{uCPTu} = \frac{q_t - \sigma_v}{N_{kt}}
\]  

[6-8]  

[6-9]

The CPTu and DMT data was corrected to fit the field vane data in Figure 6-11. The DMT data was corrected based on the findings of Roche, Rabasca and Benoît (2008) in Portland, ME. In Marchetti’s original equation (Equation [6-8]), a coefficient of 0.22 was used to represent the ratio of the undrained shear strength to the effective vertical overburden pressure. Using the findings specific to their site, the 0.22 was replaced by 0.137. The Newington-Dover site appears to best be represented by a coefficient of 0.13 as seen in Equation [6-10].

\[
S_{uNewington-Dover DMT} = 0.13\sigma'_{v0}(0.5K_p)^{1.25} \times \text{for } I_D < 1.2
\]  

[6-10]
The CPTu data was corrected as recommended by Murray (1995). Murray suggests the use of the Larsson and Mulabdić (1991) method where the cone factor \( (N_m) \) is estimated based on the liquid limit. The data was divided into
sublayers based on the OCR of the deposit to determine the cone factor. Table 6-1 summarizes the cone factors used to calculate the corrected profile. The updated cone factor was then used in Equation [6-9].

\[ N_{kt} = 13.4 + 6.65(LL) \]  

Table 6-1: Site Specific Cone Factor (N_{kt}) Based on Larsson and Mulabdić (1991)

<table>
<thead>
<tr>
<th>Elevation (ft)</th>
<th>( N_{kt} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>6.75 to -4.81</td>
<td>15.7</td>
</tr>
<tr>
<td>-4.81 to -15.86</td>
<td>15.8</td>
</tr>
<tr>
<td>-15.86 to -34.17</td>
<td>15.0</td>
</tr>
<tr>
<td>-34.17 to -41.72</td>
<td>15.9</td>
</tr>
<tr>
<td>-41.72 to -51.98</td>
<td>15.7</td>
</tr>
<tr>
<td>-51.98 to -58.98</td>
<td>15.3</td>
</tr>
</tbody>
</table>

In addition to laboratory testing, Ladd (1972) had performed shear vane tests using a Geonor Field Vane. Ladd (1972) does not mention specifics regarding the dimensions of the field vane used, so it will be assumed they were in compliance with the ASTM standard. Additionally, Ladd (1972) did not apply corrections to his data and will therefore be compared to the uncorrected values found in Newington-Dover. Figure 6-12 shows the undrained shear strength profile found by Ladd. Ladd's findings appear to be within the range of 200 to 400 psf. The uncorrected undisturbed shear strength of the marine deposit found in Newington-Dover is within the range of 200 to 600 psf between the same elevations. The Newington-Dover deposit extends deeper than that seen in Portsmouth. It is interesting to note that Ladd's data includes a jump in strength halfway through the marine deposit, as does the Newington-Dover deposit. This
discontinuity feature is being investigated as part of an other research project at UNH.

**NHDOT Geotechnical Test Embankment**
**Newington-Dover, NH**

Figure 6-11: Shifted Undrained Shear Strength Found in Segment 1
6.5 - Consolidation Properties

A summary of the consolidation testing data can be found in Appendix A. The overconsolidation ratio, compression index, recompression index and coefficient of consolidation will be compared with the work performed by Ladd (1972) and Findlay (1991).

6.5.1 - Overconsolidation Ratio (OCR)

Both the dilatometer and piezocone estimate the overconsolidation ratio and give a comparison to the laboratory findings. Equation [6-12] was used to calculate the OCR for the DMT data (Marchetti et al., 2001). Equation [6-13] was used to calculate the OCR for the CPTu data (Robertson et al., 2010).
\[ OCR_{DMT} = (0.5K_d)^{1.56}, \text{for } I_d < 1.2 \]  \[ [6-12] \]

\[ OCR_{CPTu} = k_{OCR}Q_{tu} \]  \[ [6-13] \]

As seen in the unit weight and undrained shear strength data, the OCR data was shifted to match the laboratory findings in Figure 6-13. Equation [6-14] and [6-15] were used to correct the OCR data for Newington-Dover.

\[ OCR_{Newington-Dover \, DMT} = OCR_{DMT} - 1.5 \]  \[ [6-14] \]

\[ OCR_{Newington-Dover \, CPTu} = OCR_{CPTu} - 0.5 \]  \[ [6-15] \]

As seen in Figure 6-13, the beginning of the profile has overconsolidated soils and around 20 ft below the ground surface the soil is closer to normally consolidated, increasing slowly. The OCR found in the top overconsolidated layer ranges between 1 and 8. Around elevation -8 ft, the OCR is 1 and increases at an approximate rate of 0.03/ft until elevation -48 ft.

The data found by Findlay’s laboratory testing is summarized in Table 6-2 and plotted with the Newington-Dover data in Figure 6-13. Findlay’s data ranges between a depth of 6.1 ft and 26.5 ft. Within the top desiccated layer, Findlay found an OCR range between 0.8 and 8.5. Within the normally consolidated layer, Findlay found an OCR range between 0.25 and 2.7. The OCR found in Newington-Dover appears to be very similar to what was found in PAFB, however most values are shown as greater than unity.
Figure 6-13: Shifted Overconsolidation Ratio (OCR) Found in Segment 1 and by Findlay (1991)
Table 6-2: Summary of Marine Deposit Consolidation Properties in Portsmouth, NH (Findlay, 1991)

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>OCR</th>
<th>OCR Compression</th>
<th>Recompression</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>low</td>
<td>high</td>
<td>(c_n) (ft²/day)</td>
</tr>
<tr>
<td>15.2</td>
<td>1.3</td>
<td>2.59</td>
<td>-</td>
</tr>
<tr>
<td>13.4</td>
<td>1.02</td>
<td>1.46</td>
<td>0.29</td>
</tr>
<tr>
<td>13.8</td>
<td>1.28</td>
<td>1.56</td>
<td>-</td>
</tr>
<tr>
<td>13</td>
<td>3.02</td>
<td>4.52</td>
<td>-</td>
</tr>
<tr>
<td>12.5</td>
<td>0.78</td>
<td>0.94</td>
<td>0.17</td>
</tr>
<tr>
<td>8</td>
<td>6.37</td>
<td>8.58</td>
<td>-</td>
</tr>
<tr>
<td>8.5</td>
<td>4.65</td>
<td>6.24</td>
<td>1.09</td>
</tr>
<tr>
<td>10.7</td>
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<td>11</td>
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<td>3.92</td>
<td>-</td>
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<tr>
<td>26.5</td>
<td>2.34</td>
<td>2.73</td>
<td>-</td>
</tr>
<tr>
<td>26</td>
<td>0.24</td>
<td>0.4</td>
<td>0.79</td>
</tr>
<tr>
<td>21.5</td>
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<td>0.93</td>
<td>-</td>
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<td>1.79</td>
<td>-</td>
</tr>
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<td>1.35</td>
<td>2.25</td>
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<tr>
<td>23.6</td>
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<td>0.43</td>
<td>-</td>
</tr>
<tr>
<td>24.4</td>
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<td>1.01</td>
<td>0.11</td>
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<td>4.17</td>
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<td>16.4</td>
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<td>0.25</td>
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<td>12.5</td>
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<td>2.67</td>
<td>-</td>
</tr>
<tr>
<td>14.8</td>
<td>1.14</td>
<td>1.77</td>
<td>-</td>
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<td>2.14</td>
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</tr>
<tr>
<td>10.9</td>
<td>2.7</td>
<td>3.89</td>
<td>-</td>
</tr>
</tbody>
</table>
6.5.2 - Compression Index ($C_c$) and Recompression Index ($C_r$)

Figure 6-14 shows the $C_c$ and $C_r$ determined in the UNH laboratory as part of this work, along with Findlay's (1991) work. Table 6-3 summarizes the compression and recompression indices determined by Findlay (1991). The $C_c$ in Newington-Dover ranges between 0.15 and 0.37 and the $C_r$ ranges between 0.03 and 0.07. Findlay (1991) found a range of 0.1 to 0.59 for the $C_c$ and 0.01 to 0.08 for the $C_r$. The Newington-Dover data fits in the same range found by Findlay (1991).
Figure 6-14: Compression and Recompression Indices Found in Segment 1 and by Findlay (1991)
Table 6-3: Summary of Marine Deposit Atterberg Limit and Consolidation Properties in Portsmouth, NH (Findlay, 1991)

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>w (%)</th>
<th>w (%)</th>
<th>PL</th>
<th>LL</th>
<th>V_T (pcf)</th>
<th>e_o</th>
<th>C_c</th>
<th>C_r</th>
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<tr>
<td>15.2</td>
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<td>110</td>
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<td>0.035</td>
</tr>
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<td>20.8</td>
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<td>113.4</td>
<td>1.13</td>
<td>0.31</td>
<td>0.023</td>
</tr>
<tr>
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<td>0.333</td>
<td>0.025</td>
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<tr>
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<tr>
<td>8</td>
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<td>22.1</td>
<td>38.1</td>
<td>113.1</td>
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<td>0.465</td>
<td>0.035</td>
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<tr>
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<td>0.04</td>
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<tr>
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<td>0.33</td>
<td>0.03</td>
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<tr>
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<td>40.4</td>
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<td>34</td>
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<td>0.05</td>
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</tr>
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<td>56.3</td>
<td>19.7</td>
<td>34.8</td>
<td>106</td>
<td>1.44</td>
<td>0.56</td>
<td>0.035</td>
</tr>
<tr>
<td>17.8</td>
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<td>24.7</td>
<td>39.4</td>
<td>109.4</td>
<td>1.22</td>
<td>0.36</td>
<td>0.02</td>
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<tr>
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<td>44.8</td>
<td>47.9</td>
<td>24.7</td>
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<td>0.03</td>
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<td>24.3</td>
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<td>0.025</td>
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<td>113.2</td>
<td>1.13</td>
<td>0.31</td>
<td>0.03</td>
</tr>
<tr>
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<td>41</td>
<td>22.4</td>
<td>32.8</td>
<td>115.3</td>
<td>1.13</td>
<td>0.28</td>
<td>0.03</td>
</tr>
<tr>
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<td>21.4</td>
<td>32.4</td>
<td>117.7</td>
<td>0.89</td>
<td>0.16</td>
<td>0.025</td>
</tr>
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<td>21.4</td>
<td>32.4</td>
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<td>0.11</td>
<td>0.01</td>
</tr>
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<td>110.3</td>
<td>1.32</td>
<td>0.48</td>
<td>0.035</td>
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<td>44.9</td>
<td>21.3</td>
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<td>112.8</td>
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<td>0.02</td>
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<td>21.8</td>
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<td>114</td>
<td>1.13</td>
<td>0.37</td>
<td>0.025</td>
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<td>21.8</td>
<td>36.5</td>
<td>112.7</td>
<td>1.17</td>
<td>0.4</td>
<td>0.025</td>
</tr>
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<td>42.3</td>
<td>17.1</td>
<td>20.7</td>
<td>112.8</td>
<td>1.17</td>
<td>0.33</td>
<td>0.02</td>
</tr>
<tr>
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<td>47.8</td>
<td>17.1</td>
<td>20.7</td>
<td>112</td>
<td>1.22</td>
<td>0.33</td>
<td>0.03</td>
</tr>
<tr>
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<td>37.6</td>
<td>20.9</td>
<td>31.5</td>
<td>115.8</td>
<td>1.04</td>
<td>0.36</td>
<td>0.02</td>
</tr>
<tr>
<td>10.9</td>
<td>37.2</td>
<td>43</td>
<td>20.9</td>
<td>31.5</td>
<td>115</td>
<td>1.04</td>
<td>0.37</td>
<td>0.025</td>
</tr>
</tbody>
</table>
6.5.3 - Initial Void Ratio ($e_0$)

The initial void ratio calculated prior to the laboratory testing and by Findlay (1991) are plotted in Figure 6-15. The void ratio determined by Findlay ranges between 0.78 and 1.5 for depths 6.1 to 26.5 ft. For the same depths, the laboratory data ranges between 0.87 and 1.31, fitting well with Findlay's (1991) data.
Figure 6-15: Initial Void Ratio Found in Segment 1 and by Findlay (1991)
6.5.4 - **Coefficient of Consolidation** ($c_v$)

The laboratory consolidation tests were performed to determine the vertical coefficient of consolidation for the virgin compression and recompression curves. The dilatometer dissipation tests were performed to determine the horizontal coefficient of consolidation and are summarized in Table 6-4 and Table 6-5. The laboratory and dilatometer results are plotted in Figure 6-16.

