Finite Element Analysis of PV Drains for a Test Embankment on Soft Clay

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FINITE ELEMENT ANALYSIS OF PV DRAINS
FOR A TEST EMBANKMENT ON SOFT CLAY

BY

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B.S. Civil Engineering, University of New Hampshire, 2013

THESIS

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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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On May 4, 2015

Original approval signatures are on file with the University of New Hampshire Graduate School.
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LIST OF SYMBOLS

$C_c$ - compression index
$C_r$ - recompression index
$C_s$ - swelling index
$C_a$ - secondary compression index
$C'_a$ - modified secondary compression index
$c$ - cohesion
$c'$ - apparent cohesion
$c_h$ - coefficient of horizontal consolidation
$c_k$ - change of permeability
$c_v$ - coefficient of vertical consolidation
$d_e$ - equivalent soil cylinder diameter
$d_m$ - equivalent diameter of the mandrel
$d_{ms}$ - equivalent diameter of the mandrel shoe
$d_s$ - equivalent diameter of the smear zone
$d_w$ - band-shaped drain equivalent soil cylinder diameter
$E$ - Young's modulus of elasticity
$E_{50}$ - triaxial loading stiffness
$E_{50}^{ref}$ - reference stiffness modulus corresponding to $p_{ref}$
$E_D$ - dilatometer modulus
$E_i$ - initial tangent modulus
$E_{oed}$ - oedometer loading stiffness
$E_{ur}$ - triaxial unloading stiffness
$e$ - void ratio
$F(n)$ - vertical drain geometry factor
$F_r$ - piezocone normalized friction ratio
$F_s(n)$ - vertical drain smear geometry factor
$f$ - yield function
$f_s$ - piezocone friction sleeve resistance
$H_{dr}$ - drainage path length
$I' \quad -$ embankment influence factor

$I_c \quad -$ piezocone soil behavior type index

$I_D \quad -$ material index

$K \quad -$ earth pressure coefficient

$K_0 \quad -$ coefficient of lateral earth pressure at rest

$K_0^{nc} \quad -$ coefficient of lateral earth pressure for normally consolidated material

$K_D \quad -$ horizontal stress index

$k_h \quad -$ coefficient of horizontal permeability

$k_{h,ax} \quad -$ axisymmetric coefficient of horizontal permeability

$k_{h,ps} \quad -$ plane strain coefficient of horizontal permeability

$k_r \quad -$ remolded clay coefficient of permeability

$k_{r,ps} \quad -$ plane strain remolded clay coefficient of permeability

$k_v \quad -$ coefficient of vertical permeability

$M \quad -$ tangent of the critical state line

$m \quad -$ material stress dependency

$n \quad -$ drain spacing ratio

$OCR \quad -$ overconsolidation ratio

$P \quad -$ point load

$P_{\text{max}} \quad -$ maximum excess pore pressure

$P_{\text{stop}} \quad -$ minimum pore pressure designated to stop FEA calculation when reached

$p_0 \quad -$ dilatometer corrected “A” reading

$p_1 \quad -$ dilatometer corrected “B” reading

$p_2 \quad -$ dilatometer corrected “C” reading

$p' \quad -$ volumetric effective stress (mean effective stress)

$p_c \quad -$ preconsolidation stress

$p_{\text{excess}} \quad -$ excess pore water pressure

$p_{\text{ref}} \quad -$ reference confining pressure

$Q_{\text{trn}} \quad -$ piezocone normalized cone resistance

$q \quad -$ deviator stress (distortional stress)

$q_c \quad -$ cone resistance

$q_o \quad -$ embankment stress
$q_t$ - piezocone corrected tip resistance

$q_w$ - drain discharge capacity

$R_f$ - piezocone friction ratio

$r$ - radius

$r_e$ - equivalent soil cylinder radius

$r_s$ - equivalent radius of the smear zone

$r_w$ - equivalent drain radius

$S$ - smear zone ratio

$S_t$ - sensitivity

$s$ - drain spacing

$s_c$ - consolidation (primary) settlement

$s_i$ - immediate settlement

$s_s$ - secondary compression

$s_t$ - total settlement

$s_u$ - undrained shear strength

$T_h$ - time factor

$t$ - time

$U$ - degree of consolidation

$u$ - excess pore pressure

$u_o$ - hydrostatic pore pressure

$u_2$ - piezocone penetration pore water pressure at base of cone

$z$ - depth

$\gamma_T$ - total unit weight

$\Delta \sigma_v$ - vertical stress increase

$\delta \varepsilon_a$ - axial strain increment

$\delta \varepsilon_p$ - volumetric strain increment

$\delta \varepsilon_q$ - distortional strain increment

$\delta \varepsilon_r$ - radial strain increment

$\varepsilon_p$ - volumetric strain

$\varepsilon_q$ - distortional strain

$\varepsilon_v$ - vertical strain
κ - Cam-Clay swelling index
κ* - modified swelling index
λ - Cam-Clay compression index
λ* - modified compression index
μ* - modified creep index
ν - Poisson’s ratio
σ - principal stress
σa - axial stress
σ’a - effective axial stress
σh - total horizontal stress
σvo - overburden stress
σ’vo - effective overburden stress
σ’p - effective preconsolidation pressure
σr - radial stress
σ’r - effective radial stress
σ’vc - effective consolidation stress
τ - shear stress
φ - friction angle
φ’ - effective friction angle
ψ - dilatancy angle
ABSTRACT

FINITE ELEMENT ANALYSIS OF PV DRAINS
FOR A TEST EMBANKMENT ON SOFT CLAY

by

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University of New Hampshire, May, 2015

In 2012 the New Hampshire Department of Transportation constructed a test embankment with prefabricated vertical drains on top of soft marine clay in Dover, NH. The test embankment was built with variable drain spacing and embankment geometry in order to determine effective and efficient treatment for long term ground settlement. Findings from this study are to be implemented in future applications in the New Hampshire seacoast where soft marine clay is present. Using data collected from several in situ and laboratory tests, this thesis investigates the use of finite element analysis to predict total settlement and time rate of consolidation using the soft soil creep model within PLAXIS 2D. The model is validated by comparing finite element analysis results with geotechnical monitoring instrumentation installed within the test embankment at the time of construction. It was found that predictions of pore pressure dissipation and vertical displacements yield comparable results with that observed in the field.
1 INTRODUCTION

The New Hampshire Spaulding Turnpike in Newington and Dover faces heavy traffic congestion for morning and evening commuters as a result of its closely spaced interchanges, poor geometry, and narrow shoulders (NHDOT, 2009). Greatly exceeding its original intended capacity, the Spaulding Turnpike currently carries over 70,000 vehicles per day and a preliminary study of the road network yielded 2025 traffic projections to exceed 94,000 vehicles per day (NHDOT, 2009). The New Hampshire Department of Transportation (NHDOT) began studies in 2003 to expand the Spaulding Turnpike. The purpose of the project, designated as Newington-Dover 11238, is to improve mobility and safety along NH Route 16 between Exit 1 and the toll plaza north of Exit 6. The total cost of the project is estimated at approximately $213 million and includes a four lane traffic configuration in each direction, five interchanges, the widening and rehabilitation of existing bridges, and a park and ride facility.

The NHDOT is reconfiguring the onramp at Exit 6 from Route 4 onto the Spaulding Turnpike/Route 16 South. As a part of the reconfiguration, highway embankments will need to be constructed to serve as the foundation for the new road network. Based on previous borehole data from the surrounding area, a thick layer of silty clay was suspected and later confirmed to be approximately 60 ft thick. This clay is part of a deposit known as the Presumpscot Formation. Soil properties attributed to the Presumpscot Formation have been known to be problematic in geotechnical construction, mostly due to having low shear strength and bearing capacity as well as
high compressibility and excessive settlement characteristics over long periods of time. Prior to constructing the Exit 6 on-ramp, it was essential to stabilize the existing soft clay foundation in order to avoid differential settlement of the overlying highway system.

Due to the vast presence of the Presumpscot formation in much of the coastal regions of New Hampshire and Maine, an extensive testing program was developed by the NHDOT and the University of New Hampshire (UNH) to evaluate the properties and behavior of the clay. Several in situ testing methods including flat plate dilatometer test (DMT), field vane shear test (FVT) and piezocone test (CPTu) as well as laboratory testing were performed to characterize the site and estimate geotechnical properties of the marine deposit prior to the construction of a test embankment. A majority of the testing and site characterization was performed by former UNH graduate student Amy Getchell as a part of her Master’s research (Getchell, 2013). A test fill embankment has since been constructed, designated at the Dover Test Embankment (DTE).

In geotechnical engineering, embankments can be used as preloads for the underlying soil. The applied load consolidates the foundation soils such that the final as-built superstructure can be constructed. Constructing roadways and permitting traffic flow over compressible soil prior to allowing consolidation to occur can cause permanent deformations and/or failure (e.g. pavement cracking, slope instability, etc.), potentially costing millions of dollars in roadway and motor vehicle repair work.
Figure 1.1: Map of the test embankment at Dover Point in Dover, NH (Google Maps, 2014)
The new alignment of the RT 16 Exit 6 SB On-Ramp lies in an approximate north to south direction and is located between RT 16 and RT 4 as shown in Figure 1.1. Prior to final configuration, the Dover Test Embankment was constructed over the length of the on-ramp to serve as a physical research model for determining an effective and efficient treatment for the long-term ground settlement expected in the Newington-Dover project 11238 and for other similar projects in the New Hampshire seacoast.

