Spring 2016

SETTLEMENT PREDICTIONS OF HIGHWAY EMBANKMENTS ON MARINE CLAY USING IN SITU GEOTECHNICAL TESTING

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SETTLEMENT PREDICTIONS OF HIGHWAY EMBANKMENTS ON MARINE CLAY USING IN SITU GEOTECHNICAL TESTING

BY

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B.S. Civil Engineering, Worcester Polytechnic Institute, 2014

THESIS

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In

Civil Engineering

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This thesis has been examined and approved in partial fulfillment of the requirements for the degree of Master of Science in Civil Engineering by:

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Original approval signatures are on file with the University of New Hampshire Graduate School.
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ABSTRACT

SETTLEMENT PREDICTIONS OF HIGHWAY EMBANKMENTS ON MARINE CLAY

USING IN SITU GEOTECHNICAL TESTING

By

Adam Coen

University of New Hampshire, May, 2016

In the spring of 2014, the University of New Hampshire was approached by the New Hampshire Department of Transportation to provide engineering services for future embankments in Dover, New Hampshire. The proposed embankments will be constructed over a compressible marine deposit that can lead to significant settlement and long-term deformations. For one embankment, prefabricated vertical drains will be installed to direct pore water out of the soil and accelerate the rate of consolidation. Several in situ testing methods were performed to characterize the clay, including: flat plate dilatometer, field vane shear and piezocone. In addition to in situ testing, one-dimensional consolidation testing of undisturbed clay was performed in the UNH laboratory. The data collected from the in situ and laboratory tests was used to determine site-specific material and engineering properties of the clay deposit for settlement calculations using the finite element software PLAXIS 2D. The results will be used for comparison to field measurements during and after embankment construction.
1 INTRODUCTION

The Spaulding Turnpike in southeastern New Hampshire is one of the most heavily trafficked highways in the state due to its location and link to other major highways such as I-95. It links the New Hampshire Seacoast region to Concord via US 4 as well as the Lakes Region and White Mountains via NH 16. Not only is the turnpike an important commuter route, but the highway is a major passage for freight to the region. Geometric insufficiencies such as closely spaced interchanges and narrow shoulder widths have contributed to capacity constraints of the highway, leading to congestion during commuting hours (NHDOT, 2009). It was approximated that in 1980, about 30,000 vehicles per day travelled on the Little Bay Bridges going from Dover to Newington, New Hampshire. By 2001 the number of vehicles per day increased to approximately 70,000. A traffic study conducted on the area concluded that the traffic volume on the Little Bay Bridges could increase up to 94,000 vehicles per day by 2025 (NHDOT, 2009).

In the spring of 2014, the University of New Hampshire (UNH) was approached by the New Hampshire Department of Transportation (NHDOT) to provide engineering services for future embankments located at the proposed relocation of Exit 6N NB Off-Ramp in Dover, New Hampshire and north of the Dover tolls at Soundwall 3 in Dover, New Hampshire as shown in Figure 1-1. The proposed embankments will be constructed over a compressible marine clay that is very prevalent in the New Hampshire Seacoast region. The highly compressible behavior of the marine clay deposit can lead to significant settlement and long term deformations, which is why the NHDOT has partnered with UNH to implement an in situ and laboratory testing at the
program at the two sites. This combination of testing will help with predictions of long-term settlements of the proposed embankments to be built on the marine clay deposit.

Figure 1-1: Aerial view of the Little Bay Bridges, Exit 6N Off-Ramp and the Dover Toll Plaza (Google Maps, 2016)
The purpose of this study is to predict differential settlement of the compressible marine clays under embankment loading. Using finite element analysis software PLAXIS 2D, models of the proposed embankments were analyzed and compared to results from the research of Santamaria (2015) for validation. These settlement predictions will be used as a baseline for the NHDOT prior to embankment construction, as well as useful data for future geotechnical engineering problems on compressible clays in the New Hampshire Seacoast region.

Chapter 2 includes a literature review of consolidation theory and wick drains, as well as case histories related to the engineering properties of the Presumpscot Formation, embankment settlement with wick drains, and settlement prediction with finite element software. Chapter 3 describes the methodology for the in situ and laboratory tests used to determine the engineering parameters of the marine clay deposit. Chapter 4 discusses the material properties that were found for the compressible marine clay deposits. Chapter 5 includes a detail of the subsurface conditions and settlement predictions using the data from Chapter 4. Chapter 6 summarizes the research and provides conclusions and recommendations.
2 BACKGROUND

This chapter discusses the behavior of compressible clays upon applied embankment loading. Different characteristics of the compressible soil can change the magnitude and rate in which the soil consolidates. Introducing additional drainage paths into the soil can accelerate consolidation by draining excess porewater out at a faster rate. This chapter presents a series of case histories that are relevant to this research. Two case histories look specifically at the characteristics of the Presumpscot Formation. One case history looks at the effect of artificial drainage paths in accelerating consolidation. The last case history looks at using finite element analysis for calculating embankment settlements.

2.1 Consolidation

Consolidation is a time dependent process by which a saturated soil changes in volume due to the dissipation of porewater pressure under loading. Upon initial loading, the porewater resists all of the applied loading. This results in an increase in excess porewater pressure, which matches the total stress applied to the soil skeleton. The total stress increase causes the soil particles to pack together, forcing the excess porewater pressure to dissipate from the soil. Once all of the excess porewater has drained the soil skeleton will resist all of the load, ending primary consolidation.

For a normally consolidated soil, the equation for settlement is:

\[ \delta_c = \frac{C_c}{1 + e_o} H \log \left( \frac{\sigma'_{vf}}{\sigma'_{v0}} \right) \]  \hspace{1cm} (1)
where $\delta_c$ is consolidation settlement, $C_c$ is the virgin compression index, $e_o$ is the initial void ratio, $H$ is the thickness of the compressible soil, $\sigma'_vf$ is the final effective vertical stress and $\sigma'_v0$ is the initial effective vertical stress.

For overconsolidated soils, the recompression index ($C_r$) and preconsolidation pressure ($\sigma'_p$), or maximum past pressure, must be taken into consideration. This results in two possible cases:

When $\sigma'_v0 < \sigma'_p$ : 

$$\delta_c = \frac{C_r}{1 + e_o} H \log \left( \frac{\sigma'_v0}{\sigma'_v0} \right)$$

(2)

When $\sigma'_v0 < \sigma'_p < \sigma'_vf$ : 

$$\delta_c = \frac{C_r}{1 + e_o} H \log \left( \frac{\sigma'_p}{\sigma'_v0} \right) + \frac{C_c}{1 + e_o} H \log \left( \frac{\sigma'_vf}{\sigma'_p} \right)$$

(3)

Soils also experience secondary compression, a creep behavior after the completion of primary consolidation. Due to the sustained loading of an embankment, the soil can continue to compress after the complete dissipation of excess porewater pressure. While secondary compression settlement may account for a small portion of the total settlement, it is important to take it into consideration for long-term deformations, especially in soft compressible soils.

Settlement from secondary compression ($\delta_s$) is then calculated using the following equation:

$$\delta_s = \frac{C_r}{1 + e_o} H \log \left( \frac{t_2}{t_1} \right)$$

(4)
The secondary compression index ($C_\alpha$) is calculated with the following equation:

$$C_\alpha = \frac{\Delta e}{\log\left(\frac{t_2}{t_1}\right)} \quad (5)$$

where $t_1$ is the amount of time to completion of primary consolidation, $t_2$ is the desired total time and $\Delta e$ is the change in void ratio from $t_1$ to $t_2$.

The secondary compression index ($C_\alpha$) is found through consolidation testing or empirical relationships. Because consolidation is greatly influenced by time, other key properties include the coefficient of consolidation ($c_v$) and the length of the longest drainage path ($H_{dr}$) for excess porewater pressure to drain. The magnitude and rate of consolidation varies with the degree in which excess porewater pressure dissipates from the soil, which in turn is directly related to the permeability of the soil, as shown in Equation 6:

$$c_v = \frac{k}{\gamma_w} \frac{1 + e_o}{a_v} \quad (6)$$

where $k$ is permeability or hydraulic conductivity, $e_o$ is the initial void ratio, $\gamma_w$ is the unit weight of water and $a_v$ is the coefficient of compressibility (change in void ratio per change in stress).

The coefficient of consolidation is expressed as a unit of area over time. Based on Terzaghi’s consolidation theory, $c_v$ is also directly related to distance and time in which water will drain from the compressible soil into a pervious layer, using the following equation:

$$c_v = \frac{T H^2_{dr}}{t} \quad (7)$$
where T is a time factor, t is the amount of time for a particular settlement, and \( H_{dr} \) is the length of the longest drainage path. The length of drainage is depended on the relative permeability of the materials which are underlying and overlying the compressible soil. In a doubly drained system, more permeable layers are underlying and overlying the compressible layer. In this case the distance for porewater to travel out of the compressible layer is one half of the compressible layer thickness. In a singly drained system an impervious layer would be on one side of the compressible layer, which would make the drainage length equal to the layer thickness. Based on this theory, the rate in which compressible soils consolidate, as expressed in Equation 7, is directly dependent on the square of the longest drainage path. Since the drainage length for a singly drained system is twice the drainage length for a doubly drained system, it will take longer for the soil to consolidate.

2.2 Prefabricated Vertical Drains

In an effort to accelerate the consolidation process, Prefabricated Vertical (PV) drains, or wick drains, have been introduced to facilitate and accelerate the flow of porewater out of loaded compressible soils by providing closely spaced artificial drainage paths. Without the use of PV drains the time for a layer of compressible soil to consolidate under embankment loading could take decades. Wick drains consist of corrugated polypropylene cores designed to handle large longitudinal flow capacity. The core is covered with a highly permeable geosynthetic filter sleeve that prevents fine soil particles from permeating through and clogging the core. Water in the compressible soil moves laterally into the wick drain and is then channeled out vertically through the corrugation, as shown in Figure 2-1 (US Wick Drain, 2016).
Using a crane or an excavator equipped with a boom, the PV drains are typically pushed all the way through soft compressible soil layers. PV drains are contained in a spool and fed through a mandrel mounted on the boom. The drain is held in place at the bottom of the mandrel by an anchor plate and is pushed or vibrated through the soil to the desired depth. The mandrel is then drawn up the boom, leaving the wick drain in place. The in-place drain is then cut from the spool at ground surface, completing installation (US Wick Drain, 2016).

2.3 Case Histories

A literature review of some case studies dealing with earthwork construction on compressible soils is presented in this section. The case studies allow for a better understanding of the analysis methods and performance of soft clay deposits under loading, with or without PV drains. The case studies in this chapter also discuss the properties of the Presumpscot clay, embankment instability, and settlement analysis using finite element analysis.
2.3.1 Case History 1: Presumpscot Clay Variability

The marine clay deposit which is the compressible clay discussed in this thesis is known as the Presumpscot Formation. The deposit extends along coastal areas of New Hampshire and Maine and has historically presented many challenges for geotechnical engineers. Morgan (1987) explains the two primary concerns in engineering regarding the Presumpscot Formation. The first problem is stability of the soft clay under embankment loading. The material underlying the embankment must have sufficient shearing resistance to support the added weight of the structure, otherwise failure can occur, resulting in costly damages or possible human casualties. In many instances, the embankment must be constructed in stages to allow the clay to consolidate and gain strength before the application of additional loading. The second problem to consider is excessive settlement of the clay from embankment loading. In addition, the Presumpscot Formation takes a considerable amount of time to fully consolidate (typically decades), which presents challenges in engineering design. Evaluating these two problems is the key to properly engineering an embankment on the Presumpscot Formation.

The Presumpscot Formation has been found to be highly variable in thickness and properties, even within a few feet laterally and with depth. One example presented by Morgan (1987) was during the construction of the Maine Turnpike to the Route 1 Connector in Biddeford, Maine. Beginning in 1983, aerial photographs and preliminary subsurface explorations indicated that the thickness of the clay deposit ranged from 20 feet to 17 feet within a distance of approximately 700 feet. Another subsurface exploration plan was conducted in 1985, which yielded significantly different results. It was determined that the original test borings were conducted on each side of a valley of Presumpscot clay. The thickest part of the valley was
measured at approximately 60 feet, which in turn prompted further subsurface explorations and a change in design. The final design incorporated wick drains and stage construction to limit differential settlement, along with toe fills and a longer bridge for added stability (Morgan, 1987).

It was mentioned that a 60 foot thick deposit of Presumpscot clay was larger than average. However, there have been documented cases of Presumpscot Formation extending to a thickness of over 120 feet (Morgan, 1987). As shown in Figure 2-2, a boring log from Portland, Maine shows the clay deposit ranging from elevation -12.6 to -143.6 (Morgan, 1987). The results from their vane shear testing and Atterberg Limits tests have comparable values to those found for the marine clay deposit at Soundwall 3 (See Chapter 4). Soft, sensitive deposits of the Presumpscot Formation with lower undrained shear strength have been found, as shown in Figure 2-3, from a boring log during a field exploration at Maine Mall Road in South Portland. The vane shear strength from this boring are more representative of the values of the marine clay at Exit 6N (see Chapter 4).

These cases demonstrate that a thorough subsurface exploration program of these coastal marine clay deposits is necessary in order to adequately characterize the site, due to the variability of the Presumpscot Formation. If additional borings had not been conducted, a stability failure or long-term excessive settlement could have occurred, resulting in significant damages or costly repairs and maintenance.
Figure 2-2: Boring log from a subsurface exploration near I-295 in Portland, Maine (Morgan, 1987)
2.3.2 Case History 2: Presumpscot Clay Compressibility

The second case history consists of the construction of a one story masonry structure on the Presumpscot Formation in Portland, Maine on Warren Avenue (Cole, 1987). The area of the building measured 180 feet by 150 feet, and included a truck access floor with anticipated loading of 200 lb/ft². The site stratigraphy consisted of 8 to 9 feet of firm silty sand, followed by approximately 35 feet of soft to medium gray silty clay and underlain by a thin layer of gray silty fine sand directly above bedrock (Cole, 1987).
Nine test borings were performed and Shelby tube samples were taken for laboratory testing. After analyzing the field and laboratory data, it was determined that the upper portion of the clay deposit was overconsolidated by 2 to 3 ksf, while the lower portion of the clay deposit appeared to be normally consolidated to slightly overconsolidated (Cole, 1987). The compression index ($C_c$) averaged 0.73 and ranged from 0.52 to 0.89 while the recompression index ($C_r$) averaged 0.04 and ranged from 0.03 to 0.058. The coefficient of consolidation ($c_v$) averaged 265 ft$^2$/year and ranged from 200 to 350 ft$^2$/year for overconsolidated clay during recompression, while for the normally consolidated clay the average was 100 ft$^2$/year from a range of 50 to 200 ft$^2$/year during recompression. The calculated $c_v$ of the normally consolidated clay during virgin compression ranged between 5 to 10 ft$^2$/year. Test results yielded moisture contents between 40 to 50 percent and undrained shear strength of 300 to 500 lb/ft$^2$. Table 2-1 is a tabulation of the results.

The results found at Exit 6N as part of this thesis yield similar values of compression index ($C_c$) and recompression index ($C_r$), while the compression index of the marine clay deposit at Soundwall 3 was determined to be lower. Atterberg limits values determined from the NHDOT are also similar to the values from Warren Avenue. The full results of consolidation properties and Atterberg limits can be found in Chapter 4.
Table 2-1: Laboratory data of Presumpscot clay in Portland, Maine for settlement predictions
(From Cole, 1987)

<table>
<thead>
<tr>
<th>Sample</th>
<th>Elev. (ft)</th>
<th>Depth (ft)</th>
<th>$w_n$ (%)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI</th>
<th>$C_r$</th>
<th>$C_c$</th>
<th>$c_v$ (ft$^2$/year)</th>
</tr>
</thead>
<tbody>
<tr>
<td>B-1, 1U</td>
<td>60</td>
<td>12</td>
<td>46.5</td>
<td>42.4</td>
<td>24.4</td>
<td>18.0</td>
<td>0.058</td>
<td>0.833</td>
<td>354</td>
</tr>
<tr>
<td>B-1, 3U</td>
<td>50</td>
<td>22</td>
<td>46.0</td>
<td>37.2</td>
<td>23.4</td>
<td>13.8</td>
<td>0.050</td>
<td>0.630</td>
<td>8.7, 203</td>
</tr>
<tr>
<td>B-2, 2U</td>
<td>58</td>
<td>12</td>
<td>43.5</td>
<td>44.6</td>
<td>22.7</td>
<td>21.9</td>
<td>0.037</td>
<td>0.520</td>
<td>213</td>
</tr>
<tr>
<td>B-2, 4U</td>
<td>43</td>
<td>27</td>
<td>45.3</td>
<td>36.2</td>
<td>22.0</td>
<td>14.2</td>
<td>0.035</td>
<td>0.625</td>
<td>5.0, 57</td>
</tr>
<tr>
<td>B-101, 2U</td>
<td>58</td>
<td>12</td>
<td>47.8</td>
<td>45.4</td>
<td>22.3</td>
<td>23.1</td>
<td>0.030</td>
<td>0.590</td>
<td>248</td>
</tr>
<tr>
<td>B-101, 4U</td>
<td>44</td>
<td>26</td>
<td>50.0</td>
<td>36.0</td>
<td>22.6</td>
<td>13.4</td>
<td>0.040</td>
<td>0.830</td>
<td>5.4, 49</td>
</tr>
<tr>
<td>B-103, 2U</td>
<td>60</td>
<td>12</td>
<td>47.8</td>
<td>44.3</td>
<td>24.8</td>
<td>19.5</td>
<td>0.031</td>
<td>0.890</td>
<td>248</td>
</tr>
<tr>
<td>B-103, 4U</td>
<td>50</td>
<td>22</td>
<td>49.7</td>
<td>40.6</td>
<td>23.7</td>
<td>16.9</td>
<td>0.051</td>
<td>0.833</td>
<td>5.0, 91</td>
</tr>
</tbody>
</table>

Design calculations predicted settlement at the center of the building at about 8.8 inches, and about 3.5 inches at the building corners. The solution that was used included a site preload and installation of wick drains. With approximately 5 feet of fill for a preload, long term settlement calculations resulted in estimates between 7.8 to 12 inches at the center and 3.9 to 7 inches at the corners of the fill (Cole, 1987). It was estimated that wick drains would allow for 55 to 65 percent of consolidation within 3 to 4 months after preloading. A coefficient of consolidation ($c_v$) of 25 ft$^2$/year was used for the design calculations.
The site was cleared and a 2 ft layer of granular soil was placed as a drainage blanket. The wick drains were installed from surface at 8 foot spacing. The equivalent diameter and spacing pattern of the drains was not specified in the report.