Table 6-4: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) Dissipation Testing
Horizontal Coefficient of Consolidation

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>$c_{h,OC}$ (ft$^2$/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>1.37</td>
</tr>
<tr>
<td>20</td>
<td>1.39</td>
</tr>
<tr>
<td>23.5</td>
<td>0.20</td>
</tr>
<tr>
<td>27</td>
<td>0.11</td>
</tr>
<tr>
<td>30.5</td>
<td>0.08</td>
</tr>
<tr>
<td>34</td>
<td>0.07</td>
</tr>
<tr>
<td>37.5</td>
<td>0.07</td>
</tr>
<tr>
<td>42.5</td>
<td>0.08</td>
</tr>
<tr>
<td>47.5</td>
<td>0.07</td>
</tr>
<tr>
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<td>0.08</td>
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<tr>
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<td>5.02</td>
</tr>
<tr>
<td>67.5</td>
<td>13.01</td>
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</tbody>
</table>

Table 6-5: Segment 1 Q-B218 (Sta. 603 + 35, LT 25) Dissipation Testing
Horizontal Coefficient of Consolidation

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>$c_{h,OC}$ (ft$^2$/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>1.02</td>
</tr>
<tr>
<td>22</td>
<td>0.30</td>
</tr>
<tr>
<td>32</td>
<td>0.07</td>
</tr>
<tr>
<td>42</td>
<td>0.10</td>
</tr>
</tbody>
</table>
Figure 6-16: Horizontal and Vertical Coefficient of Consolidation Found in Segment 1 and by Findlay (1991)
The horizontal coefficient of consolidation appears to be decreasing between approximately 5 and 30 ft, and then remains constant until around 54 ft where it begins increasing. The vertical coefficient of consolidation appears to follow the same general trend decreasing between 5 and 25 ft, remaining constant until 52 ft and increasing until the bottom of the marine deposit.

Ladd (1972), Findlay (1991) and NeJame (1991) also found the coefficient of consolidation via laboratory and dilatometer testing, which have been summarized in Table 6-6. Ladd's (1972) work is graphically displayed in Figure 6-17. Finday's (1991) work is graphically displayed with the Newington-Dover data in Figure 6-16.

Table 6-6: Summary of Marine Deposit Coefficients of Consolidation in Portsmouth, NH

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>$c_v$ (ft$^2$/day)</th>
<th>$c_h$ (ft$^2$/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ladd et al. (1972)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oedometer - Virgin</td>
<td>0.1 ± 0.05</td>
<td>-</td>
</tr>
<tr>
<td>Compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oedometer - Recompression</td>
<td>1</td>
<td>-</td>
</tr>
<tr>
<td>Findlay (1991)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oedometer - Virgin</td>
<td>7.7 ± 7.6</td>
<td>1.1 ± 1</td>
</tr>
<tr>
<td>Compression</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Oedometer - Recompression</td>
<td>11 ± 10.6</td>
<td>4 ± 3.4</td>
</tr>
<tr>
<td>NeJame (1991)</td>
<td></td>
<td></td>
</tr>
<tr>
<td>DMT</td>
<td>-</td>
<td>1.5 ± 1.3</td>
</tr>
</tbody>
</table>
The $c_v$ corresponding to the virgin compression curve for Newington-Dover in the normally consolidated zone ranges between 0.10 and 0.39 ft$^2$/day. The in situ horizontal coefficient of consolidation ($c_h$) for Newington-Dover in the normally consolidated zone ranges between 0.07 and 5.02 ft$^2$/day. The $c_v$ in Newington-Dover seems to be slightly greater than Ladd's (1972) findings (0.1 ± 0.05). Findlay (1991) found a large range (7.7 ± 7.6) for the $c_v$; the Newington-Dover data falls in the lower end. The Newington-Dover $c_h$ appears to fall slightly above the range (1.5 ± 1.3) determined by NeJame (1991).
6.6 - Coefficient of Lateral Earth Pressure ($K_0$)

Both the DMT and CPTu provide estimates of the coefficient of lateral earth pressure. Findlay's (1991) work at PAFB focused on the use of a self-boring pressuremeter (SBPM), which is the most widely accepted method to measure the in situ lateral stress. Figure 6-19 presents his findings, along with an estimated profile by the Brooker and Ireland (1965) method and a 4th degree polynomial data fit. The following equations represent the Brooker and Ireland (1965) method.

$$K_0 = 0.95 - \sin \phi' \text{ for NC clay} \quad [6-16]$$

$$K_0(OC) = K_0(NC) \sqrt{OCR} \text{ for OC clay} \quad [6-17]$$

Findlay (1991) concluded the lowest (undisturbed) SBPM $K_0$ values are most likely representative of the in situ $K_0$. At PAFB, between the depths of 6.5 and 12 ft, $K_0$ ranged between 7.5 and 1, whereas between 12 and 23 ft it ranged between 0.5 and 1.

Figure 6-20 presents the DMT empirical $K_0$ values found by NeJame (1991) plotted with the SBPM data. The DMT values appear to match well with the minimum SBPM values in the upper clay layer, and maximum SBPM values in the lower clay layer.

Benoît and Lutenegger (1993) also analyzed DMT and SBPM data at PAFB. They found the DMT $P_0-u_0$ values followed the trend of the SBPM test results, but
with a large magnitude difference as seen in Figure 6-18. The $P_0-P_2$ values followed the trend and magnitude of the SBPM values.

![Graph showing horizontal effective stress profile at PAFB](image)

**Figure 6-18: Horizontal Effective Stress Profile at PAFB (Benoit and Lutenegger, 1993)**

Figure 6-21 shows the DMT estimates for $K_0$ found in Newington-Dover using $P_0-u_0$ and $P_0-P_2$ values. The $P_0-u_0$ results fluctuate between 0.75 and 1 for $K_0$ in the
lower clay layer. The $P_0-P_2$ results appear to be much lower, less than 0.2, in the lower clay layer. In the case of Newington-Dover, the $P_0-u_0$ results appear to better match the lower clay layer results found using the SBPM. The upper clay layer in both cases does not appear to exceed 2.2.

Figure 6-19: Pease Air Force Base SBPM Profile of $K_0$ (Findlay, 1991)
Figure 6-20: DMT Empirical $K_0$ Values Compared to Maximum and Minimum SBPM Results (NeJame, 1991)
Figure 6-21: Coefficient of Lateral Earth Pressure Found by the DMT in Segment 1
6.7 - Permeability \((k)\)

Lastly, the DMT, CPTu and consolidation testing can estimate the permeability. Schmertmann (1988) proposed estimating permeability based on the coefficient of consolidation found during the DMT dissipation tests. The horizontal drained constrained modulus \((M_h)\) can be calculated using Equation [6-18], then the permeability could be calculated using Equation [6-19]. The permeability can be estimated using the Soil Behavior Type for the CPTu and Table E-4 in Appendix E. The permeability based on consolidation testing can be estimated using Terzaghi's consolidation theory in Equation [6-20]. Figure 6-22 plots the permeability with depth for each method in English units. Figure 6-23 plots the permeability with depth for each method in metric units.

\[
M_h = K_0 M_{DMT} \quad \text{[6-18]}
\]

\[
k = c_h \gamma_w / M_h \quad \text{[6-19]}
\]

\[
k = \frac{c_v a_v \gamma_w}{1 + e} \quad \text{[6-20]}
\]

where \(a_v\) = coefficient of compressibility

The CPTu data appears to be the largest and is based on the Soil Behavior Type. The DMT and consolidation data are directly related to the coefficient of consolidation which is most likely more reliable. Unfortunately, there is no data from previous research to determine the reliability of the data. Due to the direct relationship between the consolidation data and Terzaghi's consolidation theory,
this data will be used through the remainder of the thesis. The permeability appears to be slightly higher in the upper clay layer and decrease with depth.

Figure 6-22: Permeability Found by the CPTu, DMT and Consolidation Testing in Segment 1 (English)
Figure 6-23: Permeability Found by the CPTu, DMT and Consolidation Testing in Segment 1 (metric)
Table 6-7, 6-8 and 6-9 summarize the results determined by Ladd (1972), Findlay (1991), the CIE 961 class (1997) and Newington-Dover for each layer of the marine deposit; a stiff upper zone, followed by a very soft normally consolidated clay and ending with a sandy lower layer. Below those three layers is till and then bedrock. Both Ladd (1972) and Findlay (1991) performed their studies in Portsmouth where the marine deposit is thinner than that found in Newington-Dover. NeJame (1991) and Murray (1995) tested at the same location as Findlay (1991) and found similar results. The CIE 961 class (1997) found a thicker layer of the marine deposit in Dover, NH over the Piscataqua River.

After comparing previous work to the work performed at the Newington-Dover site, it can be confirmed that the clay within the Seacoast area follows the same general trend in terms of geotechnical index, compressibility and strength properties. The Newington-Dover Segment 1 data shows the marine clay beginning around 10 ft below the ground surface and ending around 65 ft, for an average thickness of 55 ft. Within the clay layer, there appears to be 3 separate zones: a stiff top layer, followed by a very soft middle layer and ending with a sandy bottom layer.
Table 6-7: Summary of Layer 1 Engineering Properties Based on Ladd (1972), Findlay (1991) and CIE 961 (1997)

<table>
<thead>
<tr>
<th>Depth Below Surface (ft)</th>
<th>$\gamma_t$ (pcf)</th>
<th>$e_o$</th>
<th>LL (psf)</th>
<th>PL</th>
<th>$s_u$ (ft&lt;sup&gt;2&lt;/sup&gt;/day)</th>
<th>$c_v$ (ft&lt;sup&gt;2&lt;/sup&gt;/day)</th>
<th>OCR</th>
<th>$C_c$</th>
<th>$C_r$</th>
<th>$K_0$</th>
<th>$k$ (ft/day) (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ladd (1972)</td>
<td>0 - 8</td>
<td>109</td>
<td>48 ± 3</td>
<td>24 ± 2</td>
<td>1015 ± 615</td>
<td>0.1 ± 0.05</td>
<td>-</td>
<td>2.83</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Findlay (1991)</td>
<td>6</td>
<td>120 ± 2</td>
<td>0.84 ± 0.05</td>
<td>32.4</td>
<td>21.4</td>
<td>2000</td>
<td>15.27</td>
<td>2.14</td>
<td>7 ± 1.5</td>
<td>0.14 ± 0.02</td>
<td>0.018 ± 0.008</td>
</tr>
<tr>
<td>CIE 961 (1997)</td>
<td>6-8</td>
<td>100 ± 6</td>
<td>54.02 ± 2.48</td>
<td>29.51</td>
<td>637 ± 10</td>
<td>-</td>
<td>3.6 ± 0.9</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Newington - Dover</td>
<td>10-20</td>
<td>118</td>
<td>1.1</td>
<td>35</td>
<td>22</td>
<td>400</td>
<td>0.21</td>
<td>1.4</td>
<td>4</td>
<td>0.28</td>
<td>0.05</td>
</tr>
</tbody>
</table>
Table 6-8: Summary of Layer 2 Engineering Properties Based on Ladd (1972), Findlay (1991) and CIE 961 (1997)

<table>
<thead>
<tr>
<th>Depth Below Surface (ft)</th>
<th>Yt (pcf)</th>
<th>e₀</th>
<th>LL</th>
<th>PL</th>
<th>sₓ (psf)</th>
<th>cₓ (ft²/day)</th>
<th>cʰ (ft²/day)</th>
<th>OCR</th>
<th>Cc</th>
<th>C'r</th>
<th>k (ft/day) (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ladd (1972)</td>
<td>8-20</td>
<td>109</td>
<td>-</td>
<td>39 ± 4</td>
<td>20 ± 2</td>
<td>325 ± 75</td>
<td>0.1 ± 0.05</td>
<td>-</td>
<td>1.49</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Findlay (1991)</td>
<td>6-26</td>
<td>114 ± 8</td>
<td>1.14 ± 0.36</td>
<td>30.1 ± 9.4</td>
<td>20.9 ± 3.8</td>
<td>598 ± 333</td>
<td>1.7 ± 1.5</td>
<td>0.6 ± 0.49</td>
<td>4.42 ± 0.16</td>
<td>0.38 ± 0.02</td>
<td>0.75 ± 0.25</td>
</tr>
<tr>
<td>CIE 961 (1997)</td>
<td>8-36</td>
<td>100 ± 6</td>
<td>-</td>
<td>54.02 ± 2.48</td>
<td>29.51 ± 0.08</td>
<td>730 ± 125</td>
<td>-</td>
<td>-</td>
<td>1.9 ± 1.1</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
| Newington-Dover          | 20-60    | 110 | 1.2 | 38 | 24 | 450 | 0.12 | 0.1 | 1.5 | 0.31 | 0.05 | 1.00 2 x 10⁻⁴  
                                             |          |     |     |    |    |    |     |     |     |     |     | (7.1 x 10⁻⁶) |
Table 6-9: Summary of Layer 3 Engineering Properties Based on Ladd (1972), Findlay (1991) and CIE 961 (1997)

<table>
<thead>
<tr>
<th>Depth Below Surface (ft)</th>
<th>$y_t$ (pcf)</th>
<th>$e_o$</th>
<th>LL</th>
<th>PL</th>
<th>$s_u$ (psf)</th>
<th>$c_v$ (ft&lt;sup&gt;2&lt;/sup&gt;/day)</th>
<th>$c_h$ (ft&lt;sup&gt;2&lt;/sup&gt;/day)</th>
<th>OCR</th>
<th>$C_r$</th>
<th>$K_r$</th>
<th>$k$ (ft/day) (cm/s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ladd (1972)</td>
<td>20-34</td>
<td>109</td>
<td>-</td>
<td>32 ± 7</td>
<td>20 ± 5</td>
<td>253 ± 73</td>
<td>0.15 ± 0.05</td>
<td>-</td>
<td>1.26</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Findlay (1991)</td>
<td>26</td>
<td>123 ± 3</td>
<td>0.79 ± 0.07</td>
<td>25.5</td>
<td>20.6</td>
<td>340</td>
<td>2.49</td>
<td>0.79</td>
<td>1.49 ± 1.25</td>
<td>0.14 ± 0.04</td>
<td>0.05 ± 0.04</td>
</tr>
<tr>
<td>CIE 961 (1997)</td>
<td>36-38</td>
<td>100 ± 6</td>
<td>-</td>
<td>54.02 ± 2.48</td>
<td>29.51 ± 0.08</td>
<td>595 ± 157</td>
<td>-</td>
<td>0.8 ± 0.2</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Newington-Dover</td>
<td>60-65</td>
<td>120</td>
<td>0.8</td>
<td>28</td>
<td>20</td>
<td>900</td>
<td>0.32</td>
<td>9</td>
<td>0.19</td>
<td>0.04</td>
<td>0.73</td>
</tr>
</tbody>
</table>
Chapter 7 - SETTLEMENT PREDICTIONS

7.1 - Introduction

Prior to testing by the geotechnical group at UNH, the NHDOT had predicted a total settlement of up to 2.5 ft (0.76 m) based on the work previously performed by Ladd (1972) in Portsmouth, NH.