1.1 Use of Vertical Drains

The test embankment for the Newington-Dover project was constructed using prefabricated vertical (PV) drains to help accelerate the settlement of the underlying clay. Consolidation of clay is primarily dependent on the dissipation of pore water pressure generated from the applied load. As pressure dissipation occurs, the water is released from the void space in the soil leading to settlement and tighter and stronger soil structure. Total settlement or consolidation may take months or years to occur as it depends on the permeability of the underlying soils. Installing PV drains provides an artificial drainage path which significantly shortens the time for consolidation, as opposed to the time required for natural drainage to occur. By shortening the time for total settlement to occur, construction productivity increases. The test embankment for the Newington-Dover project was constructed with five different segment geometries with varying PV drain spacing. Each segment measures approximately 200 ft long and has a fill height of 12 ft high with side slopes of 2H:1V, with the exception of Segment 4 which was chosen to have a fill height of 18 ft and side slopes of 1.5H:1V. Segment 1 has a triangular drain spacing of 6 ft and Segment 5 has a drain spacing of 14 ft while all other segments were constructed with 10 ft drain spacing. In addition, Segment 3
drains were installed at a shallower depth in comparison with the other segments. Actual cross-sectional geometry varies based on the width of each segment.

1.2 Scope and Objective of Study

The Dover Test Embankment (DTE) was constructed between October, 2012 and February, 2013 to determine effective and efficient treatment for significant and long-term ground settlement expected in the future work of the road network expansion. The University of New Hampshire (UNH) performed field vane, flat plate dilatometer, and piezocone testing and collected soil samples for laboratory consolidation testing prior to construction in order to characterize the site and estimate geotechnical properties of the marine clay deposit (Getchell, 2013). The in situ data for the highly compressible marine clay deposit was then used to build a finite element model using PLAXIS 2D Anniversary Edition (AE).

The primary focus of this investigation is to develop an FEA model of the DTE for the prediction of total settlement and settlement rate in the field using PLAXIS 2D. For the purpose of validating the model, all settlement predictions are compared with the results from DTE settlement monitoring instrumentation installed at the time of construction. In summary, in situ testing was performed to gather empirically based data, to be used in a numerical model, which employs constitutive theory to find total and rate of settlement, which is compared to settlement data collected from a full scale physical model.
1.3 Thesis Outline

A review of consolidation theory, vertical drains, and constitutive modeling theory is presented in Chapter 2 to provide context for later chapters. Chapter 3 reviews the in situ testing methods performed by Getchell (2014). Chapter 4 describes the methodology of obtaining site stratigraphy and material properties used for determining settlement predictions in PLAXIS and presents data collected from geotechnical field instrumentation. Chapter 5 introduces the reader to the PLAXIS software and uses the soil stratigraphy and material properties from Chapter 4 to predict dissipation of pore pressure as well as vertical and horizontal deformation. FEA predictions are then compared with field instrumentation. Chapter 6 provides a summary of research, conclusions and discusses potential future research.
2 LITERATURE REVIEW

2.1 Consolidation Theory

When materials are loaded or stressed, they deform or strain. Some materials deform quickly and immediately upon loading, while others respond slowly, over time, as is the case with clay soils. The stress-strain relationship of linear elastic materials exhibits behavior where stress and strain occur linearly with one another; furthermore, if the load is removed, the material returns to its original shape (Holtz et al., 2011). Soil, once deformed, does not return to its original shape when unloaded; rather, it retains some strain or permanent deformation known as plastic behavior. Soil is also considered a non-conservative material, meaning that when it is loaded and then unloaded, the stress history is preserved, influencing the behavior of the soil if it is later reloaded. Soils are complex and prove to be among the most difficult to model due to their nonlinear stress-strain relationship, time-dependent response to loading, stress history, and complicated part elastic, part plastic behavior (Holtz et al., 2011).

2.1.1 Settlement

Settlement is the measure of total vertical deformation at the ground surface. With an increase in load, such as the placement of an embankment, movement will be downward; however, in cases where the load is decreased, as when an excavation or cut is made, an upward movement or swelling can occur. Total settlement \( (s_t) \) is made up of three different contributing modes of settlement summarized in Equation 2.1. Immediate settlement \( (s_i) \) is estimated using elastic theory and more prominent in coarse-grained soils having high permeability. Consolidation or primary settlement \( (s_c) \)
is a time-dependent process that occurs in saturated fine-grained soils with low permeability and is governed by the rate of pore water drainage (Holtz et al., 2011). Secondary compression \((s_s)\) is also a time-dependent process occurring after primary consolidation where settlement continues to occur under a constant effective stress. With fine grained soils such as silts and clays, \(s_c\) and \(s_s\) are the predominant modes contributing to total settlement, and will be the primary focus of this investigation. Immediate settlement was not considered.

\[
s_t = s_t + s_c + s_s
\]

### 2.1.2 Stress Increase under an Embankment

Before total settlement can be calculated, it is required that the vertical stress increase in the soil mass due to applied load be estimated. In 1885 Boussinesq developed mathematical relationships for determining the normal and shear stresses at any point inside homogeneous, elastic, and isotropic half-space medium due to a concentrated point load located at the surface (Das, 2011). With an applied point load, \(P\), the vertical stress increase \((\Delta \sigma_v)\) of some point \(A\), can be found using Equation 2.2 as depicted in Figure 2.1.

\[
\Delta \sigma_v = \frac{3P}{2\pi z^2 \left[ 1 + \left( \frac{r}{z} \right)^2 \right]^\frac{3}{2}}
\]

where

\[
r = \sqrt{x^2 + y^2}
\]

\(x, y, z = \text{coordinates of the point A}\)
By the same principle, vertical stress increase can be found under an embankment loading (Figure 2.2) using Equation 2.3 (Note that $\alpha_1$ and $\alpha_2$ are calculated in radians).

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

2.3

where

$q_o = \gamma H$

$\gamma = $ unit weight of the embankment fill

$H = $ height of embankment

$\alpha_2 = \tan^{-1} \left( \frac{B_1}{z} \right)$

$z = $ depth from surface to point of interest
A simplified version of Equation 2.3 results in Equation 2.4, where vertical stress increase can be calculated based solely on the applied embankment stress \( q_o \) and influence factor \( (l') \). Figure 2.3 (after Osterberg, 1957) is a simplified diagram for finding the influence factor, to be used with Equation 2.4.

\[
\Delta \sigma_v = q_o l'
\]

where

\( l' \) is a function of \( \frac{B_1}{z} \) and \( \frac{B_2}{z} \)
2.1.3 Primary Consolidation

Primary consolidation, as discussed in section 2.1.1, is a time-dependent process where settlement occurs in direct correlation with an increase in load. Using one-dimensional consolidation theory, $s_c$ can be calculated for clay using Equations 2.5, 2.6, and 2.7 (Das, 2011). Refer to section 2.1.5 for an explanation on how to obtain compression index, swelling index, and preconsolidation pressure from a consolidation test.
(for normally consolidated clay)

\[ s_c = \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_{vo}} \]

(2.5)

(for overconsolidated clay where \( \sigma'_{vo} + \Delta\sigma'_v < \sigma'_p \))

\[ s_c = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_{vo}} \]

(2.6)

(for overconsolidated clay where \( \sigma'_{vo} < \sigma'_p < \sigma'_{vo} + \Delta\sigma'_v \))

\[ s_c = \frac{C_s H_c}{1 + e_o} \log \frac{\sigma'_p}{\sigma'_{vo}} + \frac{C_c H_c}{1 + e_o} \log \frac{\sigma'_{vo} + \Delta\sigma'_v}{\sigma'_p} \]

(2.7)

where

\( s_c \) = primary consolidation settlement

\( C_c \) = compression index

\( C_s \) = swelling index

\( H_c \) = thickness of clay layer

\( e_o \) = initial void ratio

\( \sigma'_p \) = preconsolidation pressure

\( \sigma'_{vo} \) = average initial effective stress

\( \sigma\Delta'_v \) = average increase in effective stress due to loading

### 2.1.4 Secondary Compression

Secondary compression is also a time-dependent process and occurs once primary consolidation is completed. In the case of constructing an embankment, during primary consolidation the soil matrix partially relies on the slow decrease in pore water pressure to assist in sustaining the embankment load. Once primary consolidation is complete,
excess pore water pressure is fully dissipated and the effective stress increase caused by the embankment load has been fully transferred to the soil skeleton. Because the soil skeleton can no longer rely on pore water pressure for structural support, additional secondary compression or creep settlement can occur due to constant loading over time. Secondary settlement is typically a small fraction of the total settlement; however, the fraction can still be substantial enough to cause problematic settlement issues in the long term. Secondary compression settlement \( s_s \) can be calculated using Equations 2.8, 2.9, and 2.10 (Das, 2011).

\[
s_s = C'_a H_c \log \left( \frac{t_2}{t_1} \right) \tag{2.8}
\]

\[
C'_a = \frac{C_a}{1 + e_p} \tag{2.9}
\]

\[
C_a = \frac{\Delta e}{\log \left( \frac{t_2}{t_1} \right)} \tag{2.10}
\]

where

- \( C'_a \) = modified secondary compression index
- \( H_c \) = thickness of clay layer
- \( t_1 \) = time duration to complete primary consolidation
- \( t_2 \) = total desired time duration
- \( C_a \) = secondary compression index
- \( e_p \) = void ratio at end of primary consolidation
- \( \Delta e \) = change in void ratio
Secondary compression index \((C_o)\) can be found from a consolidation test or alternatively, using empirical relationships such as that provided by Mesri (1973) as shown in (Figure 2.4).