Four months after the placement of the preload, the measured settlements compared well with estimates from consolidation theory. The observed settlement at the center and corner of the preload was 8 inches and 4 inches respectively, compared to the estimates of 7.8 inches and 3.9 inches. The calculations estimated that the wick drains would account for 55 to 65 percent of consolidation in that time frame, but the data from the field measurements showed that the more than 90 percent of consolidation had occurred. The field calculated $c_v$ ranged between 75 to 130 ft$^2$/year, much faster than the estimated rate, suggesting that the wick drains effectively expedited consolidation. Pore pressure calculations were overall fairly close to the observed values from the pneumatic transducers. The calculated pore pressures were very close to the observations at shallow depths but at greater depth the measurements showed slower pore pressure dissipation than the anticipated calculated values.

Overall it was determined that wick drains and site preloading were successful in achieving rapid consolidation of the sensitive Presumpscot marine clay. Predicted settlements and total settlements were in close agreement and preloading and wick drains increased the rate of consolidation.
Case History 3: Wick Drain and Creep Effects

The Swedish Geotechnical Institute (SGI) studied the effects that wick drains and creep have in calculating the consolidation of an embankment in northwestern Poland using methods by Barron (1949) and Hansbo (1979, 1981). Wick drain installation in the field causes some degree of disturbance, or smearing, to the soil around the drain. The remolded smeared clay around the drain has a lower permeability than the undisturbed clay, which slows down the rate of consolidation. Calculations were made with and without taking into consideration the effect of smear on the permeability of the wick drains. After 840 days the calculated settlement without using the smear effect was 1.75 m (5.75 ft), while the calculated settlement with the effect of smear was 1.68 m (5.5 ft), accounting for a difference of 7.5 cm (3 in.). The observed values after 840 days in the field showed 1.78 m (5.8 ft) of settlement. The results suggest that the smear effect on permeability would calculate a longer time to reach full consolidation, as shown in Figure 2-4 (Wolski, 1988).
A comparison between the settlement of one embankment with wick drains and one embankment without wick drains showed nearly identical ultimate settlements. The settlements under the embankment with drains were marginally larger, but the applied load from the embankment was slightly larger than the applied load from the embankment without drains, which could have led to more settlement (Wolski, 1988). Based on Figure 2-5 the embankment with wick drains shows a faster rate of settlement than the embankment without drains. After 480 days the embankment with the wick drains settled approximately 1.26 m (4.1 ft), while the embankment without the wick drains settled approximately 1.13 m (3.7 ft), accounting for a difference of 13.8 cm (5.4 in.) during that time period.
The effect of creep on the compression of soils was taken into account for consolidation calculations. When compared to the measured settlements in the field, the calculations with and without creep show little to no difference in settlement values during the initial stages. The differences become more apparent with an increase in time, as the time-settlement curve of the in-field measurements began to converge with the curve including creep, as shown in Figure 2-6. Two magnetic markers were installed at different depths to track the settlement in two different soil layers. The results show that the measured values in the field compared well to the calculated values taking creep into effect, as shown in Figure 2-7. Additional magnetic markers were installed at different depths below the center of the embankment to track the settlement distribution with depth. It was determined that the measured values were in agreement to the calculated values that included creep effects, as shown in Figure 2-8 (Wolski, 1988).
Figure 2-6: Total settlement of the embankment comparing measured and calculated values (Wolski, 1988)

Figure 2-7: Settlement during stages 2 and 3 for a calcareous soil layer and a peaty soil layer under the embankment (Wolski, 1988)
2.3.4 Case History 4: Embankment Modeling in PLAXIS

The use of PV drains helped Bio Energy Luleå in Sweden to expand the area where they store their sawdust for fuel pellet production. The large vertical loads from the sawdust piles caused consolidation of the underlying soils and dissipation of pore pressures to the ground surface. The settlement of the piles caused the bottom layers of sawdust to become wet, wasting material and increasing costs to dry it out (Khan, 2012). The finite element software PLAXIS 2D was used to predict the settlements of the sawdust stacks at the site. The model geometry and site stratigraphy is shown in Figure 2-9 and Figure 2-10.
15-node triangular elements were used for the analysis. The Hardening Soil and Soft Soil constitutive models were used to simulate the behavior of these soils. The soils using the Hardening Soil model included the embankment, sand crust, fine sand and stiff sand. The Soft
Soil model was used for the soft, impermeable silty clay layers. Vertical drains were modeled in the impermeable layers, as shown in Figure 2-10. The drains were placed at 1.75 m (5.75 ft) center-to-center (Khan, 2012).

The calculation of the embankment construction and settlement was divided into seven stages:

1) Initial phase
2) Placement of sand layer
3) Vertical drain installation
4) Placement of first embankment stage
5) Consolidation
6) Placement of second embankment state
7) Minimum excess pore pressure

The calculations were performed with and without drains to compare settlement rate, as shown in Figure 2-11.
The trends show the changes from immediate settlement, to primary settlement and ending with some secondary settlement. The total settlement ended 60 days earlier when wick drains were incorporated in the analysis, proving the effectiveness of wick drains speeding up consolidation. The results from the analysis are summarized in Table 2-2. The incorporation of wick drains also reduced the excess pore pressure generated from the embankment loading by initiating dissipation during construction and accelerating dissipation during consolidation. These trends are shown in Figure 2-12. With wick drains the excess porewater pressure dissipated from the clay faster at each stage, whereas more excess porewater pressure was generated without the artificial drainage paths created with the drains, taking longer for consolidation to occur.

*Figure 2-11: Settlement versus time at the center of the embankment (Khan, 2012)*
Table 2-2: Comparison of PLAXIS results with and without wick drains (Khan, 2012)

<table>
<thead>
<tr>
<th>Items/Results</th>
<th>$u_{\text{Center of Embankment}}$</th>
<th>Total ($u$)</th>
<th>Time</th>
<th>Excess Pore Pressure</th>
</tr>
</thead>
<tbody>
<tr>
<td>With drains</td>
<td>24</td>
<td>39</td>
<td>252</td>
<td>0.1</td>
</tr>
<tr>
<td>Without drains</td>
<td>26</td>
<td>37</td>
<td>312</td>
<td>0.1</td>
</tr>
<tr>
<td>Units</td>
<td>cm</td>
<td>cm</td>
<td>days</td>
<td>kN/m$^2$</td>
</tr>
</tbody>
</table>

Figure 2-12: Excess porewater pressure over time at the center of the embankment (Khan, 2012)

This case history effectively details the process of creating a finite element model in PLAXIS for simulating consolidation with wick drains. It proves the effectiveness of wick drains in accelerating the rate of consolidation and dissipation of excess porewater pressure.
3 IN SITU AND LABORATORY TESTING

3.1 Introduction

A subsurface exploration program was conducted to determine the properties of the underlying marine clay deposit at the Soundwall 3 and Exit 6N sites to be used in the settlement evaluation of each embankment. At Soundwall 3, two field vane profiles were performed, with tests conducted at two-foot intervals within the marine clay deposit. Four flat plate dilatometer profiles were conducted at one-foot intervals until refusal. One piezocone profile was conducted with continuous pushing until refusal. At the Exit 6N NB Off-Ramp, two field vane profiles, twelve flat plate dilatometer profiles, and three piezocone soundings were conducted at the same test intervals aforementioned for Soundwall 3. In addition to in-situ tests performed at the sites, Shelby tube piston samples of undisturbed marine clay were taken from both sites for one-dimensional consolidation testing and index properties testing.

3.2 Test Summary

Aerial views of the two test sites are shown in Figure 3-1 and Figure 3-2.
Figure 3-1: Aerial view of Exit 6N Off-ramp (Google Maps, 2016)

Figure 3-2: Aerial view of Soundwall 3 (Google Maps, 2016)
A summary of the testing and sampling at both sites is shown in Table 3-1. Surface elevations italicized and in bold are estimated based on nearby tests, since elevations were not given for those tests. Details of each test method are described in this chapter. Figure 3-3 shows the NHDOT drill rig set up in preparation for a test.

Table 3-1: Summary of in situ tests and sampling

<table>
<thead>
<tr>
<th>Date</th>
<th>Site</th>
<th>Station</th>
<th>Surface Elevation (ft)</th>
<th>End Elevation (ft)</th>
<th>Test Method</th>
</tr>
</thead>
<tbody>
<tr>
<td>8/5/15</td>
<td>SW3</td>
<td>1687+50 LT. 90</td>
<td>18.83</td>
<td>-9.17</td>
<td>DMT</td>
</tr>
<tr>
<td>8/6/15</td>
<td>SW3</td>
<td>1688+00 LT. 90</td>
<td>19.29</td>
<td>-5.21</td>
<td>DMT</td>
</tr>
<tr>
<td>8/7/15</td>
<td>SW3</td>
<td>1688+00 LT. 135</td>
<td>17.06</td>
<td>-7.94</td>
<td>DMT</td>
</tr>
<tr>
<td>8/7/15-8/10/15</td>
<td>SW3</td>
<td>1687+50 LT. 132</td>
<td>17.76</td>
<td>-12.24</td>
<td>DMT</td>
</tr>
<tr>
<td>8/12/15-8/13/15</td>
<td>SW3</td>
<td>1687+63 LT. 135</td>
<td>17.76</td>
<td>1.76</td>
<td>Shelby Sampling</td>
</tr>
<tr>
<td>8/14/15-8/18/15</td>
<td>SW3</td>
<td>1687+90 LT. 135</td>
<td>16.93</td>
<td>3.26</td>
<td>FVT</td>
</tr>
<tr>
<td>8/20/15</td>
<td>SW3</td>
<td>1687+96 LT. 135</td>
<td><strong>17.06</strong></td>
<td>-3.28</td>
<td>CPTu</td>
</tr>
<tr>
<td>8/25/15-8/27/15</td>
<td>Exit 6N</td>
<td>310+00 RT. 40</td>
<td>13.52</td>
<td>-31.98</td>
<td>DMT</td>
</tr>
<tr>
<td>8/28/15</td>
<td>Exit 6N</td>
<td>309+00 RT. 30</td>
<td>13.04</td>
<td>-34.46</td>
<td>DMT</td>
</tr>
<tr>
<td>8/31/15</td>
<td>Exit 6N</td>
<td>308+00 RT. 30</td>
<td>11.34</td>
<td>-39.06</td>
<td>DMT</td>
</tr>
<tr>
<td>9/9/15</td>
<td>Exit 6N</td>
<td>307+00 RT. 30</td>
<td>10.86</td>
<td>-38.84</td>
<td>DMT</td>
</tr>
<tr>
<td>9/10/15</td>
<td>Exit 6N</td>
<td>307+00 LT. 30</td>
<td>11.81</td>
<td>-37.49</td>
<td>DMT</td>
</tr>
<tr>
<td>9/11/15</td>
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<td>308+00 LT. 19</td>
<td>11.79</td>
<td>-33.61</td>
<td>DMT</td>
</tr>
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<td>-43.67</td>
<td>CPTu</td>
</tr>
<tr>
<td>9/17/15</td>
<td>Exit 6N</td>
<td>308+06 LT. 91</td>
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<td>-39.58</td>
<td>CPTu</td>
</tr>
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<td>9/18/15</td>
<td>Exit 6N</td>
<td>308+10 RT. 30</td>
<td>10.70</td>
<td>-36.45</td>
<td>CPTu</td>
</tr>
<tr>
<td>9/22/15-9/25/15</td>
<td>Exit 6N</td>
<td>308+05 RT. 30</td>
<td><strong>11.02</strong></td>
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<td>Shelby Sampling</td>
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<tr>
<td>11/9/15</td>
<td>Exit 6N</td>
<td>308+10 LT. 98</td>
<td>17.34</td>
<td>-33.66</td>
<td>DMT</td>
</tr>
<tr>
<td>11/12/15-11/13/15</td>
<td>Exit 6N</td>
<td>309+00 LT. 75</td>
<td>19.03</td>
<td>-29.47</td>
<td>DMT</td>
</tr>
<tr>
<td>11/16/15</td>
<td>Exit 6N</td>
<td>308+96 LT. 33</td>
<td>14.93</td>
<td>-33.07</td>
<td>DMT</td>
</tr>
<tr>
<td>11/17/15</td>
<td>Exit 6N</td>
<td>310+00 LT. 61</td>
<td>21.77</td>
<td>-22.23</td>
<td>DMT</td>
</tr>
<tr>
<td>11/18/15-11/19/15</td>
<td>Exit 6N</td>
<td>311+00 LT. 30</td>
<td>24.67</td>
<td>-21.33</td>
<td>DMT</td>
</tr>
<tr>
<td>11/19/15</td>
<td>Exit 6N</td>
<td>311+00 RT. 40</td>
<td>13.52</td>
<td>-16.66</td>
<td>DMT</td>
</tr>
</tbody>
</table>
Figure 3-3: NHDOT drill rig positioned for DMT testing at Soundwall 3
3.3 Flat Plate Dilatometer Testing

The flat plate dilatometer was originally developed by Dr. Silvano Marchetti of Italy to evaluate different characteristics of soils like strength and deformation parameters, soil behavior, soil stratigraphy, and stress history (Marchetti, 2001). Empirical correlations were also developed to estimate various other material properties for cohesive and cohesionless soils. The dilatometer probe consists of a stainless steel blade with an 18° wedge tip. One side of the blade includes an expandable steel membrane of 2.54 in. (60 mm) in diameter. Cross-sectional dimensions of the blade measure at about 3.74 in. (95 mm) in width and 0.59 in. (15 mm) in thickness.

The procedure for dilatometer testing follows ASTM D6635 Standard Test Method for Performing the Flat Plate Dilatometer. The blade is connected to a control unit via a pneumatic-electrical cable that is pre-strung through the push rods. Nitrogen gas is connected to the control unit to provide pressure to expand the steel membrane. A regulator is attached to the gas tank to control feed pressure to the control unit. The control unit is also equipped with an audio-visual signal to alert the operator when readings should be taken. The control unit is shown in Figure 3-4. The DMT test consists of expanding the steel membrane into the soil at specific test intervals. During the test a series of pressures are recorded as A, B and C readings. Those readings need to be corrected for membrane stiffness and zero offsets on the pressure gauges.
The membrane calibration is done in air before the blade is advanced into the ground and recorded as $\Delta A$ and $\Delta B$. $\Delta A$ is determined by applying a vacuum to the membrane, resulting in an inward deflection. This simulates the external pressure required to seat the membrane to the A-position. $\Delta B$ is determined by applying pressure to the membrane until it is expanded 0.04 in. (1.1 mm) from the initial position. Membrane calibrations are typically performed by pulling and pushing the piston of a syringe connected to the control unit (to determine $\Delta A$ and $\Delta B$ respectively). The operator must also measure the low end and high end gauge offsets ($Z_M$).

Figure 3-5 shows a schematic of the layout during membrane calibration.
After membrane calibrations have been conducted, testing may begin. The dilatometer blade is pushed from the surface using a drill rig. Tests are conducted at specified intervals, with test depths being measured from the center of the membrane. Once the desired test depth is reached, the rig operator stops pushing and releases the vertical loading on the blade. For this project a test interval of 1.0 ft (0.30 m) was implemented. At each test interval the dilatometer operator records A, B and C pressure readings. The A pressure represents the amount of soil stress acting on the membrane prior to expansion, the B pressure represents the amount of soil stress acting on the membrane expanded 1.1 mm, and the C pressure represents the estimated porewater pressure as the membrane returns to the body of the probe after a controlled deflation.

To measure the A reading, the dilatometer operator opens the flow valve to pressurize the membrane until it has returned to its original position. During this time, the signal will turn off, prompting the operator to record the value. To measure the B reading, the operator continues the flow to the membrane until the membrane has moved 1.1 mm from the original seating.
During this time the signal will reactivate, prompting the operator to record the value. It is important that once the signal has reactivated, the operator must close the flow valve and partially open the vent to prevent the membrane from becoming over-expanded.