Using the in situ and laboratory data collected by UNH prior to construction (Phase 1), settlement calculations were performed via a variety of methods including hand calculations based on DMT results, finite element analyses using PLAXIS and the software package titled Settle 3D. Each method assumes the dimensions and drain spacing of Segment 1 due to the majority of testing performed in that location of the embankment.

7.2 - DMT Settlement Prediction

Assuming the dimensions illustrated in Figure 7-1, the settlement under the center of the embankment was calculated based on the DMT results at borehole Q-B213 and the Boussinesq Method. This profile was considered representative of Segment 1 based on the repeatability of the test.
The NHDOT recommended a unit weight of 120 pcf be used to represent the embankment fill. The Boussinesq Method for embankments was used to calculate the stress distributions below the center of the embankment assuming the dimensions above and unit weight of the fill. An explanation of the Boussinesq Method and how it was used can be found in Appendix G. Figure 7-2 shows the variation in stress distribution with depth.

The most significant stiffness parameter for settlement analyses obtained from the DMT is the constrained modulus \( (M_{D_M T}) \). The constrained modulus is used in combination with the stress distribution calculated by the Boussinesq Method to predict the total settlement as seen in equation [7-1] (Marchetti et al., 2001).

\[
S_{D_M T} = \sum \frac{\Delta \sigma_z}{M_{D_M T}} \Delta z
\]  

[7-1]
As of May 2013, the embankment had experienced 0.95 ft (0.29 m) of settlement.
The hand calculations show a total settlement of 22.3 in. or 1.8 ft (0.5 m),
meaning the current field settlement is only at 50% consolidation.

**Figure 7-2: Stress Distribution below the Center of Embankment According to the Boussinesq Method**
7.3 - Finite Element Analysis

7.3.1 - Introduction

The finite element analysis (FEA) for the Newington-Dover project was performed using the PLAXIS 2D software. PLAXIS presents several different models in simulating the mechanical behavior of soils. Traditionally, the linear elastic model based on Hooke’s law of elasticity is considered the simplest, however, least accurate (PLAXIS manual, 2011). Three separate models were considered in modeling the marine deposit, however, the Soft Soil (SS) model was considered most applicable to the laboratory and in situ data collected during Phase 1.

Eight separate cases were analyzed and are summarized in Table 7-1. The different geometric scenarios will be discussed in section 7.3.2.

Table 7-1: Summary of FEA Cases

<table>
<thead>
<tr>
<th>Drains</th>
<th>Geometry</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>Yes</td>
</tr>
<tr>
<td>Case 1</td>
<td>No</td>
</tr>
<tr>
<td>Case 2</td>
<td>Yes</td>
</tr>
<tr>
<td>Case 2</td>
<td>No</td>
</tr>
<tr>
<td>Case 3</td>
<td>Yes</td>
</tr>
<tr>
<td>Case 4</td>
<td>Yes</td>
</tr>
<tr>
<td>Case 5</td>
<td>Yes</td>
</tr>
<tr>
<td>Case 6</td>
<td>Yes</td>
</tr>
</tbody>
</table>
7.3.2 - **PLAXIS Model**

As mentioned previously, the FEA was modeled after Segment 1 of the test embankment. A plane strain model was selected as the model type. In a plane strain analysis, the geometry in the z direction is assumed to extend a long distance thus producing zero strain in the z direction. Unfortunately, there are limitations to the plane strain analysis. For example, any drain drawn on the model will be assumed present throughout the entire cross section resulting in very long, large area drains. Figure 7-3 represents the actual model that PLAXIS would calculate for based on the colored cross-section. For this research, PV drains were installed in various triangular patterns throughout the cross section of the embankment. The total area of drains modeled in PLAXIS would be greater than that experienced in the field. Therefore, the cases modeled with drains will underestimate the time it would take to reach 90% consolidation and should be acknowledged.

![Figure 7-3: Assumptions Made During Plane Strain Analysis](image)
A 15-node triangular element was selected for the mesh providing a fourth order analysis of displacement. The mesh coarseness was selected as very fine as seen in Figure 7-4. As coarseness decreases, the accuracy and time to calculate both increase. Standard fixities were assigned to the model. Vertical and horizontal fixities were drawn along the bottom plane preventing any displacements, simulating the bedrock below the glacial outwash layer. The standard fixities also assigned horizontal fixities to the vertical planes on the side of the model, as it is assumed that the conditions on either side are the same and therefore, no horizontal movement outside the model should occur.

Two different geometries were modeled based on the variation in depths found within the boring logs. The top orange trapezoidal layer represents the embankment which was drawn as 12 ft (3.7 m) tall and 120 ft (36.6 m) wide with a 2:1 slope on the sides. Beneath the embankment was the sand drainage blanket which was 1 ft (0.3 m) thick. Table 7-2 summarizes the subsurface conditions modeled below the blanket for two separate depth scenarios.

Predetermined (default) values from PLAXIS were used for the embankment (orange layer), sand drainage blanket (salmon layer), alluvium (blue layer) and glacial outwash (purple layer) layers. The glacial till was assumed to be undrained due to it's density and composition, and therefore, part of the bedrock (standard fixities).
Figure 7-4: Finite Element Model Mesh in PLAXIS 2D
Table 7-2: PLAXIS Soil Layer Thickness

<table>
<thead>
<tr>
<th></th>
<th>Alluvium</th>
<th>Marine Deposit Layer 1</th>
<th>Marine Deposit Layer 2</th>
<th>Marine Deposit Layer 3</th>
<th>Glacial Outwash</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario 1</td>
<td>10 ft</td>
<td>10 ft</td>
<td>40 ft</td>
<td>5 ft</td>
<td>10 ft</td>
</tr>
<tr>
<td>Scenario 2</td>
<td>10 ft</td>
<td>10 ft</td>
<td>40 ft</td>
<td>5 ft</td>
<td>5 ft</td>
</tr>
</tbody>
</table>

The marine deposit was divided into 3 separate layers, each undrained. As mentioned previously, the SS model was selected to represent each layer of the marine deposit. For this model, PLAXIS requires the friction angle to be greater than zero; therefore, $5^\circ$ was used to fulfill this requirement to avoid calculation errors, while remaining relatively small such that only minimal additional strength was added to the soil. Clays typically modeled by the soft soil model have a dilatancy angle of zero. Table 7-3 summarizes the geotechnical model parameters of the soil layers modeled in PLAXIS for Case 1 and 2. Layer 2 differed for Case's 3, 4, 5 and 6 and is summarized in Table 7-4.
## Table 7-3: PLAXIS Geotechnical Model Parameters for Case 1 and 2

<table>
<thead>
<tr>
<th>Soil Model</th>
<th>Thickness (ft)</th>
<th>$Y_{sat}$ (pcf)</th>
<th>$Y_{unsat}$ (pcf)</th>
<th>$e_o$</th>
<th>$s_u$ (psf)</th>
<th>OC</th>
<th>$C_c$</th>
<th>$C_r$</th>
<th>$K_o$</th>
<th>$k$ (ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Embankment Hardening Soil Drained</td>
<td>12</td>
<td>120</td>
<td>120</td>
<td>0.5</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>0.5</td>
<td>0.3957</td>
</tr>
<tr>
<td>Sand Blanket Hardening Soil Drained</td>
<td>1</td>
<td>127.3</td>
<td>127.3</td>
<td>0.5</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>0.47</td>
<td>3.281</td>
</tr>
<tr>
<td>Alluvium Linear Elastic Drained</td>
<td>10</td>
<td>127.3</td>
<td>127.3</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1</td>
<td>0.8135</td>
</tr>
<tr>
<td>Marine Deposit Layer 1 Soft Soil Undrained</td>
<td>10</td>
<td>118</td>
<td>118</td>
<td>1.1</td>
<td>400</td>
<td>4</td>
<td>0.28</td>
<td>0.05</td>
<td>1.22</td>
<td>6x10⁻⁴</td>
</tr>
<tr>
<td>Marine Deposit Layer 2 Soft Soil Undrained</td>
<td>40</td>
<td>110</td>
<td>110</td>
<td>1.2</td>
<td>450</td>
<td>1.5</td>
<td>0.31</td>
<td>0.05</td>
<td>1.00</td>
<td>2x10⁻⁴</td>
</tr>
<tr>
<td>Marine Deposit Layer 3 Soft Soil Undrained</td>
<td>5</td>
<td>120</td>
<td>120</td>
<td>0.8</td>
<td>900</td>
<td>1</td>
<td>0.19</td>
<td>0.04</td>
<td>0.73</td>
<td>7x10⁻⁵</td>
</tr>
<tr>
<td>Glacial Outwash Hardening Soil Drained</td>
<td>5 or 10</td>
<td>127.3</td>
<td>108.2</td>
<td>0.5</td>
<td>-</td>
<td>1</td>
<td>-</td>
<td>-</td>
<td>0.44</td>
<td>1.968</td>
</tr>
</tbody>
</table>
Table 7-4: PLAXIS Geotechnical Model Parameters for Layer 2 in Case 3, 4, 5 and 6

<table>
<thead>
<tr>
<th>Model</th>
<th>$Y_{sat}$ (pcf)</th>
<th>$Y_{unsat}$ (pcf)</th>
<th>$e_o$</th>
<th>$S_u$ (psf)</th>
<th>OCR</th>
<th>$C_c$</th>
<th>$C_r$</th>
<th>$K_o$</th>
<th>$k$ (ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 3</td>
<td>110</td>
<td>110</td>
<td>1.2</td>
<td>450</td>
<td>1.5</td>
<td>0.31</td>
<td>0.05</td>
<td>1.0</td>
<td>$2 \times 10^3$</td>
</tr>
<tr>
<td>Case 4</td>
<td>110</td>
<td>110</td>
<td>1.2</td>
<td>450</td>
<td>1.5</td>
<td>0.31</td>
<td>0.05</td>
<td>1.0</td>
<td>$2 \times 10^2$</td>
</tr>
<tr>
<td>Case 5</td>
<td>110</td>
<td>110</td>
<td>1.2</td>
<td>450</td>
<td>1.5</td>
<td>0.31</td>
<td>0.95</td>
<td>0.7</td>
<td>$2 \times 10^4$</td>
</tr>
<tr>
<td>Case 6</td>
<td>110</td>
<td>110</td>
<td>1.2</td>
<td>450</td>
<td>1.5</td>
<td>0.31</td>
<td>0.05</td>
<td>1.3</td>
<td>$2 \times 10^4$</td>
</tr>
</tbody>
</table>
Case 1 and 2 were run both with and without drains. The drains were spaced apart at 6 ft (1.8 m) beginning at the top of the sand drainage blanket through to the bottom of the marine deposit layer 3. The constructions phases for with and without drains are summarized in Table 7-5. The water table was placed 3 ft (0.9 m) below ground surface for all models.

Table 7-5: PLAXIS Construction Phases

<table>
<thead>
<tr>
<th>Model w/ Drains</th>
<th>Model w/o Drains</th>
</tr>
</thead>
<tbody>
<tr>
<td>PLAXIS Calculation Type and Loading Input</td>
<td>PLAXIS Calculation Type and Loading Input</td>
</tr>
<tr>
<td>Initial Phase</td>
<td>Initial Phase</td>
</tr>
<tr>
<td>-Gravity Loading</td>
<td>-Gravity Loading</td>
</tr>
<tr>
<td>-Staged Construction</td>
<td>-Staged Construction</td>
</tr>
<tr>
<td>Placement of Sand Drainage Blanket</td>
<td>Placement of Sand Drainage Blanket</td>
</tr>
<tr>
<td>-Consolidation (EPP)</td>
<td>-Consolidation (EPP)</td>
</tr>
<tr>
<td>-Staged Construction 1 day</td>
<td>-Staged Construction 1 day</td>
</tr>
<tr>
<td>Placement of PV Drains</td>
<td>-</td>
</tr>
<tr>
<td>-Consolidation (EPP)</td>
<td>-</td>
</tr>
<tr>
<td>-Staged Construction 7 days</td>
<td>-</td>
</tr>
<tr>
<td>Embankment Construction</td>
<td>Embankment Construction</td>
</tr>
<tr>
<td>-Consolidation (EPP)</td>
<td>-Consolidation (EPP)</td>
</tr>
<tr>
<td>-Staged Construction 60 days</td>
<td>-Staged Construction 60 days</td>
</tr>
<tr>
<td>90% Consolidation</td>
<td>90% Consolidation</td>
</tr>
<tr>
<td>-Consolidation (EPP)</td>
<td>-Consolidation (EPP)</td>
</tr>
<tr>
<td>-Staged Construction</td>
<td>-Staged Construction</td>
</tr>
<tr>
<td>Additional Waiting Time</td>
<td>Additional Waiting Time</td>
</tr>
<tr>
<td>-Consolidation (EPP)</td>
<td>-Consolidation (EPP)</td>
</tr>
<tr>
<td>-Staged Construction 100000 days</td>
<td>-Staged Construction 100000 days</td>
</tr>
</tbody>
</table>

where EPP: Excess Pore Pressures
Table 7-6 summarizes the results found in each case.