![Graph showing Modified secondary compression index versus natural water content](image)

**Figure 2.4:** Modified secondary compression index versus natural water content, after Mesri, 1973 (Holtz et al., 2011)
2.1.5 One-Dimensional Consolidation Testing

One-dimensional laboratory consolidation tests can be conducted (ASTM D2435) to determine the consolidation settlement caused by various incremental loadings. An undisturbed (minimally disturbed), fully saturated, cylindrical soil specimen is incrementally and vertically loaded, while measuring vertical strain. Between loading steps pore water pressure is allowed to fully dissipate. After graphing the effective consolidation stress \((\sigma'_{vc})\) versus vertical strain \((\varepsilon_v)\) or void ratio \((e)\) to obtain a consolidation curve, soil properties such as preconsolidation pressure \((\sigma'_p)\), compression index \((C_c)\), and swelling index \((C_s)\) can be obtained graphically.

2.2 History and Development of Vertical Drains

The primary purpose of vertical drains is to accelerate the consolidation time by providing shortened radial drainage paths. Vertical drains are most commonly suited for fine grained, inorganic high water-content, low strength soils (Holtz et al., 1991). Since most natural clay deposits are inherently anisotropic with respect to their flow properties, the coefficient of permeability for horizontal flow \((k_h)\) is typically higher than flow in the vertical direction \((k_v)\). Therefore, by providing radial drainage, vertical drains offer an additional advantage to accelerating consolidation.

The idea of vertical drains originated in the U.S. with D. E. Moran who in the 1925 proposed the use of a sand drain for deep soil stabilization. In 1926, Moran obtained a U.S. patent for the concept and the first practical sand drain was constructed in California a few years later.
In the mid 1930’s Walter Kjellman, a Swedish engineer, developed the first prefabricated drain made out of cardboard. Wager, who worked to develop the patented prefabricated drain with Kjellman, reported there were problems in the early stages concerning rapid deterioration of the cardboard drains, particularly at the top of the drain/clay layer interface. In order to address the deterioration, the cardboard was treated with a retarding agent; however, the retardant was noted to cause a reduction in drain permeability from about $10^{-3}$ to $10^{-5}$ cm/s, significantly reducing the effectiveness of the drain (Holtz et al., 1991).

Kjellman drains have been occasionally used in Europe and Japan in the past 60 years; however, leading up until the early 1970’s most vertical drains used in practice were sand drains (Holtz et al., 1991).

In 1971, Wager adapted Kjellman’s prefabricated design into a drain with a core made entirely of grooved polyethylene plastic, thus eliminating the concern of rapid deterioration. Later adaptations incorporated paper and woven textile filters around the plastic core. In recent years, various types of prefabricated drains have become readily available and extensively used in soft soil improvement techniques.

2.2.1 Types of Vertical Drains

Various types of vertical drains are in use today, but all are built on the same principle and are comprised of a central grooved or channeled core encased within a filtering sheath. The two most common types are sand drains and prefabricated drains; both are
briefly discussed in the following sections, though the focus of this thesis is with the latter.

### 2.2.2 Sand Drains

With their low cost and simple design, driven, displacement sand drains are still used in many geotechnical and construction projects today. However, the method of driving a mandrel down into the soil causes high disturbance to the surrounding soil in turn causing reduction in shear strength to the foundation clay, and forming a highly disturbed smear zone along the perimeter of the drain side walls (Holtz et al., 1991). These effects are discussed later in 2.4.1.

Sand drains can also be installed using a jetted technique which minimizes the disturbance and smearing effect. A continuous-flight hollow-stem auger method can also be used to install sand drains. Disturbance of the soil does occur more so than jetted drains but to a lesser degree than the driven, displacement method. Based on laboratory studies by Singh and Hattab (1979), closed-end cross-shaped mandrel and jetted installation methods rate among the most efficient based on radial drainage and required drain spacing (Holtz et al., 1991).

### 2.2.3 Prefabricated Drains

Prefabricated drains, or wick drains, typically come in rolls of rectangular strips. While sand drains range from 0.5 ft to 1.5 ft in diameter (0.15 m - 0.5 m), prefabricated drains, have much smaller dimensions where the equivalent diameter ranges from about 2 in. to 6 in. (0.06 m - 0.15m) (Holtz et al., 1991).
The typical method of installation for prefabricated drains is by closed-end mandrel (Figure 2.5). While the mandrel is above the ground surface, the drain is fed through the mandrel and attached to a disposable shoe which acts as an anchor and prevents the drain from retracting back up the mandrel during installation. To avoid soil entry and clogging, the slack is then taken back until the shoe is seated against the bottom of the mandrel to make a closed-end. The mandrel is then simply pushed or sometimes vibrated through the soil to the desired depth, retracted, and the PV drain is cut at the ground surface. The process is then repeated sequentially over the intended treatment area.

Disturbance associated with prefabricated drain installation can be attributed to the sensitivity of the clay, and size of the mandrel in addition to the size and shape of shoe used (Holtz et al., 1991). Various disposable shoes are used, some shown in Figure 2.5. Figure 2.5a shows a V-shaped piece of sheet metal and Figure 2.5b shows a plastic tube. The amount of disturbance or smear is significantly less than that associated with conventional sand drain placement due to the smaller volume displacement during installation (Holtz et al., 1991). The use of prefabricated drains has increased in recent years, likely due to their simple design, quick installation, efficiency, and overall low unit cost.
Figure 2.5: Typical shoes used with prefabricated drain mandrels (Holtz et al., 1991)
2.3 Radial Drainage

The rate of consolidation depends on the length squared of the longest path traveled by a drop of water in order for pore water pressure to dissipate. The length is known as the drainage path \( (H_{dr}) \). In a vertically drained system there are two types. A doubly drained system assumes a permeable layer is both above and below the clay layer, and thus the \( H_{dr} \) is equal to half the thickness of the clay layer. In a single drained system only one permeable layer is present, typically at the top of the clay layer, and \( H_{dr} \) is taken as the full thickness of the clay layer. In a system with vertical drains, the drainage path is no longer only vertical and a function of the clay thickness, but primarily radially and a function of the drain spacing which is typically much less than the thickness of the clay layer. Additionally, in the case of anisotropic soils where horizontal permeability and horizontal coefficient of consolidation are higher than their vertical counterparts, radial drainage will significantly govern the rate of consolidation.

While drain spacing installations can be triangular (Figure 2.6a) or square (Figure 2.6b), in both cases, the boundary conditions of the problem to be analyzed refer to an equivalent soil cylinder of diameter, \( d_e \) (Figure 2.6c) (Holtz et al., 1991). Equations 2.11 and 2.12 show how to calculate \( d_e \) given the drain placement pattern. Hansbo (1979) found that band-shaped and circular drains lead to the same degree of consolidation, provided their circumferences are equal. Therefore, when developing a band-shaped drain equivalent cylinder, the equivalent diameter of the drain \( (d_w) \) can be found by Equation 2.13.
(for triangular pattern)

\[ d_e = 1.05s \]

(for square pattern)

\[ d_e = 1.128s \]

where

\[ s = \text{drain spacing} \]

\[ d_w = \frac{2(a + b)}{\pi} \]

where

\[ a = \text{width of band-shaped drain} \]

\[ b = \text{thickness of band-shaped drain} \]

Consolidation with the use of vertical drains uses Terzaghi's one-dimensional consolidation theory with the possibility of two different boundary conditions. Free vertical strain assumes the vertical surface stress is constant during consolidation resulting in non-uniform surface displacements while equal vertical strain assumes the vertical surface displacement is constant within the drained area resulting in non-uniform vertical stress at the surface (Holtz et al., 1991). Field observations found that actual boundary conditions are a mixture of the two. Barron's (1948) free and equal strain solutions indicated both yield almost the same degree of consolidation when a drain spacing ratio \((n)\) greater than 5 and time factor \((T_n)\) greater than 0.1 is satisfied.
Therefore, the simpler equal strain solution for excess pore water pressure at some time, \( t \), and some radius from the drain, \( r \), is preferable (Equation 2.14).

\[
\begin{align*}
  u &= \frac{u_o}{r_e^2 F(n)} \left[ r_e^2 \log_e \left( \frac{r}{r_w} \right) - \frac{(r^2 - r_w^2)}{2} \right] \exp(\lambda) \\
  \lambda &= \frac{-8T_h}{F(n)} \\
  T_h &= \text{the horizontal time factor} = \frac{c_h t}{d_e^2} \\
  n &= \text{drain spacing ratio} = \frac{d_e}{d_w} \\
  F(n) &= \frac{n^2}{n^2 - 1} \log_e(n) - \frac{3n^2 - 1}{4n^2}
\end{align*}
\]

where:

\( u \) = excess pore pressure
\( u_o \) = initial excess pore pressure
\( r_e \) = equivalent soil cylinder radius = \( \frac{d_e}{2} \)
\( r_w \) = equivalent drain radius = \( \frac{d_w}{2} \)
\( \lambda \) = \( \frac{-8T_h}{F(n)} \)
\( T_h \) is the horizontal time factor = \( \frac{c_h t}{d_e^2} \)
\( n \) = drain spacing ratio = \( \frac{d_e}{d_w} \)
\( F(n) = \frac{n^2}{n^2 - 1} \log_e(n) - \frac{3n^2 - 1}{4n^2} \)

Similarly, the average degree of consolidation with respect to radial flow at time, \( t \), is shown in Equation 2.15.

\[
\overline{U_h} = 1 - \exp \left[ \frac{-8T_h}{F(n)} \right]
\]

where:

\( \overline{U_h} \) is the average degree of consolidation with respect to radial flow
Figure 2.6: Drain patterns and equivalent cylinder (Holtz et al., 1991)
2.4 Factors Influencing the Vertical Drain Efficiency

2.4.1 Smear Zone

The radial drainage theory discussed in Section 0 assumes that the installation procedure does not change the properties of the soil; however, in practice drain installation does disturb the soil to a degree depending on several factors including, sensitivity and macrofabric of the soil (Holtz et al., 1991). Barron (1948) and Hansbo (1979, 1981) investigated the effect of disturbance and assumed a smear zone of clay with diameter, \( d_s \), around the equivalent drain diameter, \( d_w \) (Figure 2.7). The ring of smeared clay generated by \( d_s \) has a lower coefficient of permeability due to remolding \( (k_r) \) compared to the \( k_h \) of the undisturbed clay. This creates a new boundary condition between the undisturbed and smeared soil and is addressed by changing the vertical drain geometry factor, \( F(n) \), to account for the smear zone (Equation 2.16).

\[
F_s(n) = \log_e \left( \frac{n}{S} \right) - 0.75 + \left[ \frac{k_h}{k_r} \right] \log_e (S) \tag{2.16}
\]

where

\[
S = \text{smear zone ratio} = \frac{d_s}{d_w}
\]

\( k_r = \text{remolded clay coefficient of permeability} \)
2.4.2 Size and Shape of the Mandrel

The size and shape of the mandrel (Figure 2.8) and driving shoe is directly related to the amount of soil disturbance. From similar effects of pile driving (e.g. Orrje and Broms, 1967) and sand drains (e.g. Holtz and Holm, 1972; Massarsch and Kamon, 1983), the degree of remolding reduces with increasing radial distance from the drain. Thus, the $k_r$ value of the clay increases with increasing radial distance away from the drain until
$k = k_h$ at a distance equivalent to the radius of the smear zone ($r_s$). To simplify, an average value of $k_r$ is assumed within the smear zone as shown in Figure 2.7. Although there is a lack of theoretical and experimental evidence, Levadoux and Baligh (1980) completed a comprehensive analysis of pore pressure developed during cone penetration and found excess pore pressure was affected even at a radial distance of two times the mandrel diameter ($d_m$) (Holtz et al., 1991). Based on these limited data, it is a conservative to assume significant disturbance thus estimating $d_s$ as 2.5-3 times the equivalent diameter of the mandrel used in installation (Holtz et al., 1991).

Figure 2.8: Mandrel used for DTE PV drain installation
2.4.3 Installation Method

The installation of vertical drains creates significant stress changes in the surrounding soil, but the smear zone is also influenced by the method of installation. With the jetted installation technique, disturbance is minimized, resulting in a smear zone ratio ($S$) and $k_s/k_r$ ratio of approximately equal to 1. In low-displacement prefabricated drain installation (Figure 2.9), Hansbo (1981) recommends using a value of $S=1.5$.

Figure 2.9: Typical PV drain installation
2.5 Evaluation of Design Parameters

With or without vertical drains, the governing parameters for compression and rate of consolidation of soft cohesive soil are the coefficient of permeability, and vertical coefficient of consolidation \((c_v)\), both affected by stress history (preconsolidation pressure, \(\sigma_p\), and overconsolidation ratio, OCR). With drains, additional parameters for consideration include, the equivalent diameter of the smear zone \((d_s)\), the permeability coefficient of the remolded clay \((k_r)\) within the smear zone, and discharge capacity of the drain \((q_w)\) as it relates to time \((t)\), and change in total horizontal stress \((\sigma_h)\) (Holtz et al., 1991). For prefabricated band-shaped drains, where radial drainage governs, the horizontal coefficient of consolidation \((c_h)\) of the soil is essential for design, as well as the smear effects and discharge capacity.

2.6 Methods of Analysis

Engineering itself is primarily concerned with finding solutions to problems, and in order to solve a complex problem, engineers make many assumptions and simplifications. A model, in many ways, is a simplification of reality which is why engineering is almost synonymous with modeling. Whether it is empirical, theoretical, numerical, constitutive, or physical, all methods of modeling are used by engineers in various ways. The challenge to the engineer is to understand the differences between all models and most importantly to find an appropriate level of simplification, thus being able to identify the key features needed from those that can be omitted in order to solve the problem.

2.6.1 Empirical Models

True empirical models are purely based on experience. Many engineering equations used today are based on empirical formulas gathered from years of data collection and
observation. If applied correctly these equations can be useful in design; however, in geotechnical engineering the problem will always be the tremendous variability of soil from one geographical location to another. The advantage and disadvantage, depending on the view, is that empirical solutions provide satisfactory answers even though there may not be any fundamental or true understanding. For this reason, empirical models are useful in allowing design to continue in the interim of finding a more suitable theoretical explanation or solution.

### 2.6.2 Theoretical Models

Theoretical models can be seen as well thought-out solutions using known engineering laws to explain certain material behavior. In the case of geotechnical engineering, models based on theory often require understanding of observed soil behavior and massaging it into the framework of the theoretical model to obtain an approximate theoretical solution (Muir Wood, 2004). This can be done by modifying the observed boundary conditions, or making use of a numerical solution.

### 2.6.3 Numerical Modeling

Obtaining exact solutions for many engineering problems can prove to be quite difficult. This may be attributed to extremely complex differential equations controlling the problem, or due to intricate boundary and initial conditions. To address these complexities, numerical analysis and approximations may be used. In contrast to analytical solutions which show the exact behavior of a system at any given point within the system, numerical solutions approximate exact solutions only at discrete points, called nodes (Sathananthan, 2005). The first step in the numerical procedure is
discretization, where the material is divided into a number of small sub-regions with nodes.

Finite element analysis (FEA) is a numerical method that uses integral formulations to create a system of algebraic equations. An approximate continuous function is assumed to represent the solution for each element. The complete solution is then generated by assembling the individual solutions in order to satisfy overall stability of the system. The deformation response of each element is defined by element shape, the displacement variation with each element, and the constitutive model (stress-strain behavior) employed to represent element behavior (Sathananthan, 2005). The application of numerical modeling in geotechnical engineering has greatly increased over the past few decades due to significant advances and availability of computing power and software such as PLAXIS. This program is useful in solving any number of complex problems and focuses on geotechnical engineering solutions and was used for the numerical modeling in this research work.

2.6.4 Constitutive Modeling

In order to model a system numerically, there must be an accepted constitutive relationship that predicts the interaction between stresses and strains. Constitutive modeling attempts to form statements of equilibrium and compatibility that relate stress and strain. The constitutive model of choice is up to the discretion of the modeler but is typically chosen based on judgment and prior experience. Again, it is important to understand the difference and applicability of each constitutive model as it applies to specific soil types and loading conditions. Some models are more widely used than
others. For example, the Linear Elastic, Mohr Coulomb, Hardening-Soil, Soft Soil, and Cam Clay models are constitutive models that have undergone significant research and are widely accepted as suitable models to characterize the complex relationships between stress and strain in many soils (Muir Wood, 2004). These models are available within Plaxis 2D AE.

### 2.6.5 Physical Models

Physical modeling is performed in order to validate theoretical or empirical solutions. The simplification and uncertainties of numerical and theoretical models can sometimes diverge from what is observed in the field. By constructing a small-scale or full scale physical model and implementing extensive observation methods, theoretical or empirical solutions can be supported, disproved, or advanced. For example, when analyzing settlement under a load, laboratory testing of small elements of soil (e.g. triaxial) or in situ field testing (e.g. cone penetration testing) yield test specific soil properties and form a basis of how to model the soil response. Constructing a full-scale model and observing displacements and pore pressures allow engineers to compare the observed data to theoretical predictions. If the theoretical approach is found to be adequate, the model can be applied independently to future design on similar soils; or, if the theory does not fit the field data the physical model can independently serve as a basis for approximating future work.