The C reading is measured by slowly venting the membrane until it returns to its original seating. The audio signal will be on after the B reading is recorded. The signal will turn off while venting, then reactivate when the membrane is at its original seating, thus prompting the operator to record the C-reading. Figure 3-6 and Figure 3-7 show the working principle and layout of the dilatometer test.

![Working Principle of Dilatometer](image)

*Figure 3-6: Working principle of dilatometer (Marchetti et al., 2001)*
3.3.1 Field Data Reduction

Marchetti’s SDMT Elab software was used for analyzing the data recorded during field testing. The user inputs the A, B, and C readings with depth into the software which are then corrected for membrane stiffness and low pressure gauge zero offset as shown in the following equations (Marchetti et al., 2001).

\[
p_0 = 1.05(A - Z_M + \Delta A) - 0.05(B - Z_M - \Delta B) \tag{8}
\]

\[
p_1 = B - Z_M - \Delta B \tag{9}
\]

\[
p_2 = C - Z_M + \Delta A \tag{10}
\]
p₀, p₁, and p₂ are the corrected first, second, and third readings respectively. The corrected pressure readings are used to determine the intermediate indices described by Marchetti. The three intermediate parameters are as follows: material index (I₀), horizontal stress index (K₀), and dilatometer modulus (E₀).

The material index, I₀, is used to evaluate soil type based on the mechanical behavior of the material. Grain size distribution is not considered in expressing I₀, so a very rigid material could be expressed as silt rather than clay (Marchetti et al., 2001).

\[ I₀ = \frac{(p₁ - p₀)}{(p₀ - u₀)} \]  \hspace{1cm} (11)

The horizontal stress index, K₀, is used as a preliminary source to determine multiple parameters from the dilatometer test, such as: coefficient of lateral earth pressure at rest (K₀), undrained shear strength (sᵤ), constrained modulus (M) and overconsolidation ratio (OCR) (Marchetti et al., 2001):

\[ K₀ = \frac{p₀ - u₀}{σ'₀₀} \]  \hspace{1cm} (12)

where u₀ is porewater pressure and σ'₀₀ is the effective overburden stress.

The dilatometer modulus, E₀, is based upon the theory of elasticity, but is not suggested to be used as a primary parameter in analysis. This is due to the fact that the modulus formula does not include stress history (Marchetti et al., 2001).
\[ E_D = 34.7(p_1 - p_0) \]  

Marchetti suggests presenting the data with constrained modulus, \( M_D \), instead. The constrained modulus is a corrected form of the dilatometer modulus, and is calculated as follows:

\[ M_{DMT} = R_M E_D \]  

where \( R_M \) is a correction factor dependent on the material index \( (I_D) \) and horizontal stress index \( (K_D) \). \( R_M \) is determined by the following equations:

\[ R_M = 0.14 + 2.36\log K_D \]  

\[ R_M = R_{M,O} + (2.50 - R_{M,O})\log K_D, \text{ if } I_D < 0.6 \]  

\[ R_{M,O} = 0.14 + 0.15(I_D - 0.6), \text{ if } 0.6 < I_D < 3.0 \]  

\[ R_M = 0.50 + 2.00\log K_D, \text{ if } I_D \geq 3.0 \]  

\[ R_M = 0.32 + 2.18\log K_D, \text{ if } K_D > 10 \]

A typical profile from the DMT is shown in Figure 3-8. The data is plotted as suggested by Marchetti in an effort to standardize data presentation. These results are discussed in detail in Chapter 4.
3.4 Field Vane Testing

The field vane is a commonly used instrument in geotechnical investigations for determining the undrained shear strength of saturated clays. The procedure for field vane testing follows ASTM D2573 Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils. The test involves pushing a four-bladed vane into the soil and rotating it from the surface. The
vane is rotated until the soil is sheared to a given torque. The measured torque is then correlated to the shearing resistance on the failure plane of the soil being tested.

The instrument used for this study was a Geonor H-10 Vane Borer. A vane blade with dimensions 65 mm diameter and 130 mm height was chosen, complying with the ASTM specification of having a height to diameter ratio between 1 and 2.5. While vanes are available in varying dimensions and configurations, the operator must choose the proper vane that has a maximum torque capacity larger than the expected torque of the soil being tested. Figure 3-9 shows the components of the Geonor H-10 Vane Borer assembly, while Figure 3-10 shows the complete assembly in the field.
Before the test can begin, the housing with the vane retracted is advanced into the ground to a specified depth. The vane is then pushed out of the housing approximately 20 in. (50 cm), where the test depth is recorded. The operator begins the test and applies the torque to the vane. To ensure that the outer rod does not rotate during the test, it is highly recommended.
that the rod be firmly held in place with pipe wrenches or a clamp as shown in Figure 3-10. Permissible rate of rotation of the inner rods from ASTM ranges from 0.05 to 0.2 deg/s. A rotation rate of 0.1 deg/s was used for these tests. Readings were recorded every 15 seconds until the soil failed in shear. After failure the vane was rotated 10 full revolutions, and another test was performed until a maximum value was achieved. This second test determines the remolded strength of the clay, which is necessary to determine the sensitivity of the material. Clay sensitivity is the ratio of undisturbed undrained shear strength to remolded undrained shear strength.

3.4.1 Field Data Reduction

The primary reason for performing a field vane test is to determine the undrained shear strength of saturated clays. For a rectangular vane with height to diameter ratio of 2 (as was used for this testing program) and assuming a uniform stress distribution along the vertical edges and the top and bottom of the vane blades, the following equation determines the undrained shear strength:

\[ S_u = \frac{6T_{\text{max}}}{7\pi D^3} \]  \hfill (20)

where D is the diameter of the vane, and \( T_{\text{max}} \) is the maximum torque value.

The equation for sensitivity is as follows:

\[ S_t = \frac{S_{u,\text{peak}}}{S_{u,\text{remolded}}} \]  \hfill (21)
Figure 3-11 shows a typical vane shear test giving the undrained shear strength versus angular rotation for an undisturbed and remolded test. From this test, the undrained shear strength is measured at 277 psf and the remolded strength at 18 psf, giving a sensitivity of 15.4.

*NH DOT Exit 6N Embankment*
*Dover, NH*

*University of New Hampshire*
*Geonor H-10 Vane Borer Testing*

*Figure 3-11: Shear stress versus angular rotation for a test at Sta. 307+90 RT. 30*
3.5 Piezocone Testing

Piezocone (CPTu) testing is being increasingly used in geotechnical site investigations. The piezocone is a probe that can be pushed in various soil types and can be used to estimate several geotechnical parameters. The piezocone test procedure follows ASTM D5778 Standard Test Method for Electronic Piezocone Penetration Testing. The typical piezocone is a cylindrical device with a conical tip. The tip has a 60° apex with a diameter of 35.7 mm (1.41 in.) leading to a projected area of 10 cm². For this testing program a Vertek 10 Ton Cone Penetrometer was used.

Prior to testing, the cone cable is strung though the push rods and connected to the control unit. The fluid cavity and porous element of the cone must be saturated before the start of the test to ensure good pore pressure response. The porous elements used for testing were pre-saturated with glycerin prior to field investigations, while the cone was saturated in the field. A funnel was placed around the cone and de-aired water was poured into the funnel. At this point, the cone tip is unthreaded from the shaft, and a syringe filled with water is used to remove air bubbles from the tip. Once this is achieved, the porous element and tip are threaded back onto the shaft. The tip is covered with a prophylactic membrane in order to preserve saturation before being pushed into the ground.

After initial baseline readings are recorded the cone is pushed into the ground at a steady rate of 2 cm/s (0.8 in./s). Four channels were monitored for testing: tip resistance, sleeve resistance, pore water pressure and inclination.
3.5.1 Field Data Reduction

After a profile is completed, measurements of cone resistance, sleeve resistance and pore water pressure are uploaded into Vertek’s software and GeoLogismiki’s CPeT-IT software for data reduction. Cone resistance, \( q_c \), is calculated from the following equation:

\[
q_c = \frac{Q_c}{A_c}
\]  

(22)

where \( Q_c \) is the pushing force on the cone, and \( A_c \) is the cone base area. The cone used for this project has a cone base area of 10 cm\(^2\), as stated previously. Cone resistance is measured as force per unit area, i.e. MPa, ton/ft\(^2\). Cone resistance should be corrected using the area ratio as per Jamiolkowski et al. (1985). The corrected cone resistance, \( q_t \), is then calculated using the following equation:

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

(23)

where \( u_2 \) is porewater pressure measured behind the cone tip, and \( a_n \) is the net area ratio. For this cone the net area ratio was evaluated as 0.8.

Friction sleeve resistance, \( f_s \), is calculated from the following equation:

\[
f_s = \frac{Q_s}{A_s}
\]  

(24)
where $Q_s$ is the force on the sleeve, and $A_s$ is the area of the sleeve. For a cone with a base area of 10 cm$^2$, the friction sleeve area is typically 150 cm$^2$. Friction sleeve resistance is measured as force per unit area (i.e. kPa, ton/ft$^2$).

Figure 3-12 presents a typical CPTu profile in terms of corrected tip resistance, friction sleeve resistance and pore pressure versus depth. Analysis of these profiles is presented in Chapter 4.

![Figure 3-12: Typical CPTu Profile at Exit 6N](image)

3.6 Piston Sampling

During the in situ testing program, undisturbed samples of Presumpscot marine clay were obtained from both Soundwall 3 and Exit 6N for one-dimensional consolidation tests at the UNH soils laboratory. Samples were obtained using a piston sampler and thin-walled Shelby
tubes. Refer to ASTM D1587 Standard Practice for Thin-Walled Sampling of Fine-Grained Soils for Geotechnical Purposes for a complete description of the procedure.

3.7 One-Dimensional Consolidation Testing

The purpose for laboratory consolidation testing was to determine the magnitude and rate of consolidation of the Presumpscot marine clay. The UNH laboratory has two Geocomp LoadTrac-III systems with ICONP software to perform and record the consolidation test. The sample preparation and test procedure for one-dimensional consolidation testing followed ASTM D2435 One-Dimensional Consolidation Properties of Soils Using Incremental Loading.

Samples of undisturbed, fully saturated soil from Shelby tube sampling were used for testing. A pipe cutter was used to cut a sample of approximately 3 in. in length from a section of the tube. It should be noted that in order to use soil of least disturbance, it is recommended to not use the soil from the tube within 1-1.5 times the tube diameter from the ends (DeGroot and Ladd, 2005). The sample is then extruded from the cut section of the tube with a hydraulic jack, and trimmed carefully with a wire saw.

The sample of clay is then turned over onto a glass plate, and a greased metal consolidation ring is carefully pushed onto the sample. The ring should be pushed in the same direction as the clay entered the sampling tube, or else the shear stresses on the sides of the sample can reverse, yielding inaccurate results. A wire saw is used to trim away excess soil around the edges, top and bottom of the consolidation ring to fit the clay at a uniform volume to the ring. The excess trimmings are used for determining the natural water content of the sample prior to testing.
A saturated porous stone is placed in the consolidometer with a piece of filter paper on top. The clay specimen in the consolidation ring is placed on top of the stone and paper, while another piece of filter paper and a porous stone are placed on top of the sample. A load plate with a ball are then placed on top of the porous stone, completing the assembly. The consolidometer is placed on the platen of the LoadTrac, with an LVDT positioned on top of the consolidometer. The platen on the LoadTrac is adjusted until the load cell is about to make contact with the ball. The complete set-up of a consolidation test is shown in Figure 3-14. Refer to ASTM D2435 One-Dimensional Consolidation Properties of Soils Using Incremental Loading, the Geocomp user manual and Getchell (2013) for further instruction on sample preparation. A typical consolidation curve is shown in Figure 3-13.
Figure 3-13: Typical consolidation curve of void ratio versus applied stress
Figure 3-14: Set up of a consolidation test using Geocomp LoadTrac-III
4 INTERPRETATION OF TEST RESULTS

All of the data collected from *in situ* testing and laboratory testing was analyzed and reduced to determine the geotechnical properties of the subsurface materials at the Soundwall 3 and Exit 6N sites. Each test method yielded various parameters that were compared to each other to determine correlations specific to each site. This data was then compared to results from previous research findings of Ladd (1972), Findlay (1991) and Getchell (2013) from tests conducted in marine clay deposits in the New Hampshire seacoast region. When data correlation and repeatability were confirmed, results were analyzed to determine the behavior of the marine clay deposit that aided settlement predictions using finite element analysis.

4.1 Consolidation Testing

Many factors associated with drilling, sampling and specimen preparation of sensitive clay can cause sample disturbance, which can adversely affect consolidation test results. DeGroot et al. (2005) outlined steps involved in producing disturbance throughout the process from drilling to specimen preparation, as shown by Ladd and DeGroot (2003) in Figure 4-1. The steps can be described as follows:
Figure 4-1: Stress path of low OCR clay during sampling and specimen preparation (Ladd and DeGroot, 2003)

Path 1-2: Drilling. Open boreholes reduce the total vertical stress ($\sigma_v$) in the soil, which can lead to stress relief at the bottom of the boring. It is suggested to use a weighted drilling mud to decrease the magnitude of stress relief. The recommended drilling mud weight should range from 1.2 to 1.3 times the unit weight of water.

Path 2-3-4-5: Sampling. Past research has shown the effect of tube sampling on sample disturbance. According to Baligh et al. (1987), the soil along the centerline of the tube experiences compression ahead of the tube (Point 2 to Point 3), but then will experience shear extension upon entering the tube (Point 3 to Point 4). Once the material has fully entered the tube, it will then experience compression (Point 4 to Point 5). The changes in shear stress induce straining of the clay, which could then lead to destructuring and positive pore pressures.
It is recommended to use a 3 in. (76 mm) diameter Shelby tube with a 5 to 10° cutting edge and an inside clearance ratio (ICR) of approximately zero. The inside clearance ratio (ICR) is the difference of the interior diameter of the cutting edge to the interior diameter of the sampling tube, as expressed in the following equation:

\[
ICR = \frac{D_t - D_c}{D_c} \times 100\%
\]  

(25)

where \(D_t\) is the interior diameter of the sampling tube and \(D_c\) is the interior diameter of the cutting edge.

For this investigation Shelby tube samples remained in the ground for 20 minutes after pushing and before extraction to allow dissipation of some of the excess porewater pressure from pushing and for the soil to adhere to the inside of the tube.

Path 5-6: Tube Extraction. When extracting the sample, the suction at the bottom of the tube creates resistance, which can lead to additional disturbance. The soil at the top of the sample typically consists of disturbed material present at the bottom of the borehole prior to pushing. It is recommended to use a fixed piston sampler rather than a free piston sampler, to reduce the amount of debris from entering the tube. A study was done on samples of Boston Blue Clay comparing the behavior of using a free piston with no drilling mud to using a fixed piston with weighted drilling mud. Figure 4-2 shows the results from constant rate of strain (CRS) consolidation tests on the two samples. The trends show a significant difference in the preconsolidation stress and compressibility of the samples. For the investigations at Soundwall 3 and Exit 6N a free piston without weighted drilling mud was used.
Figure 4-2: Differences in preconsolidation pressure of a fixed piston sampler and free piston sampler (DeGroot et al., 2005)

Path 6-7: Transportation and Storage. Prior to transportation, the ends of the tubes should be sealed with wax and a plastic cap. It is suggested to use a 50:50 mixture of paraffin wax and petroleum jelly. Samples collected at Exit 6N and Soundwall 3 were sealed with paraffin wax only. Vibrations during transportation can also lead to significant sample disturbance. Samples must be restrained during transport to minimize movements. It is suggested to keep the samples upright in a box, lined or filled with a damping material like foam padding or wood chips.
Path 7-8: Sample Extraction. Over time the clay begins to bond to the interior of the tube. The ensuing extraction of the material from the tube can lead to disturbance.

Path 8-9: Specimen Preparation. Stress relief from trimming can lead to additional disturbance. The process of sampling material and preparing it for laboratory testing should be conducted with consideration to all of the potential instances of disturbance as described.

Sample disturbances can have a great effect on the determination of preconsolidation pressure ($\sigma'_p$) from consolidation testing. Determination of preconsolidation pressure is needed to calculate the overconsolidation ratio (OCR) and the field corrected virgin compression index ($C_c$). DeGroot et al. (2005) studied the effects of sample disturbance for determining preconsolidation pressure. Lunne et al. (1997) developed a sample quality method that takes into consideration the ratio of change in void ratio to initial void ratio ($\Delta e/e_o$) during reconsolidation to the existing effective overburden stress ($\sigma'_vo$). The criteria for the designation from Lunne et al. (1997) is shown in Table 4-1.