Table 7-6: PLAXIS Settlement Results

<table>
<thead>
<tr>
<th></th>
<th>Maximum Movement (ft)</th>
<th>Maximum Vertical Settlement (ft)</th>
<th>Time After Construction to Reach 90% Consolidation (yr)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case 1</td>
<td>w/o Drains</td>
<td>0.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Case 1</td>
<td>w/ Drains</td>
<td>0.4</td>
<td>2.6</td>
</tr>
<tr>
<td>Case 2</td>
<td>w/o Drains</td>
<td>0.5</td>
<td>2.5</td>
</tr>
<tr>
<td>Case 2</td>
<td>w/ Drains</td>
<td>0.4</td>
<td>2.6</td>
</tr>
<tr>
<td>Case 3</td>
<td>w/ Drains</td>
<td>0.4</td>
<td>2.6</td>
</tr>
<tr>
<td>Case 4</td>
<td>w/ Drains</td>
<td>0.3</td>
<td>2.5</td>
</tr>
<tr>
<td>Case 5</td>
<td>w/ Drains</td>
<td>0.4</td>
<td>2.6</td>
</tr>
<tr>
<td>Case 6</td>
<td>w/ Drains</td>
<td>0.4</td>
<td>2.6</td>
</tr>
</tbody>
</table>

The difference between Case 1 and Case 2 is the thickness of the glacial outwash layer below the clay. It does not appear to have much effect on the settlement results, and therefore, the thicker layer (Case 1, 10 ft) was used in the remainder of the models. The cases with the PV drains settle more quickly than those cases without the PV drains as expected. For example, in Case 1, the model with drains took about 7 years to reach 90% consolidation, whereas the model without drains took over 14 years to obtain 90% consolidation. The remaining cases were solely run with drains and geometry 1.

Figure 7-5 shows the vertical settlement below the center of the embankment for each case with respect to depth. Case 3 and 4 were the same as Case 1, except
the permeability in Layer 2 was greater. It appears that as the permeability increases, the settlement rate increases. Case 5 was the same as Case 1, except the $K_o$ value was decreased in Layer 2 to 0.7. Case 6 was the same as Case 1, except the $K_o$ value was increased in Layer 2 to 1.3. By increasing and decreasing the $K_o$ value, the settlement did not appear to change significantly, however, the time to reach 90% consolidation was slightly greater.

Figure 7-6 shows the horizontal movement below the center of the embankment. It should be noted that the vertical scale is very small and each case experienced minimal to zero movement in the horizontal direction. The negative sign denotes movement to the left and the positive sign denotes movement to the right. Each case with drains appears to level off sooner than those cases without drains. The cases without drains also appear to experience larger movements than those with drains. Case 1 without drains does not appear to level off like Case 2.

In addition to the lateral movement below the center of the embankment, the lateral movement below the crest and toe were graphed. Figure 7-7 shows the lateral movement below the crest and Figure 7-8 shows the lateral movement below the toe. In both figures, the cases without drains experience movement to the left while the cases with drains experience movement to the right. The movement experienced below the toe is greater than that experienced below the crest which is greater than that experienced below the center of the embankment. For example, in Case 1 with drains, the lateral movement below the center of the embankment is about $7.5 \times 10^{-5}$ in., below the crest of the
embankment is about 0.003 in. and below the toe of the embankment is about 0.015 in. at 10,000 days.

Similar figures to Figure 7-9 and Figure 7-10 were obtained for each of the PLAXIS models. Figure 7-9 shows the maximum vertical settlement directly below the center of the embankment. The settlement appears to decrease with depth as expected. Figure 7-10 shows the maximum lateral displacement located beneath the ends of the embankment. The least amount of lateral displacement is found beneath the center of the embankment as shown previously in Figure 7-6.

Additionally, the excess pore pressures were compared between Model 1 with PV drains and Model 1 without PV drains to confirm the drains were working. Figure 7-11 represents the model without the PV drains, the excess pore pressures remain below the embankment even after settlement has reached 90% consolidation. Figure 7-12 represents the model with PV drains, where the excess pore pressures have moved away from the embankment, confirming the drains are working properly. The excess pore pressures for Model 1 without the drains are also larger (149 psf) than those found with the drains (78 psf).
Figure 7-5: PLAXIS Settlement Predictions below the Center of the Embankment
Figure 7-6: PLAXIS Lateral Movement below the Center of the Embankment
Figure 7-7: PLAXIS Lateral Movement below the Crest of the Embankment
Figure 7-8: PLAXIS Lateral Movement below the Toe of the Embankment
Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 2)
Minimum value = -2.612 ft (Element 580 at Node 4540)

Figure 7-9: PLAXIS Model 1 with PV Drains Vertical Displacement
Total displacements $u_x$

Maximum value = 0.4150 ft (Element 980 at Node 6048)
Minimum value = -0.4150 ft (Element 507 at Node 3028)

Figure 7-10: PLAXIS Model 1 with PV Drains Lateral Movement
Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)

Maximum value = 0.000 lbf/ft$^2$ (Element 1 at Node 2)
Minimum value = -149.0 lbf/ft$^2$ (Element 695 at Node 4557)

Figure 7-11: PLAXIS Model 1 without PV Drains Excess Pore Pressure
Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)

Maximum value = 0.02370 lbf/ft$^2$ (Element 477 at Node 2682)

Minimum value = -77.76 lbf/ft$^2$ (Element 254 at Node 1188)

Figure 7-12: PLAXIS Model 1 with PV Drains Excess Pore Pressure
The PLAXIS settlement figures for the other models can be found in Appendix H.

Figure 7-13 shows the different PLAXIS models with the NHDOT vertical settlement data. The field curve most closely follows Model 1 and 2 without drains. It appears the models with the drains have a much steeper slope prior to the settlement leveling off. As mentioned previously, by modeling the embankment in plane strain analysis the drains are assumed to be infinitely long, not representative of their small, approximately 4 in. (10.2 cm), rectangular cross section. Therefore, it makes sense the field conditions are much closer to those modeled without drains.

Figure 7-14 shows the lateral deformation with depth and time experienced by the field inclinometer at Station 602+98, LT 64, next to the toe of the embankment. Figure 7-8 showed the lateral deformation found in PLAXIS directly below the toe at ground surface with time. It appears the movement experienced in the field at the top was towards the left, similar to Case 1 and 2 without drains. The field has currently experienced about 1.15 in. of movement. PLAXIS showed the maximum movement occurring at the beginning as well with a magnitude of about 0.09 ft (1.08 in.).

Overall, Case 1 and 2 without drains model the field conditions best.
Figure 7-13: Vertical Settlement predicted by PLAXIS Models with NHDOT data
Figure 7-14: INCL1, Inclinometer Data Lateral Deformation (Sta. 602+98, LT 64) (NHDOT June 2013 Instrumentation Report)
Lastly, a software package titled Settle 3D was used to predict the settlement as a third comparison. This software calculates settlement in the vertical dimension only while it can compute three dimensional stresses from applied surface loads. The dimensions described previously in section 7.2 were used along with the subsurface conditions set for Case 1 in the FEA. Settle 3D assumes a linear or non-linear material type for the soil which does not require the same parameters used in PLAXIS. Table 7-7 summarizes the geotechnical parameters used to model the embankment and subsurface. The generic value for a given parameter was used when unknown.

Two separate trials were performed: both with and without PV drains. The construction phases described in Table 7-5 for the FEA were used, as well as placing the water table 3 ft below the surface.

Table 7-8 summarizes the settlement predictions made by the Settle 3D software. The results show that the PV drains did not influence the total settlement below the center of the embankment, just the time to reach 90% consolidation as expected.
### Table 7-7: Settle 3D Geotechnical Model Parameters

<table>
<thead>
<tr>
<th>Material Name</th>
<th>Color</th>
<th>Unit Weight (tons/ft³)</th>
<th>Sat. Unit Weight (tons/ft³)</th>
<th>Material Type</th>
<th>Cc</th>
<th>Cr</th>
<th>OCR</th>
<th>e0</th>
<th>mv (ft²/l)</th>
<th>mvur (ft²/l)</th>
<th>K (ft/ly)</th>
<th>Kr (ft/ly)</th>
<th>B-bar</th>
<th>Ch/Cv Ratio</th>
<th>Kh/Kv Ratio</th>
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</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>□</td>
<td>0.06365</td>
<td>0.06365</td>
<td>Linear</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.01957</td>
<td>0.01957</td>
<td>295.93</td>
<td></td>
<td></td>
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<td>Marine Deposit Layer 1</td>
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<td>0.059</td>
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<td>0.05</td>
<td>4</td>
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<td>0.0365</td>
<td>0.365</td>
<td>0.365</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Marine Deposit Layer 2</td>
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<td>0.055</td>
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<td>1.5</td>
<td>1.2</td>
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<td>1.2</td>
<td></td>
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<td></td>
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<tr>
<td>Marine Deposit Layer 3</td>
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<td>0.06</td>
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<td>0.04</td>
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<td>0.0365</td>
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<td>0.3</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glacial Outwash</td>
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<td>0.064</td>
<td>Linear</td>
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<td></td>
<td></td>
<td></td>
<td>0.01957</td>
<td>0.01957</td>
<td>718.32</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Drainage Blanket</td>
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<td>0.064</td>
<td>0.064</td>
<td>Linear</td>
<td></td>
<td></td>
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<td></td>
<td>0.01957</td>
<td>0.01957</td>
<td>1197.57</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

### Table 7-8: Settle 3D Settlement Results

<table>
<thead>
<tr>
<th>Settlement Below the Center of the Embankment (ft)</th>
<th>Approximate Time to 90% Consolidation (yrs)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Case w/o PV Drains</td>
<td>2.1</td>
</tr>
<tr>
<td>Case 1 w/o PV Drains</td>
<td>2.1</td>
</tr>
<tr>
<td></td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>38</td>
</tr>
</tbody>
</table>
7.5 - Conclusions

As of May 2013, approximately 5 months (0.42 yr) after construction was completed, the NHDOT settlement platforms were reading an average total settlement in Segment 1 of approximately 0.95 ft. Table 7-9 and Figure 7-15 compare the NHDOT readings with the different vertical settlement prediction methods.

Table 7-9: Summary of Settlement Estimates

<table>
<thead>
<tr>
<th>Settlement (ft)</th>
<th>Time to Reach 90% Consolidation After Embankment Construction (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>DMT Hand Calculations 1.8</td>
<td>-</td>
</tr>
<tr>
<td>PLAXIS Case 1 w/o PV Drains 2.5</td>
<td>14.4</td>
</tr>
<tr>
<td>PLAXIS Case 1 w/ PV Drains 2.6</td>
<td>7.1</td>
</tr>
<tr>
<td>PLAXIS Case 2 w/o PV Drains 2.5</td>
<td>14.3</td>
</tr>
<tr>
<td>PLAXIS Case 2 w/ PV Drains 2.6</td>
<td>6.8</td>
</tr>
<tr>
<td>PLAXIS Case 3 w/ PV Drains 2.6</td>
<td>2.2</td>
</tr>
<tr>
<td>PLAXIS Case 4 w/ PV Drains 2.5</td>
<td>0.5</td>
</tr>
<tr>
<td>PLAXIS Case 5 w/ PV Drains 2.6</td>
<td>7.1</td>
</tr>
<tr>
<td>PLAXIS Case 6 w/ PV Drains 2.6</td>
<td>7.1</td>
</tr>
<tr>
<td>Settle 3D Case 1 w/ PV Drains 2.1</td>
<td>5</td>
</tr>
<tr>
<td>Settle 3D Case 1 w/o PV Drains 2.1</td>
<td>38</td>
</tr>
</tbody>
</table>
As mentioned previously, the NHDOT had anticipated a total settlement of about 2.5 ft based on the work performed in Portsmouth, NH. Based on this analysis, there appears to be a range in settlement between 1.8 ft and 2.6 ft which seems
reasonable based on the initial prediction. However, as seen in Figure 7-15, the current settlement the field is experiencing has not leveled off yet and cannot be compared to the final settlement determined by each method.