### 2.7 PLAXIS

PLAXIS 2D is a two-dimensional finite element program that was originally developed at Delft University of Technology in the Netherlands. In conjunction with the Dutch Ministry of Public Works and Water Management, the program was initially developed for
analysis of river embankments on soft soils in the lowland regions of Holland (Brinkgreve et al., 2014). Since its development in 1987, the PLAXIS company was formed in 1993, PLAXIS 2D was released in 1998, and the software has developed into a useful tool for a variety of geotechnical applications. As of 2010, PLAXIS released a 3D version and continues to evolve with geotechnical theory and application.

2.7.1 PLAXIS 2D AE

Upon opening the program the user is presented with the Project Properties window in which the user can name the project. More importantly, under the Model tab, the user can define the type of model (plane strain or axisymmetric) and choose between using a 15-node or 6-node triangular element. Additionally, units and contour window limits may be set. Once the project properties are defined, the user may create the soil model using the following modes: Soil, Structures, Mesh, Water conditions, and Staged construction.

2.7.2 PLAXIS Soil and Structures

In the Soil mode, boreholes can be used to configure the soil stratigraphy, water conditions and material sets. Boreholes may be placed anywhere within the contour window. The Material sets tab allows the user to name the material and choose the material model, drainage type and input parameters such as material stiffness and strength, unit weight, void ratio, permeability and overconsolidation ratio. In a similar fashion, structural components such as tunnels, vertical drains, embankments, foundations, retaining walls, and cuts and fills may be defined in the Structures mode. These two modes are referred to as the geometry modes. When the geometry model is complete, the finite element model mesh can be generated.
2.7.3 PLAXIS Mesh

The Mesh mode allows the user to define the mesh properties for discretization and transformation of the model geometry into a finite element model. In terms of element distribution, the mesh options available are very coarse, coarse, medium, fine, and very fine. As the mesh becomes finer, more elements are introduced and distributed within the model geometry. Finer mesh can sometimes produce more precise results; however, it will result in longer computation times. The mesh can also be coarsened or refined in user defined areas as needed.

The generated mesh, in PLAXIS, consists of triangular elements with the option of 6-node or 15-node elements (Figure 2.10) defined in Project Properties. By default, PLAXIS uses a 15-node triangular element as it provides a fourth order interpolation for displacements and involves the integration of twelve stress points for increased accuracy (Brinkgreve et al., 2014). In contrast, 6-node triangular elements may be used, providing second order interpolation for displacements and involves the integration of three stress points.
Figure 2.10: Positions of nodes and stress points in triangular elements (Brinkgreve et al., 2014)

2.7.4 PLAXIS Staged Construction

The *Staged construction* mode allows the user to activate and deactivate parts of the geometry model and modify properties in sequential phases, representing stages of construction for calculation. Once appropriate phases have been set, element nodes or stress points must be selected for points of interest, and then the model may be calculated. During calculation, the progress of each calculation phase is continuously updated showing all relevant information, including the $P_{\text{max}}$ curves and iteration process, in real time. $P_{\text{max}}$ reflects the change in maximum excess pore pressure over time for each stage of construction. Once the calculation is complete, the results of each phase may be viewed in the *Output* window of PLAXIS. The user may specify to view displacements, stresses, pore pressures, or excess pore pressures visually for the
entire geometry or the specified points of interest may be selected and graphed (e.g. vertical displacement vs. time) in the Curves manager.

### 2.8 Constitutive Models for Soils Available in PLAXIS 2D AE

When a material is loaded (stress is applied), it deforms accordingly (strain occurs). This relationship is known as the stress-strain relationship and is unique to all materials. Some materials, when unloaded, retain their original shape (elastic behavior), while others remain permanently deformed (plastic behavior). Constitutive models aid in explaining material behavior by theorizing the stress-strain response of materials. Hooke’s law of linear, isotropic elasticity is often thought of as the simplest stress-strain relationship, as it involves only two input parameters, Young’s modulus of elasticity (E) and Poisson’s ratio (ν), to explain material behavior. Contrary to its simplicity, linear elasticity is not sufficient in modeling soil behavior. Soil is nonlinear and can be complex to model, but several constitutive models have been developed, simplifying the problem by incorporating specified degrees of linearity and nonlinearity, in order to obtain a reasonable estimate of its behavior. A typical stress-strain diagram of soil is shown in Figure 2.11. This section describes basic soil models available within PLAXIS and explains the progression into the more advanced and applicable soil models used in this investigation.
2.8.1 Triaxial Stress and Strain Variables

Since constitutive models are based on the stress-strain relationship of materials, it is important to characterize the stresses applied to elements of soil. The triaxial test provides an axial symmetric loading condition and is one of the most common and widely used test methods to classify the mechanical behavior of soil (Muir Wood, 2004). Although many geotechnical applications do not exhibit the axial symmetry implied in triaxial testing, the test is regarded as an appropriate simplification. Combined with the ample data available, the triaxial test serves as a typical means of explaining constitutive models. Figure 2.12 shows a general element of soil subjected to all six components of stress where normal stresses (σ) and shear stresses (τ) are represented.
in their respected x, y, and z planes. In contrast, Figure 2.13 shows the axisymmetric stress conditions of a triaxial test in terms of axial stress ($\sigma_a$) and radial stresses ($\sigma_r$).

**Figure 2.12:** Soil element subjected to general state stress (Muir Wood, 2004)

**Figure 2.13:** Triaxial testing state of stress (Muir Wood, 2004)
Certain stress and strain variables must be discussed prior to reviewing the constitutive models. Stress strain relationships are often explained by plotting material behavior in stress-strain space; however, more complex constitutive models plot material behavior according to triaxial parameters of stress and strain. Using axial and radial strain increments, $\delta \varepsilon_a$ and $\delta \varepsilon_r$ respectively, volumetric (Equation 2.17) and distortional (Equation 2.18) strains increments can be found. The corresponding axial and radial effective stresses, $\sigma'_a$ and $\sigma'_r$, are similarly used to define volumetric effective stress (Equation 2.19) and distortional stress (Equation 2.20). Volumetric effective stress ($p'$) and distortional stress ($q$) can also be taken as the mean effective stress and deviator stress, respectively. Again, these variables will be useful in understanding the soil response with respect to the applied constitutive models discussed in the following sections.

\[
\delta \varepsilon_p = \delta \varepsilon_a + 2\delta \varepsilon_r \quad \text{(2.17)}
\]

\[
\delta \varepsilon_q = \frac{2}{3}(\delta \varepsilon_a - \delta \varepsilon_r) \quad \text{(2.18)}
\]

\[
p' = \frac{1}{3}(\sigma'_a + \sigma'_r) \quad \text{(2.19)}
\]

\[
q = \sigma_a - \sigma_r \quad \text{(2.20)}
\]

where:

$\delta \varepsilon_p$ is the volumetric strain increment

$\delta \varepsilon_q$ is the distortional strain increment
2.8.2 Linear Elastic Model

The Linear Elastic (LE) model is based on Hooke’s law or isotropic elasticity and involves only two input parameters to describe soil elasticity: Young’s modulus of elasticity \( E \) and Poisson’s ratio \( \nu \). This model is not intended for soil; rather, for the use of modeling stiffer materials that may be common with geotechnical practice, such as steel struts for bracing, concrete foundation walls, or large rock masses. Figure 2.14 shows the linear stress-strain relationship associated with elastic materials.

![Figure 2.14: Linear stress-strain relationship for compression of an elastic element (Muir Wood, 2004)](image-url)
2.8.3 Mohr-Coulomb Model

The Mohr-Coulomb model follows an elastic perfectly-plastic stress-strain relationship as shown in Figure 2.15, and involves five input parameters. Young’s modulus ($E$) and Poisson’s ratio ($\nu$) are used as elastic stiffness parameters, while friction angle ($\varphi$), dilatancy angle ($\psi$), and cohesion ($c$) are used as the plastic strength parameters.

![Diagram of Mohr-Coulomb model]

*Figure 2.15: Typical elastic perfectly-plastic response and tangent variation for soil (Muir Wood, 2004)*

Comparing Figure 2.15 with Figure 2.11, one can understand how, on a stress strain curve, the Mohr-Coulomb model breaks typical soil behavior into two regions: an elastic region with a linear stress-strain relationship and perfectly-plastic region where the
tangent shear stiffness is zero and any strain afterwards is irrecoverable. The two regions are defined by a yield surface Figure 2.16. That is, a boundary in principal stress space where a combination of loading (stress) defines the immediate change in material behavior from elastic, within the yield boundary, to perfectly-plastic behavior, on the yield surface. Stresses outside the defined yield surface are defined as inaccessible. Further, the yield locus (Figure 2.17a) and plastic potentials (Figure 2.17b) of the Mohr-Coulomb model can be expressed in space of distortional stress versus volumetric effective stress, where the soil properties $M$ and $M^*$ are related to the angle of shearing resistance ($\varphi$) and angle of dilation ($\psi$), respectively. In this sense, $M^*$ is one of the key parameters in defining the plastic potentials (dashed lines) to which the plastic strain increments are normal. Figure 2.18 is a three dimensional representation of the Mohr-Coulomb yield surface in principal stress space for a cohesionless soil as defined by PLAXIS. Six yield functions define the hexagonal cone structure of the yield surface.