<table>
<thead>
<tr>
<th>$\Delta e/e_o$ (Lunne et al.)</th>
<th>OCR = 1-2</th>
<th>OCR = 2-4</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\Delta e/e_o$</td>
<td>$\Delta e/e_o$</td>
<td></td>
</tr>
<tr>
<td>&lt;0.04</td>
<td>&lt;0.03</td>
<td></td>
<td>Very Good to Excellent</td>
</tr>
<tr>
<td>0.04-0.07</td>
<td>0.03-0.05</td>
<td></td>
<td>Good to Fair</td>
</tr>
<tr>
<td>0.07-0.14</td>
<td>0.05-0.10</td>
<td></td>
<td>Poor</td>
</tr>
<tr>
<td>&gt;0.14</td>
<td>&gt;0.10</td>
<td></td>
<td>Very Poor</td>
</tr>
</tbody>
</table>

The preconsolidation pressure for samples with a rating of Poor to Very Poor would be considered inaccurate, thus rendering unreliable OCR results. Based on the results from laboratory consolidation tests of samples from Exit 6N, two samples had a rating of Poor,
shown in Table 4-2. The two samples with the worst disturbance rating were taken from near the bottom of the marine deposit where the clay begins to transition to the glacial outwash. The OCR values for these samples could have been much higher in reality. The trends from the *in situ* tests and the other consolidation tests were evaluated for choosing the OCR for the PLAXIS models. Please refer to Appendix D for the consolidation curves for this research.

**Table 4-2: Sample quality of consolidation test samples of marine clay taken from Exit 6N**

<table>
<thead>
<tr>
<th>Sample</th>
<th>Depth (ft)</th>
<th>$\Delta e/e_o$</th>
<th>Rating</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>15.17</td>
<td>0.036</td>
<td>Very Good to Excellent</td>
</tr>
<tr>
<td>2</td>
<td>22.67</td>
<td>0.055</td>
<td>Good to Fair</td>
</tr>
<tr>
<td>3</td>
<td>30.58</td>
<td>0.083</td>
<td>Poor</td>
</tr>
<tr>
<td>7</td>
<td>8.50</td>
<td>0.015</td>
<td>Very Good to Excellent</td>
</tr>
<tr>
<td>8</td>
<td>19.00</td>
<td>0.058</td>
<td>Good to Fair</td>
</tr>
<tr>
<td>9</td>
<td>26.67</td>
<td>0.061</td>
<td>Good to Fair</td>
</tr>
<tr>
<td>10</td>
<td>34.67</td>
<td>0.128</td>
<td>Poor</td>
</tr>
</tbody>
</table>

4.2 Atterberg Limits

Atterberg Limits testing was performed by the NHDOT for the marine clay deposit at both sites. A summary of average values of liquid limit (LL), plastic limit (PL), plasticity index (PI) and natural water content ($w_n$) from each test are shown in Table 4-3 and Table 4-4 for Exit 6N and Soundwall 3, respectively.
Table 4-3: Index properties of the marine clay at Exit 6N

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>w_n (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.50</td>
<td>35.5</td>
<td>22.3</td>
<td>13.2</td>
<td>36.5</td>
</tr>
<tr>
<td>10.75</td>
<td>37.1</td>
<td>22.0</td>
<td>15.1</td>
<td>46.2</td>
</tr>
<tr>
<td>15.17</td>
<td>34.5</td>
<td>20.9</td>
<td>13.6</td>
<td>42.9</td>
</tr>
<tr>
<td>16.50</td>
<td>36.0</td>
<td>21.3</td>
<td>14.7</td>
<td>46.2</td>
</tr>
<tr>
<td>18.75</td>
<td>39.1</td>
<td>22.2</td>
<td>16.9</td>
<td>46.6</td>
</tr>
<tr>
<td>20.25</td>
<td>36.3</td>
<td>21.4</td>
<td>14.9</td>
<td>43.7</td>
</tr>
<tr>
<td>22.75</td>
<td>34.5</td>
<td>21.7</td>
<td>12.8</td>
<td>38.3</td>
</tr>
<tr>
<td>24.33</td>
<td>36.0</td>
<td>23.2</td>
<td>12.8</td>
<td>38.3</td>
</tr>
<tr>
<td>26.33</td>
<td>34.8</td>
<td>21.1</td>
<td>13.7</td>
<td>36.4</td>
</tr>
<tr>
<td>30.75</td>
<td>27.2</td>
<td>19.6</td>
<td>7.6</td>
<td>28.7</td>
</tr>
<tr>
<td>32.33</td>
<td>27.2</td>
<td>18.8</td>
<td>8.4</td>
<td>29.4</td>
</tr>
</tbody>
</table>

Table 4-4: Index properties of the marine clay at Soundwall 3

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>LL (%)</th>
<th>PL (%)</th>
<th>PI (%)</th>
<th>w_n (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>5.00</td>
<td>40.6</td>
<td>21.5</td>
<td>19.1</td>
<td>21.3</td>
</tr>
<tr>
<td>13.00</td>
<td>31.4</td>
<td>19.6</td>
<td>11.8</td>
<td>23.9</td>
</tr>
</tbody>
</table>

These values were compared to results from the research on the marine clay in Portsmouth and Dover, New Hampshire, as summarized in Table 4-5. Compared to Ladd (1972), Findlay (1991) and Getchell (2013), the Atterberg limits values from Exit 6N were comparable, while the natural water content appears to be slightly smaller. Ladd (1972) found the natural water content of the marine clay in Portsmouth, New Hampshire to be at 50 ± 5%, while the natural water content of the marine clay at Exit 6N was determined to be at 39 ± 6%. For the marine clay at Soundwall 3, the Atterberg limits were comparable to past research, while the natural water content was determined to be lower than the other values at 23 ± 1%. Figure 4-3 displays the graphical results from Atterberg limits testing.
Table 4-5: Summary of marine deposit index properties of clay in Portsmouth, NH and Dover, NH

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>LL</td>
<td>35 ± 5%</td>
<td>34 ± 3.2%</td>
<td>36 ± 4%</td>
<td>34 ± 4%</td>
<td>36 ± 5%</td>
</tr>
<tr>
<td>PL</td>
<td>20 ± 1%</td>
<td>21 ± 1.9%</td>
<td>23 ± 2%</td>
<td>21 ± 1%</td>
<td>21 ± 1%</td>
</tr>
<tr>
<td>PI</td>
<td>15 ± 3%</td>
<td>13 ± 1.3%</td>
<td>13 ± 3%</td>
<td>13 ± 3%</td>
<td>15 ± 4%</td>
</tr>
<tr>
<td>w_n</td>
<td>50 ± 5%</td>
<td>42 ± 6.7%</td>
<td>41 ± 5%</td>
<td>39 ± 6%</td>
<td>23 ± 1%</td>
</tr>
</tbody>
</table>

The results of Ladd (1972) and Findlay (1991) are based on tests performed on marine clay in Portsmouth, NH and summarized by Getchell (2013). The results from Getchell (2013) and at Exit 6N and Soundwall 3 are based on both the overconsolidated and normally consolidated marine clays. As evidenced in Figure 4-3, the index properties of the marine clay do not vary much between the upper marine clay deposit and the lower marine clay deposit. The liquid limit decreases the most below depth 30 ft where the clay starts to transition to the underlying glacial till and outwash layers. In general, the natural water content is greater than the liquid limit which is typical of sensitive clays.
Figure 4-3: NHDOT Atterberg limits results
4.3 Total Unit Weight

The total unit weight ($\gamma_T$) of the marine clay deposit was determined through one dimensional consolidation testing of relatively undisturbed clay samples obtained from Shelby tube sampling. Total unit weight was also estimated empirically from CPTu and DMT data. The total unit weight determined from the laboratory was used as the baseline, since it could be directly measured. CPTu and DMT results were then shifted to develop a site-specific correlation, using Equations 26 and 27 for the clay from Exit 6N. The corrections were only made to the marine clay deposit, since the correlation could yield results too large for the existing alluvium and glacial layers. Since the site from Getchell’s (2013) research is less than one mile from the Exit 6N site, the same corrections were applied for consistency to the particular region. Figure 4-4 depicts the total unit weight with depth at Exit 6N for original estimates and shifted correlations.

\begin{equation}
\gamma_{T\ Exit\ 6N\ DMT} = \gamma_{T\ DMT} + 10 \text{ pcf}
\end{equation}

\begin{equation}
\gamma_{T\ Exit\ 6N\ CPTu} = \gamma_{T\ CPTu} + 20 \text{ pcf}
\end{equation}
Figure 4-4: Original and shifted total unit weight found at Exit 6N
While the site-specific correlations were determined from data collected from one boring, it is assumed that these parameters are consistent throughout all free-field conditions at the site. The site-specific correlations match those proposed by Getchell (2013). The total unit weight of the marine clay deposit was found to range between 106.8 to 118.5 pcf from consolidation testing. This is comparable to data from Getchell (2013), where the total unit weight of the marine clay deposit less than one mile away ranged from 107 to 120 pcf at the Newington-Dover Test Embankment.

For data collected at Soundwall 3, shifts were applied to the data from the DMT and CPTu to match the results from consolidation testing. For the DMT data, the total unit weight was shifted by 10 pcf for the first 5 feet of soil, as shown in Equation 28. For the rest of the profile the total unit weight was shifted by 3 pcf, as shown in Equation 29. The CPTu data was corrected by shifting the data by 5 pcf, as shown in Equation 30.

For depth 0 ft to 5 ft

\[ \gamma_{T\ Soundwall\ 3\ DMT} = \gamma_{T\ DMT} + 10\ pcf \]  
\[ (28) \]

For depth 5 ft to refusal

\[ \gamma_{T\ Soundwall\ 3\ DMT} = \gamma_{T\ DMT} + 3\ pcf \]  
\[ (29) \]

\[ \gamma_{T\ Soundwall\ 3\ CPTu} = \gamma_{T\ Soundwall\ 3} + 5\ pcf \]  
\[ (30) \]
Figure 4-5 depicts total unit weight with depth at Soundwall 3 for original estimates and shifted correlations. Based on consolidation test data, the total unit weight of the marine clay deposit was found to range between 119 and 131 pcf.

Figure 4-5: Original and shifted total unit weight found at Soundwall 3
4.4 Undrained Shear Strength

The undrained shear strength \((s_u)\) was determined through \textit{in situ} testing results. While the field vane test can directly measure undrained shear strength, DMT and CPTu tests empirically derive undrained shear strength based on test results of various clays studied throughout the world. This means that empirical relationships from the DMT and the CPTu tests may not be blindly applicable to the Presumpscot clay without development of a site-specific correlation.

While field vane tests can measure undrained shear strength directly, two corrections were applied to the free field conditions that take into consideration the plasticity of the material and the overconsolidation ratio. Studies by Chandler (1988) recommend that undrained shear strength must be corrected prior to stability analysis of embankments constructed upon soft material. For clays and silts with a plasticity index (PI) greater than 5\%, the following equations are recommended:

\[
S_u = \mu_R S_{u\text{uncorrected}} \tag{31}
\]

\[
\mu_R = 1.05 - b(PI)^{0.5} \tag{32}
\]

\[
b = 0.015 + 0.0075 \log(t_f) \tag{33}
\]

where \(b\) is a time rate factor and \(t_f\) is the time to failure in minutes (Chandler, 1988).

Values of plasticity index were determined based on Atterberg Limits data. For test depths in which Atterberg limits data were not available, values were estimated based on linearized trends. While the corrections from Chandler (1988) only take plasticity index into consideration, a correction proposed by Aas et al. (1986) incorporates the plasticity index and
overconsolidation ratio of the material. Undrained shear strength is corrected with a correction factor, using the same expression as Equation 31. Figure 4-6 shows the diagrams from Aas et al. (1986) used to determine the correction factor.

Figure 4-6: Charts to determine field vane correction factor based on plasticity index and stress history (Aas et al., 1986)
The undrained shear strength from the DMT and CPTu were calculated using Equations 34 and 35 respectively:

\[ S_{u,\text{DMT}} = 0.22\sigma'_v(0.5K_D)^{1.25}, \text{for } I_D < 1.2 \]  
(34)

\[ S_{u,\text{CPTu}} = \frac{q_t - \sigma_v}{N_{kt}} \]  
(35)

where \( N_{kt} \) is a cone factor correction. Figure 4-7 shows one plot of undrained shear strength values without unit weight corrections or cone factor \((N_{kt})\) corrections, and one plot of undrained strength values based on shifted unit weight and updated cone factor. With corrected total unit weight values, the DMT data was further corrected using a coefficient based on the work of Roche, Rabasca and Benoît (2008) and confirmed by Getchell (2013). From Equation 34, the coefficient of 0.22 was replaced by 0.13. This shows a better trend in relation to the baseline data from field vane testing. The correction is shown in Equation 36.

\[ S_{u,\text{Exit}6N_{DMT}} = 0.13\sigma'_v(0.5K_D)^{1.25}, \text{for } I_D < 1.2 \]  
(36)

The cone factor is determined using the liquid limit. The software used for the CPTu data reduction, CPeT-IT, assumes \( N_{kt} \) to be 14 throughout. Corrections were made using liquid limit data from Atterberg Limits testing performed by the NHDOT. The cone factor is calculated as follows:

\[ N_{kt} = 13.4 + 6.65(LL) \]  
(37)
Values of $N_{kt}$ based on Atterberg Limits testing can be found in Table 4-6.

**Table 4-6: Updated cone factor based on NHDOT Atterberg limits results**

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>$N_{kt}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>8.50</td>
<td>15.8</td>
</tr>
<tr>
<td>10.75</td>
<td>15.9</td>
</tr>
<tr>
<td>15.17</td>
<td>15.7</td>
</tr>
<tr>
<td>16.50</td>
<td>15.8</td>
</tr>
<tr>
<td>18.75</td>
<td>16.0</td>
</tr>
<tr>
<td>20.25</td>
<td>15.8</td>
</tr>
<tr>
<td>22.75</td>
<td>15.7</td>
</tr>
<tr>
<td>24.33</td>
<td>15.8</td>
</tr>
<tr>
<td>26.33</td>
<td>15.7</td>
</tr>
<tr>
<td>30.75</td>
<td>15.2</td>
</tr>
<tr>
<td>32.33</td>
<td>15.2</td>
</tr>
</tbody>
</table>

The uncorrected undisturbed shear strength of the marine clay at Exit 6N is determined to range between 268 and 768 psf. For this elevation range the values of undrained shear strength are slightly higher than those observed by Getchell (2013), which ranged from 200 to 600 psf.
Figure 4-7: Updated and shifted undrained shear strength of the marine clay at Exit 6N

At Soundwall 3, the deposit was too stiff to develop a full profile of vane shear tests. The trend of undrained shear strength with depth is significantly different than what is estimated using the DMT and the CPTu tests. For the two successful vane shear tests, the uncorrected undisturbed shear strength of the marine clay at Soundwall 3 was determined to be 794 and 1162 psf. The results are plotted in Figure 4-8.
Figure 4-8: Undrained shear strength of the marine clay at Soundwall 3
4.5 Consolidation Properties

4.5.1 Overconsolidation Ratio (OCR)

Values of overconsolidation ratio (OCR) were determined from the results of laboratory consolidation testing and estimated from dilatometer testing and piezocone testing. Getchell (2013) used a site-specific correction of OCR for DMT and CPTu results, shown in Equations 38 and 39. The corrections were used on only the marine clay and not the alluvium or glacial till layers.

\[
OCR_{\text{Dover(DMT)}} = OCR_{\text{DMT}} - 1.5
\]  
\[
OCR_{\text{Dover(CPTu)}} = OCR_{\text{CPTu}} - 0.5
\]  

The data shifts determined by Getchell (2013) were found to work for the DMT and CPTu data found at Exit 6N. Figure 4-9 shows OCR values from Exit 6N. Values of OCR were also analyzed from test results from Soundwall 3. The OCR of the upper deposit was found to be 7 from consolidation testing, and decreases down to 1.25 to the lower deposit. Values of OCR of the lower marine clay were found to vary between 0.9 and 2.2. It should be noted that for the last two consolidation tests the OCR should be higher than the value observed because of sample disturbance. Due to the stiffer behavior of the marine clay deposit at Soundwall 3, the DMT and CPTu tests greatly over-estimated the values.
Figure 4-9: Overconsolidation ratio (OCR) of the marine clay at Exit 6N
The observed OCR from DMT and CPTu data for Soundwall 3 appeared to be overestimated in comparison to results from consolidation testing. Consolidation test results show a range of OCR from 3.9 to 9.2. For the DMT data the original expression for determining OCR by Marchetti (1980) is as follows:

\[
OCR_{DMT} = (0.5 K_D)^{1.56}
\]  

(40)

Since material parameters from the DMT were derived empirically, a site-specific relationship was applied to the data, expressed as:

\[
OCR_{Soundwall 3 \, DMT} = (0.27 K_D)^{1.25}
\]  

(41)

This site-specific shift is based upon the baseline values from consolidation tests and shows a better indication of the OCR of the marine clay deposit.

There have been multiple relationships studied to determine OCR from CPTu data. For interpreting CPTu data in fine-grained soil, Lunne et al. (1997) suggest using the normalized cone resistance \(Q_t\) and using a coefficient, \(k\), to determine OCR:

\[
OCR_{CPTu} = k \left( \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \right)
\]  

(42)

where \(\left( \frac{q_t - \sigma_{vo}}{\sigma'_{vo}} \right) = Q_t\), and \(k\) ranges from 0.2 to 0.5. Using updated overburden stresses from the shifted unit weight, and \(k\) of 0.20, the OCR from the CPTu data was shifted. Figure 4-10 shows updated OCR with depth for Soundwall 3. All of the tests show an increase in OCR of the upper
layer of clay, followed by a decrease with depth. The updated DMT data converges to near normally consolidated, while the CPTu data still greatly overestimated OCR. The CPTu data was updated again with the coefficient $k$ equal to 0.12. Despite being outside the Lunne et al.’s suggested range for the $k$ coefficient, the CPTu data fits the data from the other tests better with that $k$ value, as shown in Figure 4-10.