Settle 3D and PLAXIS calculated different time values to 90% consolidation. Settle 3D did not provide an actual time to 90% consolidation and was, therefore, estimated based on the time versus settlement plot seen in Figure 7-15. PLAXIS provides the user with the actual time to 90% consolidation. For Case 1 with drains, Settle 3D predicted a time frame of 5 years and PLAXIS predicted a time frame of 7.1 years. For Model 1 without drains, Settle 3D predicted a time of 38 years and PLAXIS 14.4 years. Both programs show a greater time without the drains, however, Settle 3D has predicted greater than twice the amount of time estimated by PLAXIS.

The differences in settlement and time to 90% consolidation between each method can be due to several factors. Each method was provided with different data for the subsurface conditions based on the different soil model types. The Settle 3D software did not require as many parameters as PLAXIS to define the soil type and therefore may not have modeled the behavior as accurately. Additionally, more assumptions were made in Settle 3D when defining the geotechnical parameters. The drains modeled in PLAXIS were not spaced in a triangular pattern due to modeling restrictions, rather as infinitely long drains in the z-direction. Settle 3D did model the drains in a triangular spacing similar to
that in the field. Lastly, in the field there were various fill heights and spacing between PV-drains that was not addressed in any of the models.

Similar to the PLAXIS models, it appears the Settle 3D model with the drains has a much steeper slope while settling than what the field is experiencing. However, the best fit model still appears to be PLAXIS Model 1 without the drains. It can be assumed that the embankment has at least 3 more years before reaching 90% consolidation, and another 0.5 ft of settlement.
Chapter 8 - PHASE 2 TESTING

8.1 - Introduction

Phase 2 testing was performed 4 months after completion of the embankment construction. This phase consisted of both DMT and CPTu testing. A CPTu profile was performed in each of the five segments of the embankment for comparison with the first phase of testing. Dilatometer testing was performed in Segments 1 and 4, along with a dissipation profile in Segment 1. The primary objective of this testing was to evaluate any increases in strength due to the consolidation of the subsurface soils from the embankment load.

8.2 - Dilatometer Results

Five successful DMT profiles were carried out, four of which were obtained in Segment 1 of the embankment. The phase 2 DMT index parameter profiles performed in Segment 1 have been plotted together in Appendix I. These plots follow similar trends validating the reproducibility of the DMT test during Phase 2.

A typical phase 2 profile is graphed in Figure 8-1. The remainder of the individual plots can be found in Appendix I. The curves appear to follow similar trends to those found during Phase 1. This is more evident in Figure 8-2 with plots for Phase 1 and 2 shown side by side.
Figure 8-1: Phase 2 Segment 1 (Sta. 603 + 20, LT 5) DMT Readings and Material Index Classification
Figure 8-2: Phase 1 and 2 Segment 1 DMT Readings

Figure 8-3 shows one of the Phase 1 plots graphed with two of the Phase 2 DMT index parameter plots. Only one plot from Phase 1 was used due to the similarity between the 3 DMT profiles performed. Both Phase 1 and 2 appear to demonstrate similar trends, however, the Phase 1 curve for each parameter
seems to be slightly greater than what was found during Phase 2, indicating a decrease in strength. Typically an increase in strength would be seen after four months of pore pressure dissipation.

Figure 8-3: Segment 1 DMT Phase 1 and Phase 2

Figure 8-4 plots the horizontal coefficient of consolidation determined by the DMT dissipation tests of Phase 2 with the Phase 1 results. The Phase 1 and 2 results appear to be very similar, indicating little change.
Figure 8-4: Segment 1 Coefficient of Consolidation for Phase 1 and Phase 2
8.3 - Piezocone Results

Five successful CPTu profiles were performed, one in each segment of the embankment. Figure 8-5 shows the tip resistance, sleeve resistance and penetration porewater pressure recorded during Phase 1 and 2 of Segment 1 plotted on the same graph. The curves appear to be very similar with the sleeve resistance being slightly greater for Phase 2. Each segment appears to demonstrate the same trend as seen in Appendix J.

Although the undrained shear strength had been adjusted for Phase 1 to follow the FVT trend, the original curve along with Phase 2 have been plotted in Figure 8-6. Both curves appear to be very similar up to 46 ft below the surface where the Phase 2 curve is slightly less than the Phase 1 curve. This is most likely insignificant, indicating the strength of the soil did not change significantly between Phase 1 and 2.

8.4 - Conclusions

In general, the DMT showed a decrease in strength from Phase 1 to Phase 2, whereas, the CPTu showed little change at all between the two phases. A decrease in strength is not reasonable unless each test was performed on top of a PV drain or within the resulting remolded zones produced during PV installation. Future FVT should be performed to determine the current strength of the marine deposit. Additionally, further research should be done in the understanding of how the PV drains affect the DMT and CPTu testing results.
Figure 8-5: Segment 1 Phase 1 and 2 CPTu profiles for $q_t$, $f_s$ and $u_2$
Figure 8-6: CPTu Undrained Shear Strength for Phase 1 and 2 of Segment 1
Chapter 9 - SUMMARY AND CONCLUSIONS

9.1 - Summary

Laboratory and in situ testing was performed at the future location of a NHDOT geotechnical test embankment to assist in the prediction of long-term and time rate ground settlement of the marine clay native to the site and others in the general seacoast area. In situ testing included: dilatometer testing, dissipation testing using the dilatometer, field vane testing and piezocone testing. Laboratory consolidation testing was performed using samples obtained via piston sampling. After the data collected in the field and laboratory was analyzed, it was used to predict the settlement below the embankment using dilatometer settlement software, hand calculations, finite element analysis and Settle 3D software.

Additional field testing was performed after PV drains were installed and the embankment was constructed to measure the change in strength of the marine deposit. Future testing will be performed to determine long term behavior of the soil.

The following is a summary of the testing results:

1. During Phase 1 of testing, the dilatometer and piezocone tests were used to determine the stratigraphy of the subsurface. In combination with the SPT performed by the NHDOT, the subsurface was identified by 10 ft of
alluvium, followed by the soft marine deposit, glacial till and bedrock. The thickness of the glacial till varied throughout the site. Approximately 55 ft of the marine deposit was found and divided into three sublayers where the top 10 ft and bottom 5 ft was overconsolidated and the 40 ft in between was near normally consolidated.

2. The field and laboratory data appeared to match well with previous research performed in the Seacoast area. Ladd et al. (1972), Findlay (1991), NeJame (1991), Murray (1995) and the UNH CIE 961 class (1997) identified similar stratigraphy and geotechnical parameters to what was found in Newington-Dover. Ladd et al. (1972) performed laboratory and field vane tests on the marine deposit found in Portsmouth. Findlay (1991) performed self-boring pressuremeter tests, NeJame (1991) performed dilatometer tests and Murray (1995) performed Wissa piezocone tests in Portsmouth as well. The CIE 961 (1997) class performed dilatometer and field vane tests in Dover. In general, the Newington-Dover site contained thicker deposits of the marine clay.

3. The unit weight and overconsolidation ratio were estimated using the dilatometer and piezocone data and compared to the consolidation testing data. The undrained shear strength estimated using the dilatometer and piezocone was adjusted to fit the field vane data using recommended methods by Rabasca and Benoît (2008) and Murray (1995). Both corrections worked well.
4. Several consolidation parameters were evaluated including: overconsolidation ratio, compression index, recompression index, initial void ratio and coefficient of consolidation. After correcting the overconsolidation ratio data, an upper overconsolidated layer was found followed by a normally consolidated layer which appeared to match Findlay's (1991) findings. The compression and recompression indices and void ratio were also within the same range as Findlay (1991). The coefficient of consolidation corresponding to the virgin compression curve for Newington-Dover in the normally consolidated zone ranges seemed greater than Ladd's (1972) findings and within the lower end of Findlay's (1991). The in situ horizontal coefficient of consolidation for Newington-Dover in the normally consolidated zone fell slightly above the range determined by NeJame (1991).

5. The coefficient of lateral earth pressure was estimated using the dilatometer. The dilatometer data matched well with Findlay's (1991) minimum self-boring pressuremeter data in the upper clay layer and maximum self-boring pressuremeter data in the lower clay layer.

6. The dilatometer, piezocone and consolidation testing each provided estimates for the permeability of the marine clay. The piezocone permeability was estimated based solely on the soil behavior type and was determined the least accurate. The dilatometer and consolidation data fell within one order of magnitude.
7. Using the Phase 1 testing data, the total settlement was estimated using the DMT settlement software, PLAXIS and Settle 3D. PLAXIS Model 1 without the PV drains appeared to best fit the current field data.

9.2 - Conclusions

The marine deposit extends as deep as 70 ft below the ground surface with an average thickness of 55 ft. The consolidation testing, along with FVT, DMT and CPTu, provided sufficient data in classifying the marine deposit in terms of geotechnical soil parameters which were in good agreement with past research findings.

The marine deposit can be divided into three sub layers with the following geotechnical properties.

<table>
<thead>
<tr>
<th>Depth Below Surface (ft)</th>
<th>Y (pcf)</th>
<th>e_o</th>
<th>LL</th>
<th>PL</th>
<th>s_u (psf)</th>
<th>c_v (ft²/day)</th>
<th>c_h (ft²/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
<td>10-20</td>
<td>118</td>
<td>1.1</td>
<td>35</td>
<td>22</td>
<td>400</td>
<td>0.21</td>
</tr>
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<td>Layer 2</td>
<td>20-60</td>
<td>110</td>
<td>1.2</td>
<td>38</td>
<td>24</td>
<td>450</td>
<td>0.12</td>
</tr>
<tr>
<td>Layer 3</td>
<td>60-65</td>
<td>120</td>
<td>0.3</td>
<td>38</td>
<td>20</td>
<td>900</td>
<td>0.32</td>
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</table>

<table>
<thead>
<tr>
<th>OCR</th>
<th>C_s</th>
<th>C_r</th>
<th>K_o</th>
<th>k (ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Layer 1</td>
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<td>0.28</td>
<td>0.05</td>
<td>1.22</td>
</tr>
<tr>
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<td>0.31</td>
<td>0.05</td>
<td>1.00</td>
</tr>
<tr>
<td>Layer 3</td>
<td>1</td>
<td>0.19</td>
<td>0.04</td>
<td>0.73</td>
</tr>
</tbody>
</table>

After about 5 months, the marine deposit has experienced 0.95 ft of settlement and is expected to settle at least another 0.5 ft in the next 3 years.
9.3 - **Future Research**

For phase 3 of testing, a FVT profile in addition to the DMT and CPTu profiles would benefit this research. During phase 2, it was uncertain whether the strength in the marine deposit had remained the same or decreased. The FVT would provide the actual strength of the deposit versus the estimates made from the DMT and CPTu.

Next, further investigation in the effects of PV drains on in situ testing results should be addressed. The DMT and CPTu did not provide consistent results, although, prior to construction, they did. The area of influence around the drain should be considered. Additionally, based on the FEA results, the drains may not be working as well as they could which may be the result of smearing and clogging of the pores.
REFERENCES


Congress 2012: State of the Art and Practice in Geotechnical Engineering, Oakland, CA.


Ladd, Charles C., M.ASCE. "Test Embankment on Sensitive Clay." (1972)


APPENDIX A - PHASE 1 CONSOLIDATION LABORATORY DATA AND RESULTS
Table A-1: Segment 1 Q-B212 (Sta. 603 + 29, LT 1) Consolidation Test Results

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>Elevation (ft)</th>
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<th>$c_v$ (ft$^2$/day)</th>
<th>$c_r$ (ft$^2$/day)</th>
<th>$k$ (ft/day)</th>
<th>$C_c$</th>
<th>$C_r$</th>
<th>$\sigma'$ (tsf)</th>
<th>$\sigma''$ (tsf)</th>
<th>OCR</th>
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</thead>
<tbody>
<tr>
<td>11.7</td>
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<td>1.20</td>
<td>0.42</td>
<td>0.16</td>
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<td>2.05</td>
<td>0.35</td>
<td>5.94</td>
</tr>
<tr>
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<td>0.35</td>
<td>6.05</td>
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<td>-3.2</td>
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</tr>
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<td>-8.4</td>
<td>0.98</td>
<td>0.75</td>
<td>0.10</td>
<td>1.55E-04</td>
<td>0.33</td>
<td>0.04</td>
<td>0.70</td>
<td>0.51</td>
<td>1.37</td>
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<td>26.8*</td>
<td>-14.6</td>
<td>1.31</td>
<td>1.82</td>
<td>0.11</td>
<td>3.72E-04</td>
<td>0.31</td>
<td>0.06</td>
<td>0.70</td>
<td>0.63</td>
<td>1.11</td>
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<td>30.4*</td>
<td>-18.2</td>
<td>1.36</td>
<td>0.69</td>
<td>0.12</td>
<td>2.03E-04</td>
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<td>0.06</td>
<td>0.90</td>
<td>0.70</td>
<td>1.29</td>
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<td>30.9</td>
<td>-18.7</td>
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<td>0.64</td>
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<td>0.85</td>
<td>0.71</td>
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<td>35.9*</td>
<td>-23.7</td>
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<td>0.45</td>
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<td>3.62E-04</td>
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<td>40.4</td>
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<td>0.12</td>
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<td>1.05</td>
<td>0.98</td>
<td>1.08</td>
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<td>6.05E-05</td>
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<td>0.06</td>
<td>1.10</td>
<td>1.07</td>
<td>1.03</td>
</tr>
<tr>
<td>55.4</td>
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<td>1.56</td>
<td>0.21</td>
<td>6.05E-05</td>
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<td>0.04</td>
<td>1.90</td>
<td>1.17</td>
<td>1.63</td>
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<td>60.5</td>
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<td>65.8</td>
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<td>0.03</td>
<td>1.10</td>
<td>1.36</td>
<td>0.81</td>
</tr>
</tbody>
</table>

* Software issues with initial load
APPENDIX B - DILATOMETER TESTING DATA REDUCTION
Table B-1 summarizes the DMT calibration data that was used to correct the initial A, B and C readings for membrane stiffness and zero offset to $p_0$, $p_1$ and $p_2$. The SDMT Elab software does not allow the user to input both the low and high gage offsets. Therefore, the low gage offset was entered into the software because it was the primary gage in testing and used for calibrating each reading. An Excel worksheet was also formulated to use both the low and high gage offsets.