*Figure 2.16: Elastic-perfectly plastic yield surface (Muir Wood, 2004)*
The elastic perfectly-plastic model is the first of a series of fundamental models used in PLAXIS and forms the basis from which the other models gradually converge towards modeling true soil behavior (Muir Wood, 2004).
The Mohr-Coulomb model is commonly used in geotechnical analysis to provide a rough estimate of soil behavior; however, one can see the disconnect between actual, nonlinear soil behavior and the sudden shift from linear elastic to plastic behavior. The model does not consider stress-dependency, stress-path dependency, strain dependency of stiffness, or anisotropic stiffness, and it is for this reason PLAXIS recommends the use of the MC model only for a preliminary estimate.

2.8.4 Elastic-Hardening Plastic Model

The Hardening Soil model (HS) was introduced as an extension from the Mohr-Coulomb model to include a region in which the yield surface varies nonlinearly with plastic strain. That is, the yield surface of the HS model is not fixed in stress space, but can expand due to plastic straining (Figure 2.19). Similar to how PLAXIS defines the MC model, the plastic strength parameters for the HS model are defined by friction angle ($\phi$), dilatancy angle ($\psi$), and cohesion ($c$); however, soil stiffness is represented by three different input stiffnesses (Figure 2.20): the triaxial secant loading stiffness ($E_{50}$), the triaxial unloading/reloading stiffness ($E_{ur}$), and the oedometer tangent loading stiffness ($E_{oed}$) (Brinkgreve et al., 2014). PLAXIS default settings recommend $E_{ur} = 3E_{50}$ and $E_{oed} = E_{50}$. Many soil types are successfully modeled using these assumptions; although, very soft and very stiff soils will give other ratios of stiffness which will change the shape of the stress-strain soil response curve. Note that the initial slope of the stress-strain curve, often referred to as the initial tangent modulus ($E_i$), is not used in the HS model.
Figure 2.19: Elastic-hardening plastic model yield surface and hardening yield surface in principal stress space (Muir Wood, 2004)

Figure 2.20: Hyperbolic stress-strain relation in primary loading for a standard drained triaxial test (Brinkgreve et al., 2014)

The triaxial secant stiffness ($E_{50}$) of a standard drained triaxial test (Figure 2.20) may be calculated using Equation 2.21, where $p_{ref}$ is the reference confining pressure in a triaxial test, $m$ quantifies the material stress dependency and defines the shape of the
yield loci, and \( E_{50}^{\text{ref}} \) is the reference stiffness modulus corresponding to the reference confining pressure. PLAXIS default settings use \( p_{\text{ref}} = 100 \) stress units. To simulate a logarithmic compression behavior, as observed in soft clays, \( m \) should be taken as equal to 1.0; alternatively, for hard soils \( m \) may be taken as equal to 0.5, while Von Soos (1990) suggests sands and silts lie in between those two values (Brinkgreve et al., 2014). Logarithmic compression refers to the stress dependent stiffness behavior that can be captured on a logarithmic scale and is most often applied to soft normally consolidated soils.

\[
E_{50} = E_{50}^{\text{ref}} \left( \frac{c \cos \varphi - \sigma_3' \sin \varphi}{c \cos \varphi + p_{\text{ref}} \sin \varphi} \right)^m
\]

In two-dimensions, elastic hardening can be generalized in \( p'-q \) space (Figure 2.21a) where a yield locus separates the elastic region from the elastic-hardening region, and the failure locus defines the plastic region. Figure 2.21b establishes the separate yield loci and shows the normality of plastic strain increments to those loci. Figure 2.22 introduces the yield loci and plastic potential curves for the HS model, where \( M \) is the critical state stress ratio at which constant volume shearing can occur and plastic strain increments are normal to the plastic potential curves as shown in Figure 2.22 (Muir Wood, 2004).
Figure 2.21: Elastic-hardening plastic Mohr-Coulomb model yield loci (Muir Wood, 2004)

Figure 2.22: Elastic-hardening plastic Mohr-Coulomb model yield loci and plastic potential curves (Muir Wood, 2004)
In three dimensions, PLAXIS builds off the Mohr-Coulomb hexagonal cone yield surface by providing a second type of yield surface to close the elastic region for compressive (compaction hardening) stress paths (Brinkgreve et al., 2014). The second yield surface is shown in the form of a yield cap (Figure 2.23). The triaxial modulus primarily controls the shear yield surface and thus controls the magnitude of the plastic strains associated therewith. Additionally, the oedometer modulus controls the cap yield surface and corresponding plastic strains.

In conclusion, the HS model is significantly more accurate but more complex than the MC model; however, it does not account for softening due to soil dilatancy and debonding effects associated with soft soil (Brinkgreve et al., 2014).

*Figure 2.23: Yield surface with yield cap for Hardening Soil model in principal stress space for cohesionless soil (Brinkgreve et al., 2014).*
2.8.5 Modified Cam-Clay Model

The Cam Clay model was developed in the early 1960s and was the first hardening plastic constitutive model successfully used and widely adopted for soft soils (Muir Wood, 2004). The important distinction between Cam Clay and other existing soil models is the consideration of large volume change that occurs when soft clays are in compression. To take this behavior into account an additional yield mechanism is utilized. PLAXIS uses a Modified Cam Clay (MCC) model described herein.

During virgin isotropic compression, a logarithmic relation is assumed between void ratio ($e$) and mean effective stress ($p'$) as shown in Equation 2.22 and Figure 2.24.

\[
e - e_0 = -\lambda \ln \left( \frac{p'}{p^0} \right)
\]

where:

- $e_0 = \text{initial void ratio}$
- $\lambda = \text{Cam Clay compression index}$
- $p' = \text{mean effective stress}$
- $p^0 = \text{initial mean effective stress}$

Where the compressibility of the material in initial primary loading is given by $\lambda$, which is the slope of the normal consolidation line (Figure 2.24). Similarly, the slope of the unloading-reloading line in Figure 2.24 is related to the Cam-Clay swelling index ($\kappa$), given by Equation 2.23.

\[
e - e_0 = -\kappa \ln \left( \frac{p'}{p^0} \right)
\]

48
Figure 2.24: Cam-Clay linear normal compression and unloading-reloading lines in semilogarithmic compression plane (Muir Wood, 2004)

The yield function (Equation 2.24), is used to produce the yield surface ellipse shown in Figure 2.25. The ellipse boundary separates elastic and plastic strain increments. Stress paths within the boundary exhibit only elastic strain properties, whereas stress paths that cross the boundary exhibit both elastic and plastic strain behavior (Brinkgreve et al., 2014). Within the \( q \) vs. \( p' \) plane, the friction constant \( (M) \) defines the height of the yield surface while the preconsolidation stress \( (p_c) \) defines the size of the ellipse. Ultimately \( M \) is the tangent of the critical state line (CSL) and determines how heavily the ultimate deviatoric stress \( (q) \) depends on the mean effective stress \( (p') \). By this relation, \( M \) indirectly influences the coefficient of lateral earth pressure \( (K_0) \) in a normally consolidated stress state. In this regard, the MCC model primarily relies on \( \lambda \) and \( \kappa \) to describe the soil stiffness and \( M \) to reflect soil strength.

\[
f = \frac{q^2}{M^2} + p'(p' - p_c)
\]  

2.24
Figure 2.25: Modified Cam-Clay yield surface in q vs. p’ plane (Brinkgreve et al., 2014)

The dry side of the CSL (Figure 2.25) describes the region of plastic yielding as associated with softening and ultimately failure. With the modified Cam Clay model, the values of q in the dry softening region have potential for becoming unrealistically large. While Wood (2004) is a strong proponent for the use of Cam Clay to model soft clays, Plaxis advises against its use in practical applications. Plaxis states that the modified Cam Clay model may allow for extremely large shear stresses especially when stress paths cross the CSL. Further, softening behavior predicted may have convergence problems when attempting to iterate for mesh generation, therefore a great deal of fine tuning would need to be involved. The Modified Cam Clay model is strictly for use in modeling near normally consolidated clay soils, and was only added into the Plaxis code for comparison with other methods (Brinkgreve et al., 2014).

2.8.6 Soft Soil Model

The Soft Soil (SS) model is tailored to normally consolidated soil with high compressibility such as clays and peat. The theory behind this constitutive model stems from both the Cam Clay and Mohr-Coulomb models. Similar logarithmic relations as in
the Cam Clay model are used and failure behavior is modeled similar to the Mohr Coulomb criterion.