Figure 4-10: Overconsolidation ratio (OCR) of the marine clay at Soundwall 3
4.5.2  Compression Index ($C_c$) and Recompression Index ($C_r$)

The compression index ($C_c$) and recompression index ($C_r$) are obtained from laboratory consolidation testing and are used for predicting primary consolidation. For samples taken at Exit 6N the compression index ranged from 0.17 to 0.48, while the recompression index ranged from 0.03 to 0.08. For samples taken from Soundwall 3 the compression index ranged from 0.14 to 0.19, while the recompression index ranged from 0.04 to 0.06. Getchell (2013) found a $C_c$ to range between 0.15 and 0.37 and a $C_r$ to range between 0.03 and 0.07. Findlay (1991) found a $C_c$ to range between 0.10 and 0.59 and a $C_r$ to range between 0.01 and 0.08. The data from Exit 6N and Soundwall 3 show similar results found from Getchell (2013) and Findlay (1991). In many instances swelling index ($C_s$) is used interchangeably with recompression index, as is the case with PLAXIS.

*Table 4-7: Summary of material properties and compression indices of marine clay at the Dover Test Embankment (Getchell, 2013)*

<table>
<thead>
<tr>
<th>Depth</th>
<th>$γ_T$ (pcf)</th>
<th>$C_c$</th>
<th>$C_r$</th>
<th>$e_o$</th>
</tr>
</thead>
<tbody>
<tr>
<td>11.7</td>
<td>112.2</td>
<td>0.374</td>
<td>0.063</td>
<td>1.01</td>
</tr>
<tr>
<td>11.8</td>
<td>117.7</td>
<td>0.301</td>
<td>0.060</td>
<td>0.71</td>
</tr>
<tr>
<td>15.4</td>
<td>108.9</td>
<td>0.302</td>
<td>0.047</td>
<td>0.79</td>
</tr>
<tr>
<td>18.0</td>
<td>112.4</td>
<td>0.281</td>
<td>0.045</td>
<td>0.81</td>
</tr>
<tr>
<td>20.6</td>
<td>113.8</td>
<td>0.329</td>
<td>0.042</td>
<td>0.71</td>
</tr>
<tr>
<td>26.8</td>
<td>107.3</td>
<td>0.309</td>
<td>0.057</td>
<td>0.87</td>
</tr>
<tr>
<td>30.4</td>
<td>108.1</td>
<td>0.344</td>
<td>0.065</td>
<td>0.88</td>
</tr>
<tr>
<td>30.9</td>
<td>107.8</td>
<td>0.352</td>
<td>0.075</td>
<td>1.01</td>
</tr>
<tr>
<td>35.9</td>
<td>111.3</td>
<td>0.301</td>
<td>0.048</td>
<td>0.78</td>
</tr>
<tr>
<td>40.4</td>
<td>110.1</td>
<td>0.336</td>
<td>0.053</td>
<td>0.93</td>
</tr>
<tr>
<td>45.3</td>
<td>110.4</td>
<td>0.243</td>
<td>0.048</td>
<td>0.80</td>
</tr>
<tr>
<td>50.4</td>
<td>112.6</td>
<td>0.336</td>
<td>0.058</td>
<td>0.79</td>
</tr>
<tr>
<td>55.4</td>
<td>117.7</td>
<td>0.257</td>
<td>0.043</td>
<td>0.63</td>
</tr>
<tr>
<td>60.5</td>
<td>119.9</td>
<td>0.213</td>
<td>0.047</td>
<td>0.58</td>
</tr>
<tr>
<td>65.8</td>
<td>120.3</td>
<td>0.174</td>
<td>0.035</td>
<td>0.52</td>
</tr>
</tbody>
</table>
Due to sample disturbances during laboratory test preparation and *in situ* sampling, Schmertmann (1955) developed a graphical procedure to determine the field value of the compression index of the material being tested. Corrected compression indices using the method by Schmertmann are shown in Table 4-8. Since Schmertmann corrections typically yield a larger value of compression index, larger calculated settlement is expected.

*Table 4-8: Schmertmann corrected virgin compression indices*

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Site</th>
<th>(C_c)</th>
<th>(C_c) (Schmertmann Corrected)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Exit 6N</td>
<td>0.48</td>
<td>0.54</td>
</tr>
<tr>
<td>2</td>
<td>Exit 6N</td>
<td>0.34</td>
<td>0.42</td>
</tr>
<tr>
<td>3</td>
<td>Exit 6N</td>
<td>0.21</td>
<td>0.29</td>
</tr>
<tr>
<td>4</td>
<td>SW 3</td>
<td>0.14</td>
<td>0.15</td>
</tr>
<tr>
<td>5</td>
<td>SW 3</td>
<td>0.19</td>
<td>0.26</td>
</tr>
<tr>
<td>6</td>
<td>SW 3</td>
<td>0.17</td>
<td>0.20</td>
</tr>
<tr>
<td>7</td>
<td>Exit 6N</td>
<td>0.34</td>
<td>0.40</td>
</tr>
<tr>
<td>8</td>
<td>Exit 6N</td>
<td>0.57</td>
<td>0.79</td>
</tr>
<tr>
<td>9</td>
<td>Exit 6N</td>
<td>0.27</td>
<td>0.29</td>
</tr>
<tr>
<td>10</td>
<td>Exit 6N</td>
<td>0.17</td>
<td>0.26</td>
</tr>
</tbody>
</table>

After applying Schmertmann corrections, \(C_c\) ranged from 0.26 to 0.79 and 0.15 to 0.26 for Exit 6N and Soundwall 3, respectively. Values of compression indices for clay at Exit 6N and Soundwall 3 are shown in Figure 4-11, with the results from Exit 6N compared to values from Findlay (1991). The Schmertmann corrections fell within the range of values found by Findlay (1991), except for one point at depth 19 feet, where the corrected value is 0.79.
Figure 4-11: Consolidation parameters of the marine clay at Exit 6N compared to Findlay (1991) and of the marine clay at Soundwall 3.
4.5.3 Secondary Compression Index ($C_\alpha$)

Total settlement analysis takes into consideration both primary consolidation and secondary consolidation. The secondary compression index ($C_\alpha$) is the strain or change in void ratio per log cycle of time after 100% primary consolidation has occurred. This is typically determined by plotting void ratio or strain versus time on a semi-log graph. The secondary compression index can also be calculated using the method proposed by Professor Mesri, which assumes that the ratio of the secondary compression index to the virgin compression index remains constant. Table 4-9 summarizes typical values of this ratio based on the work of Terzaghi et al. (1996).

*Table 4-9: Values of $C_\alpha/C_c$ for geotechnical materials based on Terzaghi et al., 1996 (Holtz et al., 2011)*

<table>
<thead>
<tr>
<th>Material</th>
<th>$C_\alpha/C_c$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Granular soils including rockfill</td>
<td>0.02 ± 0.01</td>
</tr>
<tr>
<td>Shale and mudstone</td>
<td>0.03 ± 0.01</td>
</tr>
<tr>
<td>Inorganic clays and silts</td>
<td>0.04 ± 0.01</td>
</tr>
<tr>
<td>Organic clays and silts</td>
<td>0.05 ± 0.01</td>
</tr>
<tr>
<td>Peat and muskeg</td>
<td>0.06 ± 0.01</td>
</tr>
</tbody>
</table>

The secondary compression index for the marine clay at the Exit 6N and Soundwall 3 sites was calculated using Mesri’s method with a $C_\alpha/C_c$ ratio of 0.05. Table 4-10 is a summary of the calculated values of $C_\alpha$ for the marine clay at the Exit 6N and Soundwall 3 sites.
Table 4-10: Values of secondary compression index using Schmertmann corrected virgin compression index

<table>
<thead>
<tr>
<th>Test Number</th>
<th>Site</th>
<th>$C_c$ (Schmertmann Corrected)</th>
<th>$C_\alpha$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Exit 6N</td>
<td>0.54</td>
<td>0.03</td>
</tr>
<tr>
<td>2</td>
<td>Exit 6N</td>
<td>0.42</td>
<td>0.02</td>
</tr>
<tr>
<td>3</td>
<td>Exit 6N</td>
<td>0.29</td>
<td>0.01</td>
</tr>
<tr>
<td>4</td>
<td>SW 3</td>
<td>0.15</td>
<td>0.01</td>
</tr>
<tr>
<td>5</td>
<td>SW 3</td>
<td>0.26</td>
<td>0.01</td>
</tr>
<tr>
<td>6</td>
<td>SW 3</td>
<td>0.20</td>
<td>0.01</td>
</tr>
<tr>
<td>7</td>
<td>Exit 6N</td>
<td>0.40</td>
<td>0.02</td>
</tr>
<tr>
<td>8</td>
<td>Exit 6N</td>
<td>0.79</td>
<td>0.04</td>
</tr>
<tr>
<td>9</td>
<td>Exit 6N</td>
<td>0.29</td>
<td>0.01</td>
</tr>
<tr>
<td>10</td>
<td>Exit 6N</td>
<td>0.26</td>
<td>0.01</td>
</tr>
</tbody>
</table>

Calculated $C_\alpha$ values for the marine clay at Exit 6N ranged between 0.013 and 0.040, while $C_\alpha$ for the marine clay at Soundwall 3 ranged between 0.007 and 0.013. Mesri (1973) developed an empirical relationship of the modified secondary compression index ($C_{ae}$) to the natural water content of the soil, where:

$$C_{ae} = \frac{C_\alpha}{1 + e_0} \quad (43)$$

The relationships of various clay types around the world are shown in Figure 4-12, and the summary of calculated values is shown in Table 4-11.
Figure 4-12: Modified secondary compression index versus natural water content based on Mesri, 1973 (Holtz et al., 2011)
Table 4-11: Summary of the modified secondary compression index based on Mesri (1973)

<table>
<thead>
<tr>
<th>Test</th>
<th>Site</th>
<th>e_o</th>
<th>C_α</th>
<th>C_αε</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Exit 6N</td>
<td>1.28</td>
<td>0.0271</td>
<td>0.0119</td>
</tr>
<tr>
<td>2</td>
<td>Exit 6N</td>
<td>1.29</td>
<td>0.0211</td>
<td>0.0092</td>
</tr>
<tr>
<td>3</td>
<td>Exit 6N</td>
<td>0.96</td>
<td>0.0144</td>
<td>0.0073</td>
</tr>
<tr>
<td>4</td>
<td>SW 3</td>
<td>0.70</td>
<td>0.0074</td>
<td>0.0043</td>
</tr>
<tr>
<td>5</td>
<td>SW 3</td>
<td>0.80</td>
<td>0.0132</td>
<td>0.0073</td>
</tr>
<tr>
<td>6</td>
<td>SW 3</td>
<td>0.83</td>
<td>0.0102</td>
<td>0.0056</td>
</tr>
<tr>
<td>7</td>
<td>Exit 6N</td>
<td>1.17</td>
<td>0.0202</td>
<td>0.0093</td>
</tr>
<tr>
<td>8</td>
<td>Exit 6N</td>
<td>1.43</td>
<td>0.0397</td>
<td>0.0163</td>
</tr>
<tr>
<td>9</td>
<td>Exit 6N</td>
<td>1.04</td>
<td>0.0144</td>
<td>0.0071</td>
</tr>
<tr>
<td>10</td>
<td>Exit 6N</td>
<td>0.86</td>
<td>0.0130</td>
<td>0.0070</td>
</tr>
</tbody>
</table>

C_αε for the marine clay at Exit 6N ranges from 0.0070 to 0.0163 with an average value of 0.0097, while C_αε for the marine clay at Soundwall 3 ranges from 0.0043 to 0.0073 with an average value of 0.0057. As a comparison, for Boston blue clay with a natural water content between 30% and 50%, C_αε ranges between 0.002 and 0.004, which is lower than the ranges calculated for the marine clays at the Exit 6N and Soundwall sites. The calculated average value of C_αε for the marine clay at Exit 6N, with an average natural water content of 39.4%, is more similar to the Norwegian plastic clay where C_αε ranges between 0.010 and 0.011. The calculated average value of C_αε for the marine clay at Soundwall 3, with an average natural water content of 22.6%, is more similar to the Chicago blue clay where C_αε ranges between 0.0015 and 0.0050.
4.5.4 Initial Void Ratio ($e_o$)

Initial void ratio ($e_o$) was determined from laboratory consolidation testing. For samples taken from Exit 6N the initial void ratio ranged from 0.86 and 1.43. Values found for samples at Exit 6N are plotted in Figure 4-13. As shown in the figure, the values of initial void ratio are comparable to the data from Findlay (1991), where the initial void ratio was found to range from 0.70 and 1.50. The data also matches the results from Getchell (2013) at the Dover Test Embankment, where the initial void ratio was determined to range between 0.87 and 1.31. For samples taken from Soundwall 3, the initial void ratio ranged from 0.70 to 0.83, increasing with depth. This trend is observed in Figure 4-13. The initial void ratio of marine clay at Soundwall 3 was expected to be lower, based on the stiffer behavior in relation to the clay at Exit 6N. For settlement predictions, initial void ratio values of 0.80 and 0.50 were used for the alluvium and glacial outwash/till respectively based on the findings of Santamaria (2015).
Figure 4-13: Initial void ratio of the marine clay at Exit 6N compared to Findlay (1991) and of the marine clay at Soundwall 3.
4.5.5 Coefficient of Consolidation ($c_v$)

The vertical coefficient of consolidation ($c_v$) was found during virgin compression from laboratory consolidation testing. The values of $c_v$ at Exit 6N ranged from 0.02 to 0.44 ft$^2$/day, much lower than the values of $7.7 \pm 7.6$ ft$^2$/day from Findlay (1991), but are similar to the values calculated by Getchell (2013) at the Dover Test Embankment (0.10 to 0.39 ft$^2$/day). The values of $c_v$ at Soundwall 3 ranged from 0.49 to 1.88 ft$^2$/day, which fall within the range from Findlay (1991).

*Figure 4-14: Coefficient of consolidation of the marine clay at Exit 6N and Soundwall 3 compared to Findlay (1991)*
4.6 Hydraulic Conductivity (k)

Hydraulic conductivity (also referred to as permeability) is the ability of a fluid to flow through porous media over a unit length of time under a unit of hydraulic gradient. The term hydraulic conductivity is typically used in reference to the ability of water to flow through soil or rock. Permeability is one of the most widely varying properties of a soil, because of the dependence on numerous soil and fluid properties. Properties that affect permeability include, but are not limited to, void ratio, soil density, degree of saturation, fluid density and fluid viscosity. The permeability of clays will determine the rate in which excess porewater pressure will dissipate during compression, thus directly affecting the rate of consolidation. Figure 4-15 shows the permeability determined through *in situ* and laboratory testing at Exit 6N and Soundwall 3. Getchell (2013) found an average permeability of 1.50E-04 ft/day for the near normally consolidated clay at the Dover Test Embankment.

The permeability for the CPTu tests was calculated from the CPeT-IT software. The software uses a relationship to the soil behavior index type (\(I_c\)), which is defined with the following equation:

\[
I_c = \sqrt{(3.47 - \log Q_T)^2 + (\log F_r + 1.22)^2}
\]  \hspace{1cm} (44)

where \(Q_{tn}\) is the normalized cone penetration resistance and \(F_r\) is the normalized friction ratio.

Both \(Q_{tn}\) and \(F_r\) are calculated with the following equations:

\[
Q_{tn} = \left(\frac{q_t - \sigma_v}{p_a}\right) \left(\frac{p_a}{\sigma_{vo}}\right)^n
\]  \hspace{1cm} (45)
\[ F_r = \frac{f_s}{q_t - \sigma_{vo}} \times 100\% \quad (46) \]

where:

\( q_t \) = CPTu corrected total cone resistance  
\( f_s \) = CPTu sleeve friction  
\( \sigma_{vo} \) = pre-insertion in-situ total vertical stress  
\( \sigma'_{vo} \) = pre-insertion in-situ effective vertical stress  
\( \rho_a \) = atmospheric pressure  
\( n \) = stress exponent that varies with soil behavior type

The relationship between permeability and \( I_c \) is represented with the following equations:

When \( 1.0 < I_c \leq 3.27 \)

\[ k = 10^{(0.952 - 3.04I_c)} \text{ m/s} \quad (47) \]

When \( 3.27 < I_c < 4.0 \)

\[ k = 10^{(-4.52 - 1.37I_c)} \text{ m/s} \quad (48) \]
Figure 4-15: Permeability found at Exit 6N using CPTu and consolidation tests
4.7 Coefficient of Lateral Earth Pressure at Rest ($K_0$)

The coefficient of lateral earth pressure at rest ($K_0$) is the ratio of effective horizontal stress to effective vertical stress. This coefficient can be empirically derived from DMT and CPTu data or calculated from measured values of horizontal stress from the self-boring pressuremeter. Values of $K_0$ from in situ testing at Exit 6N are shown in Figure 4-16.