<table>
<thead>
<tr>
<th>Date</th>
<th>$Z_w$ (m)</th>
<th>$Z_{abs}$ (m)</th>
<th>$Z_{m\ low}$ (bar)</th>
<th>$Z_{m\ high}$ (bar)</th>
<th>$\Delta A$ (bar)</th>
<th>$\Delta B$ (bar)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Q-B213</td>
<td>8/5/2012</td>
<td>1.22</td>
<td>3.78</td>
<td>0.05</td>
<td>0.20</td>
<td>0.22</td>
</tr>
<tr>
<td>Q-B215</td>
<td>8/12/12</td>
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<td>3.66</td>
<td>0.06</td>
<td>0.20</td>
<td>0.25</td>
</tr>
<tr>
<td>Q-B218</td>
<td>8/31/12</td>
<td>1.22</td>
<td>3.75</td>
<td>0.06</td>
<td>0.18</td>
<td>0.22</td>
</tr>
</tbody>
</table>

The program states that if $\Delta A$ and $\Delta B$ do not remain constant from the beginning to the end of the test, the smaller value should be used. The user is also given the option to enter the unit weight of the top layer (Gamma top); this was left as 17kN/m$^3$ (108.2 pcf) due limited information regarding the site.

After $p_0$, $p_1$ and $p_2$ were calculated, the "intermediate" DMT index parameters were calculated as described in Chapter 3. Before determining the remaining parameters, the unit weight ($\gamma_T$) of the soil was estimated. Figure B-1 or equation [B-1] can be used in determining the unit weight of the soil. The unit weight determined by these methods appeared to be an average of 10 pcf less than the values calculated during consolidation testing.
After the unit weight was determined, the total overburden stress ($\sigma_{vo}$) and effective overburden stress ($\sigma'_{vo}$) were calculated. From there, the overconsolidation ratio (OCR), in situ coefficient of lateral earth pressure ($K_0$), vertical drained constrained modulus ($M_{DMT}$), undrained shear strength ($s_u$) and friction angle ($\phi'$) were evaluated.

**SOIL DESCRIPTION and ESTIMATED $\gamma/\gamma_w$**

![Diagram of soil classification and unit weight estimation](image)

Figure B-1: Chart for Estimating Soil Type and Unit Weight ($\gamma_T$) (Marchetti and Crapps, 1981)
\[ \gamma_T = 1.12 \gamma_w \left( \frac{E_D}{\sigma_{atm}} \right)^{0.1} (I_D)^{-0.05} \]  

Where \( \sigma_{atm} \) is atmospheric pressure, \( \gamma_w \) is the unit weight of water, \( E_D \) is the dilatometer modulus and \( I_D \) is the material index (NHI, 2002).

\[ \sigma_{v0} = (\gamma_T) \text{depth} \]  

\[ \sigma'_{v0} = \sigma_{v0} - u_0 \]  

\[ u_0 = (\gamma_w) \text{depth, depth > depth of water table} \]  

\[ OCR = (0.5K_D)^{1.56}, \text{for } I_D < 1.2 \]  

Where \( K_D \) is the horizontal stress index (Marchetti et al., 2001).

The in situ coefficient of lateral earth pressure is based on the measured lateral stress. An approximate estimate is sufficient and was calculated as shown in equation \([B-6]\) (Marchetti et al., 2001).

\[ K_0 = (K_D/1.5)^{0.47} - 0.6, \text{for } I_D < 1.2 \]  

The vertical drained constrained modulus and undrained shear strength are believed to be the most reliable parameters obtained by the DMT and were calculated using equation \([B-7]\) and \([B-8]\), respectively (Marchetti, 2001).

\[ M_{DMT} = R_M E_D \]  

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if \( I_D \leq 0.6 \) : \( R_M = 0.14 + 2.36 \log K_D \)

if \( I_D \geq 3 \) : \( R_M = 0.5 + 2 \log K_D \)

if \( 0.6 < I_D < 3 \) : \( R_M = 0.14 + 0.15(I_D - 0.6) + \left(2.5 - (0.14 + 0.15(I_D - 0.6))\right) \log K_D \)

if \( K_D > 10 \) : \( R_M = 0.32 + 2.18 \log K_D \)

if \( R_M < 0.85 \) : \( R_M = 0.85 \)

\[
S_u = 0.22a'\nu_0(0.5K_D)^{1.25}, \text{for } I_D < 1.2 \tag{B-8}
\]

There are currently two methods in determining the friction angle (\( \phi' \)) for sand based on the DMT results. The first method is an iterative method developed by Schmertmann (1982) that determines both the friction angle and in situ coefficient of lateral earth pressure. The second method is a derivation of the first method and was used in these calculations (Marchetti et al., 2001).

\[
\phi' = 28^\circ + 14.6^\circ \log K_D - 2.1^\circ \log^2 K_D, \text{for } I_D > 1.8 \tag{B-9}
\]

Table B-2 compares the hand calculations SDMT Elab results for Phase 1 Q-B213 at a depth of 8 ft (2.44m). The results are similar but are not completely the same because the B reading at this depth was recorded on the large gage, which has a different offset value than the software used in calculations.
The hand calculations were determined slightly more accurate and, therefore, used in the remainder of the study.

The Marchetti Dmt Dissip software plotted the A reading against the logarithm of time for the dissipation tests. From there, the software uses the following equation to determine the coefficient of consolidation \( (c_h) \) based on the time associated with the contraflexure point \( (t_{flex}) \) of the curve (Marchetti et al., 2001).

\[
c_h = \frac{7c m^2}{t_{flex}}
\]

Using the coefficient of consolidation and horizontal drained constrained modulus \( (M_h) \), the permeability \( (k) \) could be calculated (Schmertmann, 1988).
\[ M_h = K_0 M_{DMT} \]  \[ [B-11] \]

\[ k = c_h Y_w / M_h \]  \[ [B-12] \]
APPENDIX C - PHASE 1 DILATOMETER TESTING RESULTS
Figure C-1: Segment 1 Q-B213 (Sta. 603 + 28, LT 14) DMT Readings and Material Index Classification
Figure C-2: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Readings and Material Index Classification
Figure C-3: Segment 1 Q-B218 (Sta. 603 + 35, LT 25) DMT Readings and Material Index Classification
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 5.183 m

Uo,equil = 36.749 kPa

S-shape insufficiently defined to identify Tflex

Figure C-4: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -5 ft
Figure C-5: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -8 ft
Dissipation Test "DMTA"

Dissipation Depth = 7.165 m

\[ U_0, \text{equil} = 56.17 \text{ kPa} \]
\[ T_{\text{flex}} = 52 \text{ min} \]

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 2.2 \times 10^{-3} \text{ cm}^2 / \text{sec} \]

Figure C-6: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -11.5 ft
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 8.232 m

\[ U_{o,\text{equil}} = 66.627 \text{ kPa} \]
\[ T_{\text{flex}} = 100 \text{ min} \]

\[
C_{h,\text{OC}} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 1.2 \times 10^{-3} \text{ cm}^2/\text{sec}
\]

Figure C-7: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -15 ft
**N HDOT**
Newington-Dover  
Q-B215  

**DISSIPATION TEST "DMTA"**

<table>
<thead>
<tr>
<th>DISSIP</th>
<th>30.5ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Depth</td>
<td>9.299 m</td>
</tr>
</tbody>
</table>

**DISSIP DEPTH = 9.299 m**

\[
U_0, \text{equil} = 77.084 \text{ kPa} \\
T_{\text{flex}} = 140 \text{ min}
\]

![Graph of A vs. Time (min)](image)

\[
C_{n,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 8.2 \times 10^{-4} \text{ cm}^2/\text{sec}
\]

**Figure C-8: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -18.5 ft**

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DISSIP DEPTH = 10.366 m

\[ U_{0,\text{equil}} = 87.541 \text{ kPa} \]
\[ T_{\text{flex}} = 160 \text{ min} \]

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 7.4 \times 10^{-4} \text{ cm}^2/\text{sec} \]

Figure C-9: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -22 ft
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 11.433 m

U₀, equil = 97.998 kPa
Tflex = 160 min

Figure C-10: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -25.5 ft
Newington-Dover  Q-B215
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 12.957 m
U₀, equil = 112.937 kPa
Tflex = 140 min

Figure C-11: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -30.5 ft

Cₜ,OC = \( \frac{7 \text{ cm}^2}{\text{Tflex}} = 8.1 \times 10^{-4} \text{ cm}^2 / \text{sec} \)
DISSIP DEPTH = 14.482 m
U₀, equil = 127.876 kPa
T_{flex} = 150 min

Figure C-12: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -35.5 ft

C_{h, oc} = \frac{7 \text{ cm}^2}{T_{flex}} = 7.8 \times 10^{-4} \text{ cm}^2/\text{sec}
Newington-Dover Q-B215

DISSIPATION TEST "DMTA"

Dissipation Depth = 16.006 m

$U_{0,eq} = 142.814 \text{ kPa}$

$T_{flex} = 140 \text{ min}$

Figure C-13: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -40.5 ft
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 17.53 m

$U_{0,\text{equil}} = 157.753$ kPa

$T_{\text{flex}} = 36$ min

Figure C-14: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -45.5 ft

$C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 3.3 \times 10^{-3} \text{ cm}^2/\text{sec}$
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 19.055 m

\[ U_0,\text{equil} = 172.692 \text{kPa} \]
\[ T_{\text{flex}} = 2.2 \text{ min} \]

Figure C-15: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -50.5 ft

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 0.054 \text{ cm}^2/\text{sec} \]
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 20.579 m

\[ U_{o,\text{equil}} = 187.63 \text{ kPa} \]
\[ T_{\text{flex}} = 0.84 \text{ min} \]

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 0.14 \text{ cm}^2/\text{sec} \]

Figure C-16: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -55.5 ft
Newington-Dover  Q-B215  DISSIP

DISSIPATION TEST "DMTA"

DISSIP DEPTH = 22.104 m

Uo, equil = 202.569 kPa

S-shape insufficiently defined
to identify Tflex

Figure C-17: Segment 1 Q-B215 (Sta. 603 + 40, LT 4) DMT Dissipation Test at Elevation -60.5 ft
Figure C-18: Segment 1 Q-B218 (Sta. 603 + 35, LT 25) DMT Dissipation Test at Elevation 0.3 ft
Dissipation Test "DMTA"

Dissip DEPTH = 6.707 m

\[ U_{o,\text{equil}} = 51.688 \text{ kPa} \]
\[ T_{\text{flex}} = 37 \text{ min} \]

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 3.2 \times 10^{-3} \text{ cm}^2/\text{sec} \]

Figure C-19: Segment 1 Q-B218 (Sta. 603 + 35, LT 25) DMT Dissipation Test at Elevation -9.7 ft
Figure C-20: Segment 1 Q-B218 (Sta. 603 + 35, LT 25) DMT Dissipation Test at Elevation -19.7 ft
Figure C-21: Segment 1 Q-B218 (Sta. 603 + 35, LT 25) DMT Dissipation Test at Elevation -29.7 ft
APPENDIX D - PHASE 1 FIELD VANE TESTING DATA AND RESULTS
Figure D-1: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -3.8 ft
Figure D-2: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -6.1 ft
NHDOT Geotechnical Test Embankment
Newington-Dover, NH
University of New Hampshire
Geonor H-10 Vane Borer Testing

Figure D-3: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -9.38 ft
Figure D-4: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -12.66 ft
Figure D-5: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -15.94 ft
Figure D-6: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -19.22 ft
Figure D-7: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -22.5 ft
Figure D-8: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -25.82 ft
Figure D-9: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -29.1 ft
Figure D-10: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -32.38 ft

Free Field Conditions
Boring: Q-B214
Test No.: 10
Depth: 44.68 ft
7/16/2012
NHDOT Geotechnical Test Embankment
Newington-Dover, NH
University of New Hampshire
Geonor H-10 Vane Borer Testing

Free Field Conditions
Boring: Q-B214
Test No.: 11
Depth: 47.96 ft
7/16/2012

Figure D-11: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -35.66 ft

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NHDOT Geotechnical Test Embankment  
Newington-Dover, NH

University of New Hampshire  
Geonor H-10 Vane Borer Testing

Figure D-12: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at  
Elevation -38.91 ft
Figure D-13: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -42.22 ft
Figure D-14: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -45.5 ft
Figure D-15: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -48.78 ft
Figure D-16: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test at Elevation -52.06 ft
<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>$s_{u\text{ peak}}$ (psf)</th>
<th>$s_{u\text{ remolded}}$ (psf)</th>
<th>$S_t$</th>
<th>$s_{u\text{ peak}}$ Chandler correction (psf)</th>
<th>$s_{u\text{ peak}}$ Aas et al. correction (psf)</th>
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<tbody>
<tr>
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<td>466.7</td>
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<td>-</td>
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<td>547.7</td>
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<td>98.0</td>
<td>6.8</td>
<td>651.3</td>
<td>511.5</td>
</tr>
</tbody>
</table>
Table D-17: Segment 1 Q-B214 (Sta. 603 + 15, CL) Field Vane Test Shear Stress and Sensitivity Results
APPENDIX E - PIEZOCONES TESTING DATA REDUCTION
The CPeT-IT software was provided with the following assumptions for each cone profile.