Instead of assuming a logarithmic relationship using void ratio, in virgin isotropic virgin Soft Soil assumes a logarithmic relationship between volumetric strain and mean effective stress during virgin compression (Equation 2.25 and Figure 2.26).

\[ \varepsilon_v - \varepsilon_v^0 = -\lambda^* \ln \left( \frac{p'}{p_0} \right) \]  \hspace{1cm} 2.25

Where the compressibility of the material in initial primary loading is given by the modified compression index \((\lambda^*)\) which is the slope of the virgin compression line. Modified compression index differs from regular compression index by being a function of volumetric strain as opposed to void ratio. Similarly, the slope of the isotropic unloading-reloading line in the Soft Soil model can be expressed as a function of the modified swelling index of the soil \((\kappa^*)\) as it relates to volumetric strain (Equation 2.26). Again note the difference between the swelling index \((\kappa)\) and the modified swelling index \((\kappa^*)\). Although the indices for compression and swelling differ, the ratios \((\lambda/\kappa \text{ and } \lambda^*/\kappa^*)\) remain the same.

\[ \varepsilon_v^e - \varepsilon_v^{e0} = -\kappa^* \ln \left( \frac{p'}{p_0} \right) \]  \hspace{1cm} 2.26

In order to explain the yield function, PLAXIS simulates a triaxial stress state in which \(\sigma'_2 = \sigma'_3\). Under these stress state conditions the yield function \((f)\) is given by Equation 2.27 and physically represented in Figure 2.27.
\[ f = \frac{q^2}{M^2(p' + c \cot \varphi)} + p' - p_p^0 \exp \left( \frac{-\varepsilon_v^p}{\lambda^* - \kappa^*} \right) \]

Similar to the Cam Clay model, \( M \) and \( p' \) describe the size and shape of the ellipse where \( M \) controls the height and represents the stress states at post peak failure. The \( M \) parameter is ultimately responsible for the ratio of horizontal to vertical effective stress in primary one-dimensional compression and therefore influences the coefficient of lateral earth pressure.

Figure 2.26: Logarithmic relation between volumetric strain and mean effective stress (Brinkgreve et al., 2014)
Since the Soft Soil model is loosely based on the Mohr-Coulomb failure criterion, failure is not necessarily related to critical state, but rather is a function of strength parameters of internal friction angle and cohesion which differs from the line generated from the $M$ parameter (Brinkgreve et al., 2014). Corrections using the pre-consolidation stress are then implemented by creating a yield threshold. Note the yield cap and the expansion or increase that is allowed in primary compression (Figure 2.27) as compared to the yield cap generated in the HS model.

For general states of stress, the plastic behavior of the Soft Soil model is defined by three compression yield functions and three Mohr Coulomb yield functions. Basic parameters of the SS model used by PLAXIS to describe soil strength include cohesion, friction angle, and dilatancy angle. To capture soil stiffness behavior, modified compression index and modified swelling index are used. Modified compression and swelling indices can be related to Cam-Clay parameters according to Equations 2.28 and 2.29 or related to normalized compression index ($C_c$) and swelling index ($C_s$) by

![Figure 2.27: Soft Soil model yield surface in q vs. p' plane (Brinkgreve et al., 2014)]
Equations 2.30 and 2.31. Other advanced parameters may also be defined; however, they were not used in the scope of this investigation. Lastly, a representation of the total SS yield contour in three-dimensional principal stress space (Figure 2.28) can be compared with the yield contour of the MC model (Figure 2.23), clearly showing the potential allowance of an enlarged cap to simulate soft soil during primary compression.

\[
\lambda^* = \frac{\lambda}{1 + e} \tag{2.28}
\]

\[
\kappa^* = \frac{\kappa}{1 + e} \tag{2.29}
\]

\[
\lambda^* = \frac{C_c}{2.3(1 + e)} \tag{2.30}
\]

\[
\kappa^* = \frac{2C_s}{2.3(1 + e)} \tag{2.31}
\]

Figure 2.28: Soft Soil total yield contour in principal stress space (Brinkgreve et al., 2014)
2.8.7 Soft Soil Creep

The Soft Soil Creep (SSC) model, as suggested by its name, is an extension of the SS model to account for creep. By definition, creep is continued deformation under constant load and directly associated with secondary compression, and by association, a time dependent behavior. Such behavior is of highest concern in soft soils and/or overconsolidated soils. Secondary compression can often be assumed to be a certain percentage of primary compression, and it follows that for soft soils with large primary compression there will be a significant amount of secondary compression. This is especially the case for normally consolidated clays, silts, and peat under embankment loading.

Strength and stiffness parameters of the SS model still apply to the SSC model with the addition of the modified creep index ($\mu^*$) to capture the time dependent secondary compression of the soil. Using Equation 2.32, the modified creep index may be found from the normalized secondary compression index ($C_a$) and void ratio found from oedometer testing.

$$\mu^* = \frac{C_a}{2.3(1 + e)}$$

2.8.8 Summary

In summary, the constitutive models available within PLAXIS may be used for several purposes and applications; however, the model is only as good as the user’s understanding of constitutive theory. PLAXIS provides tables rating each model as they relate to intended application (Table 2.1) and material type (Table 2.2). It is
recommended to select a constitutive model based on these tables and to use the HS model for any soil layers where data is limited (Brinkgreve et al., 2014). As shown in these tables, for embankments built on soft soils, the SSC model is suggested as the most applicable model and consequently was used in this research. In addition to the key model parameters mentioned for each constitutive model, initial soil conditions, such as pre-consolidation stress, void ratio, coefficient of lateral stress, and permeability are also needed to effectively model soil behavior.
Table 2.1: PLAXIS recommended models for specific applications (Brinkgreve et al., 2014)

<table>
<thead>
<tr>
<th>Model</th>
<th>Foundation</th>
<th>Excavation</th>
<th>Tunnel</th>
<th>Embankment</th>
<th>Slope</th>
<th>Dam</th>
<th>Offshore</th>
<th>Other</th>
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</thead>
<tbody>
<tr>
<td>Linear Elastic model</td>
<td>C</td>
<td></td>
<td></td>
<td></td>
<td></td>
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<td></td>
<td></td>
</tr>
<tr>
<td>Mohr-Coulomb model</td>
<td>C</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Hardening Soil model</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
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<td>B</td>
</tr>
<tr>
<td>HS small model</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
</tr>
<tr>
<td>Soft Soil Creep model</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Soft Soil model</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Jointed Rock model</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
<tr>
<td>Modified Cam-Clay model</td>
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<td>C</td>
<td>C</td>
<td>C</td>
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<td>C</td>
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<tr>
<td>NGI-ADP model</td>
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<td>B</td>
<td>B</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td>A</td>
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<tr>
<td>Hoek-Brown model</td>
<td>B</td>
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<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
</tr>
</tbody>
</table>

A : The best standard model in PLAXIS for this application
B : Reasonable modelling
C : First order (crude) approximation
Table 2.2: PLAXIS recommended models for specific types of soil (Brinkgreve et al., 2014)

<table>
<thead>
<tr>
<th>Model</th>
<th>Concrete</th>
<th>Rock</th>
<th>Gravel</th>
<th>Sand</th>
<th>Silt</th>
<th>OC clay</th>
<th>NC clay</th>
<th>Peat (org)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Linear Elastic model</td>
<td>C</td>
<td></td>
<td>C</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Mohr-Coulomb model</td>
<td>A</td>
<td>B</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
<td>C</td>
</tr>
<tr>
<td>Hardening Soil model</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td>B</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>HS small model</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>A</td>
<td>B</td>
<td></td>
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<tr>
<td>Soft Soil Creep model</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>A*</td>
</tr>
<tr>
<td>Soft Soil model</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>A*</td>
</tr>
<tr>
<td>Jointed Rock model</td>
<td>A**</td>
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<td>Modified Cam-Clay model</td>
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<td></td>
<td>C</td>
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<tr>
<td>NGI-ADP model</td>
<td></td>
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<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td>A*</td>
</tr>
<tr>
<td>Hoek-Brown model</td>
<td>A**</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

A: The best standard model in PLAXIS for this application
B: Reasonable modelling
C: First order (crude) approximation
* : Soft Soil Creep model in case time-dependent behaviour is important; NGI-ADP model for short-term analysis, in case only undrained strength is known
** : Jointed Rock model in case of anisotropy and stratification; Hoek-Brown model for rock in general
3 REVIEW OF SAMPLING AND TESTING METHODS

This chapter briefly discusses the in situ methods used to characterize the soil at the Dover Test Embankment (DTE) site. Soil properties and stratigraphy determined by these testing methods form the foundation from which the PLAXIS model was constructed.

3.1 Piston Sampling

The piston sampling method is preferred for sampling cohesive, organic, or fine-grained soils and involves inserting a thin-walled metal tube (Shelby tube) into the soil by means of a hydraulically operated piston (ASTM D6518, 2008). Stationary piston sampling and Shelby tube assembly allow for the collection of relatively undisturbed soil samples suitable for laboratory tests to determine structural properties for site characterization. Piston sampling differs from conventional sampling methods by controlling the rate of entry of the soil during collection and holds onto the sample during withdrawal using suction (Getchell, 2014). Please refer to ASTM D6519: Standard Practice for Sampling of Soil Using the Hydraulically Operated Stationary Piston Sampler for a more detailed description of the method.

3.2 Standard Penetration Test

The standard penetration test (SPT) is a common geotechnical test and involves driving a split-spoon sampler 2 ft into the ground using a 140 lb hammer repeatedly dropped a distance of 30 inches. The SPT obtains a representative disturbed soil sample for identification and measures the resistance of the soil to the penetration of the sampler.
The standard penetration resistance (N-value) is taken as the sum of the number of blows required for the second and third 0.5 ft of penetration (1 ft of penetration in total). Disturbed soil samples are taken upon retracting and disassembling the split-spoon sampler. By performing this test at depth intervals specified by the engineer, a profile of N-values can be paired with their corresponding representative disturbed samples, and used for geotechnical engineering design. Please refer to ASTM D1586: Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils for complete description of the SPT method.