Figure 4-16: Coefficient of lateral earth pressure at rest for DMT and CPTu Soundings at Exit 6N
As expected, the values of $K_0$ derived from DMT data are higher than values from CPTu data, and have been confirmed by Santamaria (2015) and Getchell (2013). For finite element analysis $K_0$ from CPTu tests were chosen for conservative calculations. For the near normally consolidated marine clay at Exit 6N the average value of $K_0$ from the CPTu was 0.70. For the near normally consolidated marine clay at the Dover Test Embankment, Santamaria (2015) calculated $K_0$ equal to 0.70 averaged from DMT and CPTu data. If the DMT and CPTu values are averaged for the clay at Exit 6N, $K_0$ would then be equal to 0.90. One case using $K_0$ equal to 0.90 was performed to see the effect of total settlement $K_0$ has on the marine clay. This is discussed later in Chapter 5. For the overconsolidated clay at Exit 6N and the all of the clay deposit at Soundwall 3, $K_0$ was set to 1.0 for the finite element analysis.
5 SETTLEMENT ANALYSIS

5.1 Introduction

The settlement of the proposed embankments at the Soundwall 3 and the Exit 6N sites was analyzed from deformation analysis using PLAXIS 2D AE. PLAXIS was used to analyze vertical and horizontal deformations, and distribution of pore pressures at each station where in situ and laboratory testing was performed as part of this project. This chapter includes the site stratigraphy and material properties used for analysis, as well as the methods to create the models.

5.2 Site Stratigraphy

In order to develop a finite element analysis model in PLAXIS, a soil stratigraphy specific to the site being analyzed must be established in details. Figure 5-1 and Figure 5-2 show the comparison of site stratigraphy at each station of Exit 6N and Soundwall 3 based on the subsurface exploration program. The blue layer represents alluvium, the pink layer represents the upper overconsolidated marine clay, the green layer represents the lower near normally consolidated marine clay and the yellow layer represents the glacial till and outwash. The site representation is simplified based on available data from the various test soundings and borings and linear approximations between borings in PLAXIS. As shown for stations 308+00, 309+00, 310+00 and 311+00 the height of the uppermost layer (assumed to be Alluvium) increases due to the current embankment at the site as part of the existing Exit 6N. While the geometry of
the embankment and layer thickness vary at each segment, each PLAXIS model had a general soil stratigraphy based on \textit{in situ} test data.

\textbf{Figure 5-1: Comparison of site stratigraphy for stations at Exit 6N}
Figure 5-2: Comparison of site stratigraphy for stations at Soundwall 3

A consistent thickness of a sand drainage blanket was used for all models, while the thickness of the alluvium, upper marine deposit, lower marine deposit and glacial outwash/till varied from each site and each model particular to each station. Table 5-1 and Table 5-2 show the site stratigraphy used for both Exit 6N and Soundwall 3. The site stratigraphy was developed based on results from in situ testing and from exploration logs provided by NHDOT.
**Table 5-1: Site stratigraphy for each station at Exit 6N**

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>307+00</th>
<th>308+00</th>
<th>309+00</th>
<th>310+00</th>
<th>311+00</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage Blanket</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>1.5</td>
<td>ft</td>
</tr>
<tr>
<td>Alluvium</td>
<td>7.5</td>
<td>7</td>
<td>8</td>
<td>9</td>
<td>13</td>
<td>ft</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>3</td>
<td>3</td>
<td>3</td>
<td>4</td>
<td>8.5</td>
<td>ft</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>34</td>
<td>32.5</td>
<td>25.5</td>
<td>21.5</td>
<td>12.5</td>
<td>ft</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>5</td>
<td>5</td>
<td>5.5</td>
<td>7</td>
<td>5</td>
<td>ft</td>
</tr>
</tbody>
</table>

**Table 5-2: Site stratigraphy for each station at Soundwall 3**

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>1687+50</th>
<th>1688+00</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Drainage Blanket</td>
<td>1.5</td>
<td>1.5</td>
<td>ft</td>
</tr>
<tr>
<td>Alluvium</td>
<td>3</td>
<td>5</td>
<td>ft</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>1.5</td>
<td>1.5</td>
<td>ft</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>12.5</td>
<td>8.5</td>
<td>ft</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>13.5</td>
<td>9</td>
<td>ft</td>
</tr>
</tbody>
</table>
Wick drains are to be placed at 5 ft triangular spacing for the embankment at Exit 6N. For the analysis at Exit 6N, the settlement was analyzed with and without PV drains, for comparison of settlement and settlement rate values. Drains are not planned for the Soundwall 3 site and were not included in the analysis. Figure 5-3 shows the models of each embankment segment at Exit 6N using symmetry about the centerline of the embankment. The dimensions of each embankment model are shown in Table 5-3 and Table 5-4. The models of the embankment sections at Soundwall 3 are shown in Figure 5-4.

Table 5-3: Dimensions of Exit 6N embankment models

<table>
<thead>
<tr>
<th>Height (ft)</th>
<th>307+00</th>
<th>308+00</th>
<th>309+00</th>
<th>310+00</th>
<th>311+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>12</td>
<td>15.5</td>
<td>18.5</td>
<td>19.5</td>
<td>17.5</td>
<td></td>
</tr>
<tr>
<td>50</td>
<td>63.5</td>
<td>72</td>
<td>77.5</td>
<td>77</td>
<td></td>
</tr>
</tbody>
</table>

Table 5-4: Dimensions of Soundwall 3 embankment models

<table>
<thead>
<tr>
<th>Height (ft)</th>
<th>1687+50</th>
<th>1688+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>9</td>
<td>8</td>
<td></td>
</tr>
<tr>
<td>Half Width (ft)</td>
<td>28</td>
<td>26.5</td>
</tr>
</tbody>
</table>
Figure 5-3: Symmetric embankment sections for the proposed embankment at Exit 6N
5.3 Soft Soil Creep Model

For analyzing the settlement of the marine clay, the Soft Soil Creep (SSC) Model was chosen because of its ability to consider secondary consolidation (creep). Creep is an inherent behavior of highly compressible soils like normally consolidated clays, clayey silts or peat. It is the only constitutive model in PLAXIS that considers secondary consolidation. The parameters used in the Soft Soil (SS) model in PLAXIS are still applied to the SSC model, except SSC incorporates the modified creep index ($\mu^*$) as found in Equation 49:
\[
\mu^* = \frac{C_\alpha}{2.3(1 + e)} \quad (49)
\]

where \(C_\alpha\) is the secondary compression index and \(e\) is the void ratio. The modified creep index is used for the basis of creep rate during consolidation. Like the SS model, the SSC model uses a modified compression index \((\lambda^*)\) and a modified swelling index \((\kappa^*)\) to determine the compressibility of the clay during primary loading and unloading/reloading, as shown in the following equations:

\[
\lambda^* = \frac{C_c}{2.3(1 + e)} \quad (50)
\]

\[
\kappa^* = \frac{2C_s}{2.1(1 + e)} \quad (51)
\]

where \(C_c\) is the compression index, \(C_s\) is the swelling index. When the user inputs alternative values for \(C_c\), \(C_s\) and \(C_\alpha\) in PLAXIS, the program automatically updates the modified parameters based on the user input values (PLAXIS, 2015). The change in the creep rate is based on the combination of the three modified parameters.

5.4 Material Properties

Material properties for the embankment fill, sand drainage blanket, alluvium and glacial outwash/till were taken based on the findings of Santamaria (2015). Material properties of the marine clay deposits were updated based on laboratory and \textit{in situ} testing results for both sites.
as developed from the work associated with this project. A summary of the material properties used for finite element analysis is shown in Table 5-5. All material properties not specified in Table 5-5 are based on Santamaria (2015) and shown in Appendix F. Since DMT dissipation tests were not performed in the field, the horizontal permeability was not measured. The values determined by Santamaria (2015) were used, which were corrected for smear effects and plane strain conditions.

*Table 5-5: Material properties of the marine clay for finite element analysis*

<table>
<thead>
<tr>
<th></th>
<th>Exit 6N</th>
<th>Soundwall 3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Upper Marine</td>
<td>Lower Marine</td>
</tr>
<tr>
<td>Unit weight above phreatic level</td>
<td>116</td>
<td>112</td>
</tr>
<tr>
<td>Unit weight below phreatic level</td>
<td>116</td>
<td>112</td>
</tr>
<tr>
<td>Initial void ratio</td>
<td>1.17</td>
<td>1.20</td>
</tr>
<tr>
<td>Compression index</td>
<td>0.40</td>
<td>0.47</td>
</tr>
<tr>
<td>Recompression index</td>
<td>0.07</td>
<td>0.06</td>
</tr>
<tr>
<td>Secondary compression index</td>
<td>0.02</td>
<td>0.024</td>
</tr>
<tr>
<td>Vertical permeability</td>
<td>1.48E-04</td>
<td>4.56E-04</td>
</tr>
<tr>
<td>( K_0 )</td>
<td>1.0</td>
<td>0.7</td>
</tr>
<tr>
<td>OCR</td>
<td>7.0</td>
<td>1.6</td>
</tr>
</tbody>
</table>

5.5 Staged Construction

The embankments at Exit 6N and Soundwall 3 were modeled using staged construction for each section as shown in Table 5-6 and Table 5-7, respectively.
Table 5-6: Construction schedule for the Exit 6N embankment

<table>
<thead>
<tr>
<th>Phase</th>
<th>Calculation</th>
<th>Loading</th>
<th>Duration</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Phase</td>
<td>$K_0$ Condition</td>
<td>Staged Construction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sand Blanket Placement</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>2</td>
<td>Days</td>
</tr>
<tr>
<td>PV Drain Installation</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>1</td>
<td>Days</td>
</tr>
<tr>
<td>Waiting Period</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>Varies</td>
<td>Days</td>
</tr>
<tr>
<td>Place Embankment (11 ft max)</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>40</td>
<td>Days</td>
</tr>
<tr>
<td>Waiting Period</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>45</td>
<td>Days</td>
</tr>
<tr>
<td>Place Embankment (5 ft max)</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>2</td>
<td>Days</td>
</tr>
<tr>
<td>Waiting Period</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>45</td>
<td>Days</td>
</tr>
<tr>
<td>Place Embankment (To grade)</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>1</td>
<td>Days</td>
</tr>
<tr>
<td>Consolidation</td>
<td>Consolidation</td>
<td>Minimum Excess Pore Pressure</td>
<td>Varies</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 5-7: Construction schedule for the Soundwall 3 embankment

<table>
<thead>
<tr>
<th>Phase</th>
<th>Calculation</th>
<th>Loading</th>
<th>Duration</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Phase</td>
<td>$K_0$ Condition</td>
<td>Staged Construction</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Sand Blanket Placement</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>2</td>
<td>Days</td>
</tr>
<tr>
<td>Place Embankment</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>40</td>
<td>Days</td>
</tr>
<tr>
<td>Consolidation</td>
<td>Consolidation</td>
<td>Minimum Excess Pore Pressure</td>
<td>Varies</td>
<td>-</td>
</tr>
</tbody>
</table>

The waiting period between drain installation and the placement of the first embankment section varies at each station. For the analysis it was assumed that construction would start at Sta. 307+00 and move north to Sta. 311+00, so the waiting period was set to 13 days at 307+00, and sequentially decreases by a day to a 9 day waiting period at Sta. 311+00. These values were taken from Santamaria (2015) as estimates for the construction schedule, since the embankment at Exit 6N is likely to follow a similar sequence to the construction of the Dover Test Embankment.
5.6 Mesh Discretization

For all of the models, a fine mesh distribution was used to improve the overall predictions within the marine clay deposit. Compared to a medium or coarse mesh, a fine mesh divides the model into more elements and nodes to create more data points for calculating deformations. Table 5-8 and Table 5-9 show the number of elements and nodes generated by the discretization for each model.

Table 5-8: Model elements and nodes at Exit 6N

<table>
<thead>
<tr>
<th>Mesh Distribution</th>
<th>307+00</th>
<th>308+00</th>
<th>309+00</th>
<th>310+00</th>
<th>311+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Elements</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>3061</td>
<td>2896</td>
<td>2995</td>
<td>3020</td>
<td>2888</td>
<td></td>
</tr>
<tr>
<td>Number of Nodes</td>
<td>24855</td>
<td>23409</td>
<td>24211</td>
<td>24429</td>
<td>23393</td>
</tr>
</tbody>
</table>

Table 5-9: Model elements and nodes at Soundwall 3

<table>
<thead>
<tr>
<th>Mesh Distribution</th>
<th>1687+50</th>
<th>1688+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Elements</td>
<td>742</td>
<td>611</td>
</tr>
<tr>
<td>Number of Nodes</td>
<td>6117</td>
<td>5059</td>
</tr>
</tbody>
</table>

5.7 Boundary Conditions

Boundary conditions were set for groundwater flow conditions and deformation. Deformation boundary conditions have no effect on the behavior of the embankment model when set a far enough distance from the model area. PLAXIS will then ignore any deformation boundary conditions set along the x and y boundaries, resulting in groundwater flow conditions as the controlling boundary condition. PLAXIS allows the user to choose the groundwater flow
conditions that is most appropriate for the model. The seepage condition is used often in finite element analysis of embankments, and was used for this analysis.

Boundary conditions were set in the same manner as suggested by Santamaria (2015) and shown in Figure 5-5. The displacement boundaries were set to the defaults set by PLAXIS. The $y_{\text{min}}$ boundary is fully fixed for displacement while $y_{\text{max}}$ is free for displacement. Both $x$ boundaries are normally fixed. The $x_{\text{min}}$ boundary for Exit 6N models and the $x_{\text{max}}$ boundary for Soundwall 3 models are closed to groundwater flow, while all remaining boundaries are open.

*Figure 5-5: Boundary conditions set in PLAXIS models based on Santamaria (2015)*
5.8 Points for Curves

In order to plot the results from the PLAXIS calculations, points must be selected from the discretized mesh to obtain the results. For embankment deformations the closest node to the embankment center at ground surface was chosen for all models. For plotting the excess porewater pressure the closest node to the mid-point of the lower clay layer near the embankment centerline was selected for all models. Figure 5-6 is an example of the point locations for plotting deformation (Pt. A) and excess porewater pressure (Pt. B).

Figure 5-6: Example of the location of selected points for generating results curves

5.9 Degree of Consolidation

The minimum excess pore pressure calculation was used to simulate 90% consolidation for the model. Please refer to Santamaria (2015) for the procedure.

5.10 PLAXIS Results

Each model was calculated to 90% consolidation. The maximum settlement and time to reach 90% consolidation for all models with wick drains are shown in Table 5-10 and Table 5-11. An
example of the calculated settlement relative to the proposed elevation of the Exit 6N embankment crest is shown in Figure 5-7. Deformed meshes of each embankment section are shown from Figure 5-8 to Figure 5-12 for Exit 6N and Figure 5-13 to Figure 5-14 for Soundwall 3. Typical outputs of vertical and horizontal deformation contours are shown in Figure 5-15 and Figure 5-16.

*Table 5-10: Summary of results from finite element analysis of the Exit 6N embankment*

<table>
<thead>
<tr>
<th></th>
<th>307+00</th>
<th>308+00</th>
<th>309+00</th>
<th>310+00</th>
<th>311+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time to 90% consolidation (years)</td>
<td>5.81</td>
<td>4.21</td>
<td>2.94</td>
<td>2.33</td>
<td>1.75</td>
</tr>
<tr>
<td>Maximum vertical settlement (ft)</td>
<td>2.15</td>
<td>2.58</td>
<td>2.32</td>
<td>1.97</td>
<td>1.02</td>
</tr>
</tbody>
</table>

*Table 5-11: Summary of results from finite element analysis of the Soundwall 3 embankment*

<table>
<thead>
<tr>
<th></th>
<th>1687+50</th>
<th>1688+00</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time to 90% consolidation (days)</td>
<td>82.5</td>
<td>62.9</td>
</tr>
<tr>
<td>Maximum vertical settlement (ft)</td>
<td>0.19</td>
<td>0.11</td>
</tr>
</tbody>
</table>
Figure 5-7: Settlement of the Exit 6N embankment

Figure 5-8: Deformed mesh at 90% consolidation for the embankment at Exit 6N Sta. 307+00
Figure 5-9: Deformed mesh at 90% consolidation for the embankment at Exit 6N Sta. 308+00

Figure 5-10: Deformed mesh at 90% consolidation for the embankment at Exit 6N Sta. 309+00
Figure 5-11: Deformed mesh at 90% consolidation for the embankment at Exit 6N Sta. 310+00

Figure 5-12: Deformed mesh at 90% consolidation for the embankment at Exit 6N Sta. 311+00
Figure 5-13: Deformed mesh at 90% consolidation for the embankment at Soundwall 3 Sta. 1687+50

Deformed mesh $|u|$ (scaled up 10.0 times)
Maximum value = 0.1879 ft (Element 1 at Node 2)

Figure 5-14: Deformed mesh at 90% consolidation for the embankment at Soundwall 3 Sta. 1688+00

Deformed mesh $|u|$ (scaled up 10.0 times)
Maximum value = 0.1109 ft (Element 1 at Node 3)
Figure 5-15: Vertical deformation at 90% consolidation for the embankment at Exit 6N Sta. 307+00

Figure 5-16: Horizontal deformation at 90% consolidation for the embankment at Exit 6N Sta. 307+00
The magnitude of consolidation at Exit 6N ranged from 1.02 to 2.58 feet. Since the material properties remained the same for each model, the contributing factors to the differences in values were based on physical dimensions like embankment fill height, thickness of marine clay and depth to marine clay. The vertical settlement of each embankment section is graphically displayed in Figure 5-17. The increase in vertical stress from the embankment is directly related to the embankment geometry and fill height. While the marine deposit layer at Sta. 307+00 was 1.5 feet thicker than at Sta. 308+00, the embankment fill height was 3.5 feet less, which resulted in a reduction of settlement by 0.43 feet (5.2 inches). However, a thicker marine deposit required more time to achieve 90% consolidation. The embankment section at Sta. 307+00 took 1.6 years longer to reach 90% settlement than the embankment section at Sta. 308+00, as shown in Table 5-12. A hand calculation of settlement at the mid-point of the lower marine deposit layer was performed as a check for the PLAXIS results. The PLAXIS model predicts approximately 1 inch more settlement at 90% consolidation than the value obtained from the hand calculation. Please refer to Appendix G for the full calculation.