Table E-1: CPTu Input Parameters

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Atmospheric Pressure, ( p_a ) (MPa)</td>
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</tr>
<tr>
<td>Net Area Ratio for Cone, ( a )</td>
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</tr>
<tr>
<td>Relative Density Constant, ( C_{Dr} )</td>
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</tr>
<tr>
<td>Undrained Shear Strength Cone Factor For Clays, ( N_s )</td>
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</tr>
<tr>
<td>Overconsolidation Ratio Number, ( K_{OCR} )</td>
<td>0.33</td>
</tr>
<tr>
<td>Unit Weight of Water, ( \gamma_w ) (kN/m³)</td>
<td>9.81</td>
</tr>
<tr>
<td>Probe Radius (m)</td>
<td>0.0183</td>
</tr>
</tbody>
</table>

The software begins by calculating the corrected tip resistance \( (q_t) \) as seen in equation [E-1] (NHI, 2002). In soft clays, the measured tip resistance \( (q_c) \) must be corrected for pore water pressures acting on the cone. The net area ratio \( (a) \) and water pressure \( (u_2) \) at the base of the sleeve are used to correct the tip resistance.

\[
q_t = q_c + (1 - a) u_2
\]

Additionally, the measured sleeve friction \( (f_s) \) should be corrected similarly due to the unequal end areas at either end of the sleeve. For this investigation, there are no measurements available for the water pressure at the top of the sleeve \( (u_3) \) and the corrected sleeve friction \( (f_t) \) was assumed to be equal to the measured sleeve friction.
\[ f_t = f_s - \frac{u_2 A_{sb} - u_3 A_{st}}{A_s} = f_s \]  \hspace{1cm} \text{[E-2]}

Where \( A_s \) is the surface area of the sleeve, \( A_{sb} \) is the cross-sectional area of the sleeve at the base and the \( A_{st} \) is the cross-sectional area of the sleeve at the top (FHWA, 2002).

Next, the software calculates the friction ratio \( (R_f) \) using equation [E-3] (Robertson et al., 2010).

\[ R_f = \frac{f_s}{q_t} \times 100\% \]  \hspace{1cm} \text{[E-3]}

The friction ratio can then be used to determine the soil behavior type (SBT) in combination with Figure E-1. In 1990, Robertson suggested use of normalized soil behavior type \( (\text{SBT}_N) \) charts to account for the increase in effective overburden stress with depth. Figure E-2 was used to determine the normalized soil behavior type.

The normalized soil behavior type \( Q_t - F_r \) chart identifies general trends in ground response (i.e. increasing density, OCR, age and sensitivity). The normalized soil behavior type \( Q_t - B_q \) chart can help in the identification of soft, saturated fine grained soils where the excess pore pressures are too large. Equation [E-4] was used to calculate the normalized pore pressure parameter \( (B_q) \). Equation [E-5] was used to calculate the normalized friction ratio \( (F_r) \). Equation [E-6] was used
to calculate the normalized cone penetration resistance ($Q_t$) (Robertson et al., 2010).

![Diagram of CPT Soil Behavior Type (SBT)](image)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil Behavior Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive, fine grained</td>
</tr>
<tr>
<td>2</td>
<td>Organic soils - clay</td>
</tr>
<tr>
<td>3</td>
<td>Clay – silty clay to clay</td>
</tr>
<tr>
<td>4</td>
<td>Silt mixtures – clayey silt to silty clay</td>
</tr>
<tr>
<td>5</td>
<td>Sand mixtures – silty sand to sandy silt</td>
</tr>
<tr>
<td>6</td>
<td>Sands – clean sand to silty sand</td>
</tr>
<tr>
<td>7</td>
<td>Gravelly sand to dense sand</td>
</tr>
<tr>
<td>8</td>
<td>Very stiff sand to clayey sand*</td>
</tr>
<tr>
<td>9</td>
<td>Very stiff fine grained*</td>
</tr>
</tbody>
</table>

* Heavily overconsolidated or cemented

$P_c = \text{atmospheric pressure} = 100 \text{ kPa} = 1 \text{ tsf}$

Figure E-1: CPT Soil Behavior Type (SBT) (Robertson et al., 1986)
Figure E-2: Normalized CPT Soil Behavior Type (SBTN) charts (Robertson, 1990)

\[
B_q = \frac{u_2 - u_0}{q_t - \sigma'_{vo}} \quad \text{[E-4]}
\]

\[
F_r = \frac{f_s}{q_t - \sigma'_{vo}} \times 100\% \quad \text{[E-5]}
\]

\[
Q_t = \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \quad \text{[E-6]}
\]

The estimated soil unit weight (γ) is predicted according to Table E-2 and the equations below are used in determining the total overburden stress (σ_{vo}), pore pressure (u_0) and effective overburden stress (σ'_{vo}).
Table E-2: Estimated Unit Weight (Lunne et al., 1997)

<table>
<thead>
<tr>
<th>SBT</th>
<th>Approximate Unit Weight (lb/ft³)</th>
<th>(kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>111.4</td>
<td>17.5</td>
</tr>
<tr>
<td>2</td>
<td>79.6</td>
<td>12.5</td>
</tr>
<tr>
<td>3</td>
<td>111.4</td>
<td>17.5</td>
</tr>
<tr>
<td>4</td>
<td>114.6</td>
<td>18.0</td>
</tr>
<tr>
<td>5</td>
<td>114.6</td>
<td>18.0</td>
</tr>
<tr>
<td>6</td>
<td>114.6</td>
<td>18.0</td>
</tr>
<tr>
<td>7</td>
<td>117.3</td>
<td>18.5</td>
</tr>
<tr>
<td>8</td>
<td>120.9</td>
<td>19.0</td>
</tr>
<tr>
<td>9</td>
<td>124.1</td>
<td>19.5</td>
</tr>
<tr>
<td>10</td>
<td>127.3</td>
<td>20.0</td>
</tr>
<tr>
<td>11</td>
<td>130.5</td>
<td>20.5</td>
</tr>
<tr>
<td>12</td>
<td>120.9</td>
<td>19.0</td>
</tr>
</tbody>
</table>

\[
\sigma_{vo} = \gamma \times \text{depth} \quad \text{[E-7]}
\]

\[
u_{o} = \gamma_{w} \times (\text{depth} - z_{m}) \quad \text{[E-8]}
\]

\[
\sigma'_{vo} = \sigma_{vo} - u_{o} \quad \text{[E-9]}
\]

Alternatively, the Soil Behavior Type index \( (l_c) \) can be calculated. The Soil Behavior Type index represents the radius of the essentially concentric circles that represent the boundaries between each SBT zone (Robertson et al., 2010).

\[
l_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5} \quad \text{[E-10]}
\]
Table E-3: $I_c$ and SBT relationship (Robertson, 2010)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil Behavior Type</th>
<th>$I_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive, fine grained</td>
<td>N/A</td>
</tr>
<tr>
<td>2</td>
<td>Organic soils – clay</td>
<td>&gt; 3.6</td>
</tr>
<tr>
<td>3</td>
<td>Clays – silty clay to clay</td>
<td>2.95 – 3.6</td>
</tr>
<tr>
<td>4</td>
<td>Silt mixtures – clayey silt to silty clay</td>
<td>2.60 – 2.95</td>
</tr>
<tr>
<td>5</td>
<td>Sand mixtures – silty sand to sandy silt</td>
<td>2.05 – 2.6</td>
</tr>
<tr>
<td>6</td>
<td>Sands – clean sand to silty sand</td>
<td>1.31 – 2.05</td>
</tr>
<tr>
<td>7</td>
<td>Gravelly sand to dense sand</td>
<td>&lt; 1.31</td>
</tr>
<tr>
<td>8</td>
<td>Very stiff sand to clayey sand*</td>
<td>N/A</td>
</tr>
<tr>
<td>9</td>
<td>Very stiff, fine grained*</td>
<td>N/A</td>
</tr>
</tbody>
</table>

* Heavily overconsolidated or cemented

Before calculation the soil behavior type index, the software calculates the normalized cone resistance ($Q_{tn}$), recalculates the soil behavior type index and iterates until the change in $n$ is less than 0.01 (Robertson et al., 2010).

\[
Q_{tn} = \frac{q_t - \sigma_{vo}}{p_a} \times \left( \frac{p_a}{\sigma'_{vo}} \right)^n
\]  

\[
n = 0.381I_c + 0.05 \frac{\sigma'_{vo}}{p_a} - 0.15
\]

The permeability can be estimated using the normalized soil behavior type and the Table E-4.
The NHDOT most commonly performs the Standard Penetration Test (SPT) during geotechnical investigations. The SPT is affected by borehole preparation and size, sampler, rod length and energy efficiency of the hammer-anvil-operator system. The energy efficiency of the SPT system is normally expressed in terms of the rod energy ratio (ERr) where 60% is the accepted reference value. Many studies have been presented relating the SPT N-value to the CPT cone penetration resistance. Robertson et al. (1983) reviewed the correlations and presented the relationship as seen in Figure E-3. Figure E-3 requires the mean grain size in determining the N-value. If the mean grain size is not available it can be estimated directly from the SBT charts as summarized in Table E-5. (Robertson and Gregg Drilling, 2010)

Table E-4: Estimated Permeability (Lunne et al., 1997)

<table>
<thead>
<tr>
<th>SBTn</th>
<th>Permeability (ft/sec)</th>
<th>(m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>3 x 10^6</td>
<td>1 x 10^3</td>
</tr>
<tr>
<td>2</td>
<td>3 x 10^7</td>
<td>1 x 10^3</td>
</tr>
<tr>
<td>3</td>
<td>1 x 10^8</td>
<td>3 x 10^4</td>
</tr>
<tr>
<td>4</td>
<td>3 x 10^6</td>
<td>1 x 10^4</td>
</tr>
<tr>
<td>5</td>
<td>3 x 10^3</td>
<td>1 x 10^4</td>
</tr>
<tr>
<td>6</td>
<td>3 x 10^4</td>
<td>1 x 10^4</td>
</tr>
<tr>
<td>7</td>
<td>3 x 10^2</td>
<td>1 x 10^4</td>
</tr>
<tr>
<td>8</td>
<td>3 x 10^6</td>
<td>1 x 10^4</td>
</tr>
<tr>
<td>9</td>
<td>1 x 10^3</td>
<td>3 x 10^4</td>
</tr>
</tbody>
</table>
Figure E-3: CPT-SPT correlations with mean grain size (Robertson et al., 1983)

Table E-5: Suggested \( \left( \frac{q_c}{p_a} \right) / N_{60} \) ratios based on SBT (Robertson and Gregg Drilling, 2010)

<table>
<thead>
<tr>
<th>Zone</th>
<th>Soil Behavior Type</th>
<th>( \left( \frac{q_c}{p_a} \right) / N_{60} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Sensitive fine grained</td>
<td>2.0</td>
</tr>
<tr>
<td>2</td>
<td>Organic soils – clay</td>
<td>1.0</td>
</tr>
<tr>
<td>3</td>
<td>Clays: clay to silty clay</td>
<td>1.5</td>
</tr>
<tr>
<td>4</td>
<td>Silt mixtures: clayey silt &amp; silty clay</td>
<td>2.0</td>
</tr>
<tr>
<td>5</td>
<td>Sand mixtures: silty sand to sandy silt</td>
<td>3.0</td>
</tr>
<tr>
<td>6</td>
<td>Sands: clean sands to silty sands</td>
<td>5.0</td>
</tr>
<tr>
<td>7</td>
<td>Dense sand to gravelly sand</td>
<td>6.0</td>
</tr>
<tr>
<td>8</td>
<td>Very stiff sand to clayey sand*</td>
<td>5.0</td>
</tr>
<tr>
<td>9</td>
<td>Very stiff fine-grained*</td>
<td>1.0</td>
</tr>
</tbody>
</table>
Jefferies and Davies (1993) suggested the following relationship for a better estimate of the N-value than the actual SPT test, which is used in the software:

$$\frac{(q_t/p_a)}{N_{60}} = 8.5\left(1 - \frac{f_c}{4.6}\right)$$  \[E-13\]

The following relationship can be used to determine the undrained shear strength ($s_u$). $N_{kt}$ varies from 10 to 18, with an average of 14 for $s_{u(ave)}$ which was used in this analysis. Lunne et al. (1997) showed that $N_{kt}$ varies with $B_q$ inversely. The undrained shear strength would be 0 in zones 5, 6, 7 and 8.

$$s_u = \frac{q_t - \sigma_v}{N_{kt}}$$  \[E-14\]

The soil sensitivity ($S_t$) is typically determined by the Field Vane Test and represents the ratio of undisturbed peak undrained shear strength ($s_u$) to the remolded undrained shear strength ($s_{u(rem)}$). The estimate of sensitivity should be used as a guide only due to difficulties in accuracy (Robertson et al., 2010).

$$S_t = \frac{s_u}{s_{u(rem)}} = \frac{q_t - \sigma_v}{N_{kt}} (1/f_s) = 7/F_r$$  \[E-15\]

Overconsolidation ratio (OCR) is defined as the ratio of the maximum past effective consolidation stress ($\sigma'_p$) and the present effective overburden stress ($\sigma'_o$). Kulhawy and Mayne suggested (1990):
Higher values of $k$ are suggested in heavily overconsolidated clays. The overconsolidation ratio is equal to 0 in zones 5, 6, 7 and 8.