Table 5-12: Comparing embankment height and marine deposit thickness to magnitude and rate of settlement for the Exit 6N embankment

<table>
<thead>
<tr>
<th>Station</th>
<th>Height of Embankment (ft)</th>
<th>Thickness of Marine Deposit (ft)</th>
<th>Maximum Vertical Settlement at 90% Consolidation (ft)</th>
<th>Time to 90% Consolidation (years)</th>
</tr>
</thead>
<tbody>
<tr>
<td>307+00</td>
<td>12</td>
<td>37</td>
<td>2.15</td>
<td>5.81</td>
</tr>
<tr>
<td>308+00</td>
<td>15.5</td>
<td>35.5</td>
<td>2.58</td>
<td>4.21</td>
</tr>
<tr>
<td>309+00</td>
<td>18.5</td>
<td>28.5</td>
<td>2.32</td>
<td>2.94</td>
</tr>
<tr>
<td>310+00</td>
<td>19.5</td>
<td>25.5</td>
<td>1.97</td>
<td>2.33</td>
</tr>
<tr>
<td>311+00</td>
<td>17.5</td>
<td>21</td>
<td>1.02</td>
<td>1.75</td>
</tr>
</tbody>
</table>
The magnitude of consolidation at Soundwall 3 ranged from 0.111 ft (1.3 in.) to 0.188 ft (2.3 in.). The amount of consolidation was expected to be lower at this site due to much stiffer behavior of the marine clay at the location. The vertical settlement of each embankment section is graphically displayed in Figure 5-18. An additional 1 ft of embankment fill height and 4 feet of marine deposit accounted for 1 inch more settlement, as well as 19.6 additional days to achieve 90% consolidation, as shown in Table 5-13.

Table 5-13: Comparing embankment height and marine deposit thickness to magnitude and rate of settlement for the Soundwall 3 embankment

<table>
<thead>
<tr>
<th>Station</th>
<th>Height of Embankment (ft)</th>
<th>Thickness of Marine Deposit (ft)</th>
<th>Maximum Vertical Settlement at 90% Consolidation (ft)</th>
<th>Time to 90% Consolidation (days)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1687+50</td>
<td>9</td>
<td>14</td>
<td>0.19</td>
<td>82.5</td>
</tr>
<tr>
<td>1688+00</td>
<td>8</td>
<td>10</td>
<td>0.11</td>
<td>62.9</td>
</tr>
</tbody>
</table>
As shown in Figure 5-16, the horizontal deformation contours are output in an irregular “wavy” pattern. The plane strain corrections for the wick drains are only applied to the permeability and flow conditions and not to the drains directly. This causes the drains to act as an “obstacle” for horizontal deformation. The figure also suggests that embankment will experience lateral movement over time.

The excess porewater pressure generated during staged construction is shown in Figure 5-19 and Figure 5-20. As depicted in Figure 5-19, the excess porewater pressure increases during placement of the drainage blanket, wick drains and embankment, and dissipates during the waiting periods. Each embankment section exhibits similar trends to one another. Embankment height, thickness of marine clay and depth to marine clay all change the amount and distribution of excess porewater pressure generated with depth.
Figure 5-19: Excess porewater pressure versus time for the Exit 6N embankment

Figure 5-20: Excess porewater pressure versus time for the Soundwall 3 embankment
5.10.1 Drains vs. No Drains

An analysis was performed comparing the rate of settlement for a case without using wick drains. Using the section at Sta. 307+00 at Exit 6N, a new mesh was generated without considering wick drains. By eliminating wick drains the mesh becomes less discretized, as evidenced by Table 5-14.

Table 5-14: Comparison of mesh discretization with and without PV drains for the Exit 6N embankment section at Sta. 307+00

<table>
<thead>
<tr>
<th></th>
<th>With Drains</th>
<th>Without Drains</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Elements</td>
<td>3061</td>
<td>924</td>
</tr>
<tr>
<td>Number of Nodes</td>
<td>24855</td>
<td>7641</td>
</tr>
</tbody>
</table>

As expected, the rate of settlement without using wick drains should be slower than with wick drains. Since wick drains do not change the amount in which the clay will consolidate, the settlement for both cases should be approximately the same. The results are shown in Table 5-15 and graphically displayed in Figure 5-21. The vertical settlement for the case without using wick drains is slightly less, due to the fact that the mesh was not as discretized. What is more evident is the amount of time required to dissipate the excess porewater pressure in the clay, as shown in Figure 5-22. Figure 5-23 compares the distribution of excess porewater pressure at 90% consolidation for both cases. The wick drains effectively drain the excess porewater out from beneath the embankment, whereas without drains a large distribution of excess porewater pressure is still present beneath the embankment.
The amount of settlement one year after the completed embankment construction was evaluated to compare the amount of settlement that occurred for each case. For the case with wick drains the maximum vertical settlement was 1.55 feet, accounting for approximately 65% of the total settlement, whereas for the case without wick drains the maximum vertical settlement was 0.9 feet, accounting for approximately 38% of the total settlement. After one year there is a 27% difference in the amount of consolidation that would occur when using wick drains.

Table 5-15: Comparison of time and magnitude of settlement with and without PV drains for the Exit 6N embankment section at Sta. 307+00

<table>
<thead>
<tr>
<th></th>
<th>With Drains</th>
<th>Without Drains</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time to 90% Consolidation (years)</td>
<td>5.81</td>
<td>9.35</td>
</tr>
<tr>
<td>Maximum Vertical Settlement (ft)</td>
<td>2.15</td>
<td>2.11</td>
</tr>
</tbody>
</table>
Figure 5-21: Vertical settlement at 90% consolidation for cases of with and without wick drains for the embankment section at Sta. 307+00

Figure 5-22: Excess porewater pressure at 90% consolidation for cases of with and without wick drains for the embankment section at Sta. 307+00
Figure 5-23: Distribution of excess porewater pressure at 90% consolidation comparing the case with drains (top) and without drains (bottom) for the embankment section at Sta. 307+00
5.10.2 Soft Soil Creep vs. Soft Soil

In order to show the effect of secondary compression on total settlement, an analysis was performed comparing the Soft Soil Creep constitutive model to the Soft Soil model, which does not consider secondary compression. The embankment section at Sta. 307+00 with wick drains was used, with the settlement results shown in Table 5-16 and graphically displayed in Figure 5-24. The Soft Soil model predicts less excess porewater pressure generation and a faster rate of dissipation during the embankment stage waiting period, thus resulting in a faster time to reach full consolidation, as evidenced in Figure 5-25.

According to Waterman (2011), over time creep causes pore pressures in the soil to further increase, decreasing the creep rate and initiating consolidation. During consolidation the pore pressures begin to dissipate, which in turn increases the creep rate, slowing down the consolidation process. The creep behavior of the material directly affects the consolidation behavior, which is why there is such noticeable differences in degree of settlement comparing the SS model to the SSC model. Figure 5-26 shows the distribution of excess porewater pressure at 90% consolidation comparing the SS model to the SSC model. By introducing creep to the model, more excess porewater pressure is generated, thus more excess porewater pressure will be present at 90% of consolidation.

However, the model predicts over two times less vertical settlement than the Soft Soil Creep model does. Based on Santamaria (2015) the SSC model more accurately predicted the field measurements of the Dover Test Embankment. The SS model predicted approximately 1.1 feet of settlement at 90% consolidation, but settlements in the field had already been measured at 1.5 feet, suggesting the predicted value of 2.1 feet of settlement from the SSC model to be...
more accurate. Santamaria (2015) also mentions that while the SSC model better predicted the degree of settlement, the SS model may have predicted the rate of consolidation more accurately, based on the calculated field settlements at the time.

Table 5-16: Comparison of time and magnitude of settlement with SSC and SS models for the Exit 6N embankment section at Sta. 307+00

<table>
<thead>
<tr>
<th></th>
<th>Soft Soil Creep</th>
<th>Soft Soil</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time to 90% Consolidation (years)</td>
<td>5.81</td>
<td>1.81</td>
</tr>
<tr>
<td>Maximum Vertical Settlement (ft)</td>
<td>2.15</td>
<td>0.85</td>
</tr>
</tbody>
</table>

Figure 5-24: Vertical settlement comparing Soft Soil Creep Model to Soft Soil Model for the embankment section at Sta. 307+00
Figure 5-25: Excess porewater pressure comparing Soft Soil Creep Model to Soft Soil Model for the embankment section at Sta. 307+00
Figure 5-26: Distribution of excess porewater pressure at 90% consolidation comparing the case with the SSC model (top) and with the SS model (bottom) for the embankment section at Sta. 307+00
5.10.3 $K_0$ Variation

An analysis was performed comparing the total settlement with changes in the coefficient of lateral earth pressure at rest ($K_0$). As discussed previously in Chapter 4, one calculation was performed changing $K_0$ of the lower marine clay from 0.70 to 0.90 for the embankment at Sta. 307+00. The results from the finite element analysis are shown in Table 5-17.

Table 5-17: Comparison of time and magnitude of deformations with different $K_0$ values for the Exit 6N embankment section at Sta. 307+00

<table>
<thead>
<tr>
<th></th>
<th>$K_0=0.7$</th>
<th>$K_0=0.9$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Time to 90% Consolidation (years)</td>
<td>5.81</td>
<td>5.50</td>
</tr>
<tr>
<td>Maximum Vertical Settlement (ft)</td>
<td>2.15</td>
<td>1.96</td>
</tr>
<tr>
<td>Maximum Horizontal Displacement (ft)</td>
<td>0.37</td>
<td>0.33</td>
</tr>
</tbody>
</table>

Increasing $K_0$ resulted in a decrease of total vertical settlement by 0.19 feet (2.27 in.) which is approximately 9% less than the original predicted settlement. The maximum horizontal displacement decreased by 0.04 ft (0.49 in.), which is approximately 11% less than the original predicted lateral deformation. The resulting decrease in deformation also shortened the amount of time required for consolidation. Increasing $K_0$ will increase the horizontal effective stress applied to the soil at-rest. Upon loading, the deviatoric stress ($\sigma'_v-\sigma'_h$) will then be lower, thus resulting in less deformation.

5.11 Model Validation- Settle 3D

The calculations from PLAXIS need to be validated using another form of analysis. A 3D model of one of the embankments was created using the Rocscience program Settle3D. The program
has the capability of computing stresses in three dimension while calculating deformation in one dimension. The user can model the settlement in stages and analyze primary and secondary consolidation. The embankment dimensions, subsurface conditions and construction sequence of the embankment section at Sta. 307+00 of Exit 6N was used. Table 5-18 summarizes the material properties used for Settle 3D. It should be noted that any unknown properties of the material used the default values given by the program.

*Table 5-18: Material properties for Settle3D*

<table>
<thead>
<tr>
<th>Material Name</th>
<th>Color</th>
<th>Unit Weight (tonnes/ft3)</th>
<th>Sat. Unit Weight (tonnes/ft3)</th>
<th>Material Type</th>
<th>Cc</th>
<th>Cr</th>
<th>OCR</th>
<th>e0</th>
<th>mv (2%), mmv (5%)</th>
<th>K (kPa)</th>
<th>Kr (kPa)</th>
<th>B-bar</th>
<th>Cc/Cv Ratio</th>
<th>N-value</th>
<th>Secondary Consol. Method</th>
<th>Clr/Clc Ratio</th>
<th>Soil Type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td></td>
<td>0.0015</td>
<td>0.0015</td>
<td>Linear</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>0.01987</td>
<td>0.01987</td>
<td>0.8135</td>
<td></td>
<td>1</td>
<td>1</td>
<td>West</td>
<td></td>
<td>Soft Clay</td>
</tr>
<tr>
<td>Upper Marine Decei</td>
<td>0.077825</td>
<td>0.057825</td>
<td>Non-Linear</td>
<td>6.4935</td>
<td>0.5872</td>
<td>5.95</td>
<td>1.17</td>
<td>0.000148</td>
<td>0.000148</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>West</td>
<td>0.05</td>
<td>Soft Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower Marine Deposit</td>
<td>0.05567</td>
<td>0.05567</td>
<td>Non-Linear</td>
<td>6.4722</td>
<td>0.0894</td>
<td>1.56</td>
<td>1.2</td>
<td>0.000456</td>
<td>0.000456</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>West</td>
<td>0.05</td>
<td>Soft Clay</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glacial Till/Outwash</td>
<td>0.0836</td>
<td>0.0836</td>
<td>Linear</td>
<td></td>
<td>0.01967</td>
<td>0.1967</td>
<td>1.968</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>West</td>
<td></td>
<td>Soft Clay</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A trial was run using PV drains at 5 ft triangular spacing. The consolidation stage of the analysis was set to the 2120 days (5.81 years), the amount required for 90% consolidation in the PLAXIS simulation for the same embankment section. The results of Settle3D are compared to the PLAXIS results in Table 5-19 and Figure 5-27. Within the same amount of time Settle3D predicted 30% less settlement (0.644 ft) than PLAXIS. Settle3D also produced less settlement than PLAXIS in Getchell’s (2013) analysis of the Dover Test Embankment.

There are a few contributing factors that could have led to the differences in the results. Settle3D does not require as many geotechnical parameters for defining soils as PLAXIS does, which means that the soil behavior may not have been modeled as accurately. The user has the option of choosing from various constitutive models for the material type in PLAXIS, while Settle3D only assumes a linear or non-linear material type. Also, Settle3D has the capability of
modelling the wick drains in a triangular pattern, whereas PLAXIS 2D models the drains at an infinite length in the z-direction. This could mean Settle3D is more accurate in calculating the rate in which excess porewater drains from the clay.

Table 5-19: Comparison of vertical settlement with PLAXIS and Settle3D

<table>
<thead>
<tr>
<th></th>
<th>PLAXIS</th>
<th>Settle3D</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Settlement (ft)</td>
<td>2.15</td>
<td>1.51</td>
</tr>
</tbody>
</table>

Figure 5-27: Comparison of vertical settlement with PLAXIS and Settle3D of the embankment section at Exit 6N Sta. 307+00
Another trial was run setting the last stage to 10 years to see how much additional settlement would occur. The results are shown in Table 5-20. With over 4 additional years added to the calculation, Settle3D calculates an additional 0.086 ft (1.0 in.) to occur. The additional 1 in. in settlement would probably occur from secondary consolidation, since it is safe to say that 100% of primary consolidation would have occurred in the Settle3D analysis.

Table 5-20: Comparing vertical settlement in Settle3D with different time values for the consolidation stage

<table>
<thead>
<tr>
<th>Vertical Settlement (ft)</th>
<th>Trial 1 (5.81 years)</th>
<th>Trial 2 (10 years)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1.51</td>
<td>1.59</td>
</tr>
</tbody>
</table>

A plan and model view of the embankment in Settle3D are shown in Figure 5-28 and Figure 5-29.
Figure 5-28: Displacement gradient from the top view of the embankment model in Settle3D
Figure 5-29: Side view of the embankment model with wick drains and displacement gradient
6 SUMMARY AND CONCLUSIONS

6.1 Summary

In the spring of 2014, the University of New Hampshire was approached by the NHDOT to provide engineering services for future embankments as part of the road network expansion of the Spaulding Turnpike located in Dover, New Hampshire. A soft marine clay deposit, known as the Presumpscot Formation, has been documented throughout the New Hampshire Seacoast region, and has presented challenges in geotechnical construction. From past research and publications this deposit has been determined to have high compressibility, which can result in differential settlements of a highway embankment constructed upon it. Prefabricated vertical drains will be installed at one of the sites to increase the consolidation rate of the marine clay.

In situ and laboratory testing was performed to evaluate geotechnical properties of the marine clay prior to embankment construction. The in situ testing program included dilatometer testing, field vane testing and piezocone testing, as well as Shelby tube sampling for obtaining undisturbed samples of clay. The clay samples were taken to UNH for laboratory consolidation testing. The results from the in situ and laboratory tests were analyzed to provide soil properties of the marine clay to help predict settlements that would occur from the embankment loads. Overall the field and laboratory data appeared to be in accordance with data from previous research in the NH Seacoast area, such as: Ladd et al. (1972), Findlay (1991) and Getchell (2013). The marine clay deposit located at Exit 6N appears to have a much softer and more compressible behavior than the deposit located the Soundwall 3 site.
The geotechnical finite element analysis software PLAXIS 2D was used to predict the magnitude and rate of consolidation based on the information from the testing program. The model was compared to a calculation using Settle3D to validate the results. The values will be used as a baseline during and after the construction of the embankments. Different cases were applied to the finite element models to confirm the suggestions of Santamaria (2015).

6.2 Conclusions

The proposed embankment at Exit 6N will include prefabricated vertical drains to accelerate the rate of consolidation of the soft clay. A comparison of FEA results of an embankment with PV drains and one without showed that essentially the same degree of consolidation had occurred for both cases, but the wick drains did an effective job draining the excess porewater pressure from the soil. The results confirm that wick drains are a viable option for the site because they reach 90% consolidation much sooner, allowing the NHDOT to open the exit ramp earlier.

The calculated settlement at 90% consolidation of the proposed Exit 6N embankment ranged between 1.20 and 2.58 feet, and the calculated $c_v$ ranged between 49.7 and 63.5 ft²/year. The FEA calculations for the Dover Test Embankment from Santamaria (2015) predicted a $c_v$ ranging between 21.0 and 41.9 ft²/year. A few contributing factors caused the differences in results:

- The compression index used in Santamaria’s (2015) calculations was 0.36, as opposed to 0.47 for this research. A higher compression index is indicative of a more compressible soil, which will lead to more settlement.
The Dover Test Embankment site has a thicker layer of marine clay than the Exit 6N site. A thicker soil layer results in a longer drainage path, which then translates to a slower rate of consolidation.

The wick drain spacing for Santamaria’s (2015) models ranged between 6 feet and 14 feet, as opposed to 5 feet at the Exit 6N model. The lateral distance for excess porewater to permeate will change based on the spacing of the wick drains. The closer the drains, the less distance the porewater flows laterally into the drain, which in turn accelerates consolidation.

The embankment fill height for the proposed Exit 6N embankment ranges between 12 feet and 19.5 feet. The fill height for 4 of the 5 sections of the Dover Test Embankment was 12 feet, and one section had a fill height of 18 feet. A greater embankment fill height will increase the applied loading from the embankment, leading to more settlement. The FEA predictions of the Exit 6N models calculate degrees of settlement comparable to the Dover Test Embankment, despite having a thinner layer of marine deposit.

The results comparing the Soft Soil model to the Soft Soil Creep model proved the effect that creep has on the magnitude and rate of consolidation of the soil. If secondary compression is not taken into account the predicted values could show significantly lower settlement predictions than what may actually happen in the field, which could result in additional construction and maintenance costs.
6.3 Recommendations

The DMT and CPTu provided plenty of data to characterize the subsurface conditions of the soils at both sites. After applying specific corrections to the data, both test methods provide viable results to determine the parameters and behavior of the Presumpscot Formation. While the CPTu provides more data points and a “fuzzier” output from continuous pushing, the DMT provides sufficient means of testing the marine deposit in a very easy to use manner. Future research could include a comparison on the accuracy of the DMT versus the CPTu for characterizing the Presumpscot Formation.

Special care and consideration should be taken during the sampling, transportation, and curing of undisturbed Shelby tube samples. A container with a damping material should be made for the samples during transportation to minimize disturbance. A more in depth study of the effects of sample disturbance specifically on the Presumpscot Formation could be carried out.

Several material properties were not known and were assumed using values assigned by PLAXIS or determined by Santamaria (2015) for the marine deposit located at the Newington-Dover Test Embankment. The horizontal permeability was used from Santamaria (2015) based on the DMT dissipation tests performed by Getchell (2013). As a confirmation DMT dissipation tests should be performed at the Exit 6N and Soundwall 3 sites to see how much horizontal permeability should be adjusted. The apparent cohesion and effective friction angle were assumed from PLAXIS suggestions and should be confirmed through triaxial testing.

The secondary compression index was determined using the relationship from Mesri (1973). The LoadTrac’s in the UNH laboratory appeared to have difficulty sustaining applied loads after
90% consolidation, rendering unusual trends. It is suggested to conduct additional tests to determine the secondary compression index directly from laboratory testing.

Lastly, the sites should be monitored during and after the construction of the embankments. While the predicted values in this study should serve as a good indication of the potential behavior, on-site monitoring should be conducted to verify the validity of the results.
REFERENCES


Appendix A: Dilatometer Results
Figure A-1: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 310+00 RT. 40
Figure A-2: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 309+00 RT. 30
Figure A-3: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 308+00 RT. 30
Figure A-4: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 307+00 RT. 30
Figure A-5: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 307+00 LT. 30
Figure A-6: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 308+00 LT. 19
Figure A-7: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 308+10 LT. 98
Figure A-8: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 308+95 LT. 75
Figure A-9: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 309+00 LT. 30
Figure A-10: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 310+00 LT. 61
**Figure A-11: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 311+00 LT. 30**
Figure A-12: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 311+00 RT. 40
Figure A-13: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 1687+50 LT. 90
Figure A-14: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 1688+00 LT. 90
Figure A-15: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 1688+00 LT. 135
Figure A-16: DMT Readings, Material Index, Constrained Modulus, Undrained Shear Strength and Horizontal Stress Index for Profile at Sta. 1687+50 LT. 132
Appendix B: Field Vane Results
NHDOT Exit 6N Embankment
Dover, NH

University of New Hampshire
Geonor H-10 Vane Borer Testing

Figure B-1: Sta. 307+90 RT. 30 Field Vane Test at Elevation -2.8 ft
NHDOT Exit 6N Embankment
Dover, NH

University of New Hampshire
Geonor H-10 Vane Borer Testing

Figure B-2: Sta. 307+90 RT. 30 Field Vane Test at Elevation -4.8 ft
Figure B-3: Sta. 307+90 RT. 30 Field Vane Test at Elevation -7.8 ft
Figure B-4: Sta. 307+90 RT. 30 Field Vane Test at Elevation -9.8 ft
NHDOT Exit 6N Embankment
Dover, NH
University of New Hampshire
Geonor H-10 Vane Borer Testing

Figure B-5: Sta. 307+90 RT. 30 Field Vane Test at Elevation -11.8 ft
Figure B-6: Sta. 307+90 RT. 30 Field Vane Test at Elevation -14.8 ft
Figure B-7: Sta. 307+90 RT. 30 Field Vane Test at Elevation -18.8 ft
Figure B-8: Sta. 307+90 RT. 30 Field Vane Test at Elevation -20.8 ft
Figure B-9: Sta. 307+90 RT. 30 Field Vane Test at Elevation -22.8 ft
Figure B-10: Sta. 307+90 RT. 30 Field Vane Test at Elevation -24.8 ft
Figure B-11: Sta. 307+90 LT. 80 Field Vane Test at Elevation -3.3 ft
Figure B-12: Sta. 307+90 LT. 80 Field Vane Test at Elevation -5.3 ft
Figure B-13: Sta. 307+90 LT. 80 Field Vane Test at Elevation -7.3 ft
Figure B-14: Sta. 307+90 LT. 80 Field Vane Test at Elevation -10.3 ft
Figure B-15: Sta. 307+90 LT. 80 Field Vane Test at Elevation -12.3 ft
Figure B-16: Sta. 307+90 LT. 80 Field Vane Test at Elevation -14.3 ft
Figure B-17: Sta. 307+90 LT. 80 Field Vane Test at Elevation -16.3 ft
NHDOT Exit 6N Embankment
Dover, NH
University of New Hampshire
Geonor H-10 Vane Borer Testing

Figure B-18: Sta. 307+90 LT. 80 Field Vane Test at Elevation -23.3 ft
Figure B-19: Sta. 307+90 LT. 80 Field Vane Test at Elevation -25.3 ft
NHDOT Exit 6N Embankment
Dover, NH

University of New Hampshire
Geonor H-10 Vane Borer Testing

Free Field Conditions
Station: 307+90 LT. 80
Test No.: 10
Depth: 45'8"
11/6/2015

Figure B-20: Sta. 307+90 LT. 80 Field Vane Test at Elevation -28.3 ft
NHDOT Soundwall 3 Embankment
Dover, NH

University of New Hampshire
Geonor H-10 Vane Borer Testing

Free Field Conditions
Station: 1687+90 LT. 135
Test No.: 1
Depth: 9'8"
8/17/2015

Figure B-21: Sta. 1687+90 LT. 135 Field Vane Test at Elevation +7.3 ft
Figure B-22: Sta. 1687+90 LT. 135 Field Vane Test at Elevation +3.3 ft
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Figure C-1: Sta. 308+10 LT. 19 CPTu Readings

Figure C-2: Sta. 308+06 LT. 91 CPTu Readings
Figure C-3: Sta. 308+10 RT. 30 CPTu Readings

Figure C-4: Sta. 1687+96 LT. 135 CPTu Readings
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Figure D-2: Consolidation Curve for a Sample at Elevation -11.7 ft
**Figure D-3: Consolidation Curve for a Sample at Elevation -19.6 ft**

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Dover, NH
Figure D-4: Consolidation Curve for a Sample at Elevation +2.5 ft
Figure D-5: Consolidation Curve for a Sample at Elevation -8.0 ft
Figure D-6: Consolidation Curve for a Sample at Elevation -15.7 ft
Figure D-7: Consolidation Curve for a Sample at Elevation -23.7 ft
Figure D-8: Consolidation Curve for a Sample at Elevation +12.8 ft
Figure D-9: Consolidation Curve for a Sample at Elevation +9.1 ft
Figure D-10: Consolidation Curve for a Sample at Elevation +5.2 ft
Appendix E: PLAXIS Results
Figure E-1: PLAXIS 2D Sta. 307+00 (Drains) Vertical Deformation at 90% Consolidation

**Total displacements $u_y$**

- Maximum value = 0.000 ft (Element 2951 at Node 1277)
- Minimum value = -2.151 ft (Element 245 at Node 24576)
Figure E-2: PLAXIS 2D Sta. 307+00 (Drains) Horizontal Deformation at 90% Consolidation
Figure E-3: PLAXIS 2D Sta. 307+00 (Drains) Excess Pore Pressure at 90% Consolidation

Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)

Maximum value = 1.316 lbf/ft$^2$ (Element 1102 at Node 159)
Minimum value = -165.2 lbf/ft$^2$ (Element 2784 at Node 8510)
Figure E-4: PLAXIS 2D Sta. 308+00 (Drains) Vertical Deformation at 90% Consolidation

Total displacements $u_y$

Maximum value = 0.000 ft (Element 2808 at Node 1443)
Minimum value = -2.580 ft (Element 22 at Node 22856)
Figure E-5: PLAXIS 2D Sta. 308+00 (Drains) Horizontal Deformation at 90% Consolidation

Total displacements $u_x$
Maximum value = 0.4981 ft (Element 1071 at Node 16065)
Minimum value = -0.3231 ft (Element 12 at Node 23107)
Figure E-6: PLAXIS 2D Sta. 308+00 (Drains) Excess Pore Pressure at 90% Consolidation

**Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)**
- Maximum value = 1.685 lb/ft$^2$ (Element 950 at Node 159)
- Minimum value = -146.9 lb/ft$^2$ (Element 2638 at Node 6808)
Figure E-7: PLAXIS 2D Sta. 309+00 (Drains) Vertical Deformation at 90% Consolidation

Total displacements $u_y$

Maximum value = 0.000 ft (Element 2871 at Node 1007)
Minimum value = -2.321 ft (Element 26 at Node 24197)
Figure E-8: PLAXIS 2D Sta. 309+00 (Drains) Horizontal Deformation at 90% Consolidation

Total displacements $u_x$

Maximum value = 0.4711 ft (Element 2119 at Node 14348)
Minimum value = -0.3059 ft (Element 12 at Node 22231)
Figure E-9: PLAXIS 2D Sta. 309+00 (Drains) Excess Pore Pressure at 90% Consolidation
Figure E-10: PLAXIS 2D Sta. 310+00 (Drains) Vertical Deformation at 90% Consolidation

Total displacements $u_y$

Maximum value = 0.000 ft (Element 2891 at Node 23733)
Minimum value = -1.972 ft (Element 26 at Node 1381)
Figure E-11 PLAXIS 2D Sta. 310+00 (Drains) Horizontal Deformation at 90% Consolidation

Total displacements $u_x$
Maximum value = 0.3998 ft (Element 2354 at Node 11174)
Minimum value = -0.2748 ft (Element 13 at Node 1309)
Figure E-12: PLAXIS 2D Sta. 310+00 (Drains) Excess Pore Pressure at 90% Consolidation

Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)
Maximum value = 0.9092 lb/ft$^2$ (Element 1258 at Node 5091)
Minimum value = -171.1 lb/ft$^2$ (Element 2798 at Node 20324)
Figure E-13: PLAXIS 2D Sta. 311+00 (Drains) Vertical Deformation at 90% Consolidation
Figure E-14: PLAXIS 2D Sta. 311+00 (Drains) Horizontal Deformation at 90% Consolidation

Total displacements $u_x$
Maximum value = 0.1830 ft (Element 1755 at Node 14834)
Minimum value = -0.1749 ft (Element 1 at Node 2010)
Figure E-15: PLAXIS 2D Sta. 311+00 (Drains) Excess Pore Pressure at 90% Consolidation

Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)

Maximum value = 0.4194 lb/ft$^2$ (Element 1943 at Node 11825)
Minimum value = -103.4 lb/ft$^2$ (Element 2722 at Node 19073)
Figure E-16: PLAXIS 2D Sta. 1687+50 (No Drains) Vertical Deformation at 90% Consolidation
Figure E-17: PLAXIS 2D Sta. 1687+50 (No Drains) Horizontal Deformation at 90% Consolidation

Total displacements $u_x$

Maximum value = 0.01265 ft (Element 54 at Node 373)
Minimum value = -0.02340 ft (Element 274 at Node 2199)
Figure E-18: PLAXIS 2D Sta. 1687+50 (No Drains) Excess Pore Pressure at 90% Consolidation

Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)

Maximum value = 0.05807 lbf/ft$^2$ (Element 277 at Node 4787)
Minimum value = -28.96 lbf/ft$^2$ (Element 416 at Node 1009)
Figure E-19: PLAXIS 2D Sta. 1688+00 (No Drains) Vertical Deformation at 90% Consolidation
**Figure E-20: PLAXIS 2D Sta. 1688+00 (No Drains) Horizontal Deformation at 90% Consolidation**

**Total displacements $u_x$**

Maximum value = 0.01029 ft (Element 57 at Node 005)
Minimum value = -0.01108 ft (Element 208 at Node 2017)
Figure E-21: PLAXIS 2D Sta. 1688+00 (No Drains) Excess Pore Pressure at 90% Consolidation

Excess pore pressures $p_{\text{excess}}$ (Pressure = negative)

- Maximum value = 0.2614 lb/ft$^2$ (Element 306 at Node 435)
- Minimum value = -10.14 lb/ft$^2$ (Element 388 at Node 869)
Appendix F: Dover Test Embankment PLAXIS Properties
Table F-1: Material properties used for PLAXIS models of the Dover Test Embankment

<table>
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<tr>
<th>Parameter</th>
<th>Name</th>
<th>Embankment</th>
<th>Sand Drainage Blanket</th>
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Appendix G: Settlement Hand Calculation
Using the embankment section at Sta. 307+00 of Exit 6N

- Embankment fill height = 12 ft
- Embankment unit weight = 121 pcf
- GWT @ depth of 4 ft

**Calculating settlement at mid-point of lower marine layer**

Solving for effective overburden stress at mid-point of lower marine clay layer (Depth = 27.5 ft)

\[
\sigma'_v = 7.5' \text{ (123 pcf)} + 3' \text{ (116 pcf)} + 17' \text{ (112 pcf)} - 23.5' \text{ (62.4 pcf)} = 1708.1 \text{ psf}
\]

Solving for pre-consolidation pressure using OCR assigned for PLAXIS model

\[
\sigma'_p = \text{OCR} \times (\sigma'_v) = 1.56(1708.1 \text{ psf}) = 2664.6 \text{ psf}
\]

Solving for vertical stress increase using Osterberg’s method (Poulos and Davis, 1974)

\[
\Delta \sigma_v = \frac{q_o}{\pi} \left[ \frac{(B_1 + B_2)}{B_2} \right] \left[ (\alpha_1 + \alpha_2) \right] - \frac{B_1}{B_2}
\]

where

- \( q_o = \gamma H \)
- \( \gamma = \) unit weight of embankment fill
- \( H = \) height of embankment
- \( \alpha_1 \) (radians) = \( \tan^{-1} \left( \frac{B_1 + B_2}{z} \right) - \tan^{-1} \left( \frac{B_1}{z} \right) \)
- \( \alpha_2 \) (radians) = \( \tan^{-1} \left( \frac{B_1}{z} \right) \)
- \( z = \) depth from surface to point of interest
Figure G-1: Embankment loading from Osterberg (1957)

\[
B_1 = 28 \text{ ft}, B_2 = 22 \text{ ft}
\]

Therefore: \(\Delta \sigma_v = 1309.0 \text{ psf}\)

Stress increase from sand drainage blanket = 1.5’ (108 pcf) = 162 psf

Since \(\sigma'_v + \Delta \sigma_v > \sigma'_p\), use Equation 3 from Chapter 2

\[
\delta_c = \frac{0.06}{1+1.20} 34 \log \left( \frac{2664.6}{1708.1} \right) + \frac{0.47}{1+1.20} 34 \log \left( \frac{1708.1 + 1309.0 + 162}{2664.6} \right) = 0.74 \text{ ft}
\]

Settlement at 90% consolidation = (0.9)0.74 ft = 0.66 ft

Settlement at closest point on PLAXIS model @ 90% consolidation = 0.74 ft