Currently, there is no reliable method in determining the actual in-situ stress ratio ($K_0$) from the CPT, but estimates have been made. Figure E-4 can be used in estimating the in-situ stress ratio.

![OCR and K_0 from s_u/\sigma'_w and Plasticity Index, I_p (Andresen et al., 1979)](image)

Many studies have been made in assessing the friction angle from the CPT ($\phi'$) in clean sands. Kulhawy and Mayne suggested a relationship for clean, rounded,
uncemented quartz sands that is used in the software calculations. The friction angle would be 0 in zones 1, 2, 3, 4, and 9.

$$\varphi' = 17.6 + 11\log(Q_{tn})$$  \[E-17\]

The relative density ($D_r$) is estimated as seen in equation [E-18]. The relative density would be equal to 0 in zones 1, 2, 3, 4, and 9 (Robertson et al., 2010).

$$D_r^2 = \frac{Q_{tn}}{C_{Dr}} \times 100\%$$  \[E-18\]

Young's Modulus ($E_s$) is sensitive to stress history, aging and soil mineralogy and can therefore only be estimated. The following equation and graph can be used in calculating young's modulus (Robertson et al., 2010).

$$E_s = \alpha_E(q_t - \sigma_{vo})$$  \[E-19\]

$$\alpha_E = 0.015[10^{(0.55l_c+1.68)}]$$  \[E-20\]
The small strain shear modulus \( G_0 \) is calculated by the software as follows (Robertson et al., 2010):

\[
G_0 = \alpha_M (q_t - \sigma_{vo})
\]  \[E-21\]

\[
\alpha_M = 0.0188\left[10^{(0.55I_c+1.68)}\right]  \]  \[E-22\]

The Constrained Modulus \( M \) was calculated by following equation [E-23] (Robertson et al., 2010).

**Figure E-5: Evaluation of drained Young's modulus from CPT for young, uncemented silica sands (Robertson and Gregg Drilling, 2010)**
$$M = \alpha_M (q_t - \sigma_v)$$

when \( l_c > 2.20 \)
\[ \alpha_M = Q_t \text{ when } Q_t < 14 \]
\[ \alpha_M = 14 \text{ when } Q_t > 14 \]

when \( l_c < 2.20 \)
\[ \alpha_M = 0.0188[10^{(0.55l_c+1.68)}] \]
APPENDIX F - PHASE 1 PIEZOCONE TESTING RESULTS
Figure F-1: Segment 1 Q-B220 (Sta. 603 + 37, RT 16) CPTu Readings
NHDOT Geotechnical Test Embankment
Newington-Dover, NH
Phase 1: CPTu Q-B221

Figure F-2: Segment 1 Q-B221 (Sta. 603 + 47, RT 11) CPTu Readings
Figure F-3: Segment 2 Q-B222 (Sta. 604 + 92, LT 6) CPTu Readings
Figure F-4: Segment 2 Q-B223 (Sta. 605 + 04, LT 6) CPTu Readings
Figure F-5: Segment 3 Q-B224 (Sta. 606 + 76, CL) CPTu Readings
NHDOT Geotechnical Test Embankment
Newington-Dover, NH
Phase 1: CPTu Q-B225

Figure F-6: Segment 3 Q-B225 (Sta. 606 + 86, CL) CPTu Readings
Figure F-7: Segment 4 Q-B226 (Sta. 609 + 05, LT 10) CPTu Readings
NHDOT Geotechnical Test Embankment
Newington-Dover, NH
Phase 1: CPTu Q-B227

Figure F-8: Segment 4 Q-B227 (Sta. 609 + 09, CL) CPTu Readings
Figure F-9: Segment 5 Q-B228 (Sta. 611 + 00, LT 6) CPTu Readings
APPENDIX G - DMT SETTLEMENT CALCULATIONS
The increase in stress with respect to depth was estimated using the Boussinesq Method for embankments. In 1885, Boussinesq developed equations for the state of stress within a homogenous, isotropic, linearly elastic material for a point load acting perpendicular to the surface. From there, various load types were derived (Holtz et al., 2011). The Boussinesq Method for embankments is calculated using equation [G-1].

\[
\alpha = \tan^{-1}\left(\frac{b}{R_2}\right) - \tan^{-1}\left(\frac{b-a}{R_2}\right)
\]

\[
\beta = \tan^{-1}\left(\frac{b-a}{R_2}\right)
\]

\[
q_{oBoussinesq} = \gamma H = (120pcf)(12ft)
\]

\[
\sigma_z = 2 \cdot \frac{q_o}{\pi} \left[\beta + \frac{x\alpha}{a} - \frac{z}{R_2^2}(x - b)\right]
\]

After the Boussinesq Method was performed, the total settlement was calculated as seen in Figure G-1.
Figure G-1: Recommended Method for Settlement Calculation Using DMT
Totani et al., 1998

\[ S_{1-DMT} = \sum \frac{\Delta \sigma_v}{M_{DMT}} \Delta z \]
APPENDIX H - PLAXIS 2D 2011 RESULTS
Figure H-1: PLAXIS 2D Model 1 without Drains Horizontal Displacement
Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 2)
Minimum value = -2.509 ft (Element 1136 at Node 4876)

Figure H-2: PLAXIS 2D Model 1 without Drains Vertical Displacement
Total displacements $u_x$

Maximum value = 0.4145 ft (Element 1117 at Node 3238)
Minimum value = -0.4145 ft (Element 578 at Node 6518)

Figure H-3: PLAXIS 2D Model 2 with Drains Horizontal Displacement
Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 10160)
Minimum value = -2.606 ft (Element 655 at Node 4856)

Figure H-4: PLAXIS 2D Model 2 with Drains Vertical Displacement
Figure H-5: PLAXIS 2D Model 2 without Drains Horizontal Displacement

Total displacements $u_x$

Maximum value = 0.5087 ft (Element 879 at Node 3106)
Minimum value = -0.5085 ft (Element 862 at Node 7263)
Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 10777)
Minimum value = -2.487 ft (Element 1303 at Node 5258)

Figure H-6: PLAXIS 2D Model 2 without Drains Vertical Displacement
Figure H-7: PLAXIS 2D Model 3 with Drains Horizontal Displacement

Total displacements $u_x$

Maximum value = 0.3664 ft (Element 980 at Node 6157)

Minimum value = -0.3663 ft (Element 490 at Node 2979)
Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 2)

Minimum value = -2.557 ft (Element 580 at Node 4508)

Figure H-8: PLAXIS 2D Model 3 with Drains Vertical Displacement
Figure H-9: PLAXIS 2D Model 4 with Drains Horizontal Displacement

Total displacements $u_x$

Maximum value = 0.3462 ft (Element 718 at Node 6095)
Minimum value = -0.3462 ft (Element 417 at Node 3044)
Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 2)
Minimum value = -2.487 ft (Element 580 at Node 4508)

Figure H-10: PLAXIS 2D Model 4 with Drains Vertical Displacement
Total displacements $u_x$

Maximum value = 0.4158 ft (Element 980 at Node 6048)
Minimum value = -0.4158 ft (Element 507 at Node 2982)

Figure H-11: PLAXIS 2D Model 5 with Drains Horizontal Displacement
Figure H-12: PLAXIS 2D Model 5 with Drains Vertical Displacement

Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 2)

Minimum value = -2.618 ft (Element 580 at Node 4508)
Total displacements $u_x$

Maximum value $= 0.4158$ ft (Element 980 at Node 6048)
Minimum value $= -0.4158$ ft (Element 507 at Node 2982)

Figure H-13: PLAXIS 2D Model 6 with Drains Horizontal Displacement
Total displacements $u_y$

Maximum value = 0.000 ft (Element 1 at Node 2)
Minimum value = -2.618 ft (Element 580 at Node 4508)

Figure H-14: PLAXIS 2D Model 6 with Drains Vertical Displacement
Figure I-1: Segment 1 Phase 2 DMT Profiles for I_D, K_D and E_D
Figure I-2: Phase 2 Segment 1 (Sta. 603 + 20, LT 5) DMT Readings and Material Index Classification
Figure 1-3: Phase 2 Segment 1 (Sta. 603 + 24, LT 5) DMT Readings and Material Index Classification
Figure I-4: Phase 2 Segment 1 (Sta. 603 + 25, LT 10) DMT Readings and Material Index Classification
Figure I-5: Phase 2 Segment 1 (Sta. 603 + 40, RT 5) DMT Readings and Material Index Classification
Figure I-6: Phase 2 Segment 4 (Sta. 609 + 00, RT 5) DMT Readings and Material Index Classification
DISSIP DEPTH = 8.54 m

U₀,equl = 29.88 kPa
Tflex = 26 min

Figure I-7: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -4.3 ft
**Newington-Dover**

**DISSIPATION TEST "DMTA"**

**DISSIP DEPTH = 10.67 m**

- $U_{o,equil} = 50.79$ kPa
- $T_{flex} = 38$ min

---

**Figure I-8: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -11.3 ft**
**Newington-Dover**

**Dissipation Test "DMTA"**

**Dissipation Depth** = 12.2 m

\[ U_{o,\text{equil}} = 65.73 \text{ kPa} \]
\[ T_{\text{flex}} = 58 \text{ min} \]

**Figure I-9:** After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -16.3 ft

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 2.0 \times 10^{-3} \text{ cm}^2/\text{sec} \]
DISSIP DEPTH = 13.72 m

$U_0,\text{equil} = 80.67 \text{ kPa}$

$T_{flex} = 150 \text{ min}$

Figure I-10: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -21.3 ft

$C_{h,OC} = \frac{7 \text{ cm}^2}{T_{flex}} = 8.0 \times 10^{-4} \text{ cm}^2/\text{sec}$
Dissipation Test "DMTA"

**Dissipation Depth** = 15.24 m

- $U_{0,\text{equil}} = 95.61$ kPa
- $T_{\text{flex}} = 130$ min

**Figure I-11:** After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -26.3 ft

$$C_{h,OC} = \frac{7 \, \text{cm}^2}{T_{\text{flex}}} = 9.2 \times 10^{-4} \, \text{cm}^2/\text{sec}$$
DISSIPATION TEST "DMTA"

DISSIP DEPTH = 16.77 m

$U_0, \text{equil} = 110.55 \text{ kPa}$

$T_{\text{flex}} = 120 \text{ min}$

$C_{h,OC} = \frac{7 \text{ cm}^2}{T_{\text{flex}}} = 9.4 \times 10^{-4} \text{ cm}^2/\text{sec}$

Figure I-12: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -31.3 ft
DISSIP DEPTH = 18.29 m

$U_0,\text{equil} = 125.48$ kPa

S-shape insufficiently defined to identify $T_{flex}$

Figure 1-13: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -36.3 ft
DISSIP DEPTH = 19.82 m

\[ U_{o,\text{equil}} = 140.42 \text{ kPa} \]
\[ T_{\text{flex}} = 110 \text{ min} \]

Figure I-14: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -41.3 ft
DISSIP DEPTH = 21.34 m

U₀, equil = 155.36 kPa
Tflex = 36 min

\[ C_{h,OC} = \frac{7 \text{ cm}^2}{Tflex} = 3.2 \times 10^{-3} \text{ cm}^2/\text{sec} \]

Figure 1-15: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -46.3 ft
Figure 1-16: After Construction Segment 1 (Sta. 603 + 40, RT 5) DMT Dissipation Test at Elevation -51.3 ft
APPENDIX J - PHASE 2 PIEZOCONE TESTING DATA AND RESULTS
NHDOT Geotechnical Test Embankment
Newington-Dover, NH

Figure J-1: Segment 1 Phase 2 Comparison
NHDOT Geotechnical Test Embankment
Newington-Dover, NH

Corrected Tip Resistance
Friction Sleeve Resistance
Pore Pressure

Figure J-2: Segment 2 Phase 2 Comparison
Figure J-3: Segment 3 Phase 2 Comparison
NHDOT Geotechnical Test Embankment
Newington-Dover, NH

Corrected Tip Resistance
Friction Sleeve Resistance
Pore Pressure

Segment 4
Phase 1 Q-B226 (Sta. 609 + 05, LT 10) CPTu Readings
Phase 1 Q-B227 (Sta. 609 + 09, CL) CPTu Readings
Phase 2 (Sta. 609 + 10, RT 3) CPTu Readings

Figure J-4: Segment 4 Phase 2 Comparison
Figure J-5: Segment 5 Phase 2 Comparison