3.3 Flat Plate Dilatometer Test

The dilatometer test (DMT) is used to evaluate strength and deformation parameters of soils, stress history and soil stratigraphy. The dilatometer consists of a stainless steel blade with an 18° wedge tip and a circular flexible steel membrane 60 mm in diameter located on the face of the plate. The blade is about 95 mm wide and 15 mm thick. The blade is connected to a cable (pre-threaded through the rods) which is attached to a control unit for testing at the ground surface. Using a conventional drill rig, the dilatometer blade is pushed into the ground at a rate of approximately 0.8 inches per second and stopped at 0.5 ft intervals for testing. Nitrogen gas is then used to apply pressure to the circular membrane which is expanded into the soil to two preset deflections (0.05 mm and 1.1 mm from the face of the flat plate) corresponding to the corrected “A” pressure reading \(p_0\) and corrected “B” pressure reading \(p_1\). A third pressure reading, the corrected “C” reading pressure \(p_2\), may be taken when pressure is released and the membrane again reaches 0.05 mm of deflection from the face of the flat plate.
The membrane expansion is used for soil classification and correlation with engineering properties of soils (ASTM D6635, 2001). Using \( p_0, p_1, p_2 \), and the in situ water pressure \( (u_o) \), DMT parameters such as material index \( (I_D) \), horizontal stress index \( (K_D) \), and dilatometer modulus \( (E_D) \) can be calculated using Equations 3.1, 3.2, and 3.3, respectively. Engineering soil properties may then be estimated using the DMT parameters. Refer to ASTM D6635: Standard Test Method for Performing the Flat Plate Dilatometer for further explanation and data reduction of the DMT method.

\[
I_D = \frac{(p_1 - p_0)}{(p_0 - u_0)} \tag{3.1}
\]

\[
K_D = \frac{(p_0 - u_0)}{\sigma'_v0} \tag{3.2}
\]

\[
E_D = 34.7(p_1 - p_0) \tag{3.3}
\]

where:

\( \sigma'_v0 \) is the effective overburden stress at the test depth

### 3.4 Field Vane Shear Test

The field vane test (FVT) is used to measure the undrained shear strength \( (s_u) \) of saturated clays. A field vane shear test consists of inserting a four bladed vane into the soil and rotating until enough torque has been applied by the vane blades to create and shear along a cylindrical surface of the soil. During the test, torque is applied to rotate the vane at a rate of 0.05 to 0.2 degrees per second while instrument readings are
taken at regular intervals of time. The test continues until failure where the maximum torque at failure is measured. Residual shear strength may also be found by continuing to rotate the vane until a constant value of torque is observed. The remolded shear strength is also measured after rotating the vane a recommended 5 to 10 times after the initial undisturbed test and measuring the torque after failure. All torque readings are later converted to undrained shear strength based on the size and shape of the vane. The ratio of the peak undrained shear strength to the remolded shear strength gives yields the sensitivity ($S_t$) of the soil. Please refer to ASTM D2573: Standard Test Method for Field Vane Shear Test I in Cohesive Soil for complete testing procedure and data reduction.

### 3.5 Piezocone Penetration Test

The piezocone penetration test (CPTu) can be used for soil profiling and predicting a wide variety of geotechnical strength and deformation parameters. The piezocone consists of a cylindrical device, 1.44 inch in diameter with a 60 degree conical tip. The CPTu method involves pushing a cone into the ground at rate of 1.5 cm to 2.5 cm per second while measurements of tip resistance ($q_t$), sleeve friction ($f_s$), and pore water pressure ($u_2$) are taken at a minimum of every 5 cm of penetration. The CPTu method is considered a continuous test method, stopping periodically to add length of rod until refusal to penetration. The cone tip resistance is calculated by dividing the force applied on the cone by the projected cone area. Similarly, the sleeve friction is calculated by dividing the force applied on the sleeve by the sleeve area. Pore water pressure is measured by a pressure transducer on the interior of the cone tip. Similar to the DMT, engineering soil properties may be estimated based on these basic CPTu parameters.

### 3.5.1 Soil Behavior Type

The cone penetration test (CPT) is primarily used for soil profiling and determining soil type. Since physical samples of soil are not collected when using the CPT, accurate predictions of soil type based on physical characteristics cannot be achieved; however, soil type can be classified based on mechanical characteristics (strength and stiffness) of the soil (Robertson and Cabal, 2010). The prediction of soil type based on CPT is referred to as soil behavior type (SBT).

Robertson et al. (1986) proposed a SBT chart which uses cone resistance ($q_t$) and friction ratio ($R_f$) to determine soil behavior type (Figure 3.1) where friction ratio is found using sleeve friction and cone tip resistance (Equation 3.4). The chart can provide reasonable predictions of SBT for CPT soundings up to approximately 60 ft.

$$R_f = \frac{f_s}{q_t} \tag{3.4}$$
Penetration resistance and sleeve friction increase with depth due to increases in effective overburden stress ($\sigma'_{vo}$); therefore, normalization of the data is typical for deep CPT soundings where effective overburden stress at the top of the sounding is significantly different from the bottom. Robertson (1990) proposed a normalized behavior chart which uses normalized cone resistance ($Q_{tn}$), normalized friction ratio ($F_r$)
and the soil behavior type index ($I_c$) to find a normalized soil behavior type (SBTn) (Figure 3.2). The soil behavior type index may be found using Equation 3.5.

\[
I_c = ((3.47 - \log Q_{tn})^2 + (\log F_r + 1.22)^2)^{0.5}
\]

where:

\[
Q_{tn} = \frac{q_t - \sigma'_{vo}}{\sigma'_{vo}}
\]

\[
F_r = \left( \frac{f_s}{q_t - \sigma'_{vo}} \right) \times 100\%
\]

![Figure 3.2: Normalized CPT soil behavior type chart (Robertson, 2010)](image)

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

*Figure 3.2: Normalized CPT soil behavior type chart (Robertson, 2010)*
4 DOVER TEST EMBANKMENT

The Dover Test Embankment (DTE) was constructed between October, 2012 and February, 2013. Prior to construction, prefabricated vertical (PV) drains were installed to accelerate time rate of consolidation. The University of New Hampshire (UNH) performed field vane, flat plate dilatometer, and piezocone testing and collected soil samples for laboratory consolidation testing prior to both the installation of PV drains and construction of the embankment in order to characterize the site and estimate geotechnical properties of the marine clay deposit later used to build a finite element analysis (FEA) model. The primary focus of this investigation is to develop an FEA model of the DTE, including PV drains, for the prediction of total settlement and settlement rate in the field using PLAXIS 2D. All settlement predictions are compared with the DTE settlement monitoring equipment installed at the time of construction for validation. The DTE project consists of three phases of testing: 1) prior to fill placement, 2) approximately 16 weeks after the completion of the embankment fill, 3) approximately 85 weeks after the completion of the embankment fill. Phase 2 and 3 testing were performed to compare the change in soil properties after fill placement and therefore after a certain degree of consolidation; however, Phase 1 data is the primary focus of this investigation and will be discussed in detail in these next sections.

4.1 Phase 1 Testing

Phase 1 testing was performed between June 26 and August 9, 2012. Stationary piston sampling was used to collect soil samples within the clay layer, later used for laboratory testing completed between November 8, 2012 and February 6, 2013. The purpose of
Phase 1 testing was to obtain a detailed soil stratigraphy of the site and to determine in situ soil properties for geotechnical design and settlement analysis. Testing for Phase 1 was primarily focused on Segment 1 (Figure 4.1) where three DMT, one FVT, and two CPTu profiles were performed. Two CPTu profiles were performed within the boundaries of each segment to be constructed. Table 4.1 shows a detailed summary of all testing performed during Phase 1 while Table 4.2 summarizes monitoring of the groundwater table depth during that phase.

*Figure 4.1: DTE general segment location (Google Maps, 2014)*
Table 4.1: Summary of sampling and in situ testing during Phase 1 (Getchell, 2013)

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

Table 4.2: Groundwater table readings during Phase 1 (Getchell, 2013)

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

Location: 603 + 15 CL El. 12.3 ft
4.1.1 Soil Stratigraphy

Prior to analyzing test data for soil properties, soil stratigraphy was determined by comparing initial SPT results performed by the NHDOT with results obtained from DMT and CPTu testing methods. The soil strata determined in this section will later be used in defining the thickness of layers in the PLAXIS model. Average soil properties of each soil layer will be discussed in following sections.

4.1.2 Initial SPT

Between September 13 and November 28, 2011, prior to clearing the proposed RT 16 Exit 6 SB On-Ramp site, the NHDOT performed twelve SPT profiles and installed one observation well (Q-B109). The general location of each test boring is shown in Figure 4.2. The subsurface conditions in the test borings include the following strata, listed in the sequence which they were encountered: Fill, Alluvium, Upper Marine Deposit, Lower Marine Deposit, Glacial Outwash, and Glacial Till. Table 4.3 describes relative thicknesses, soil density/consistency and describes the composition of each strata identified by geologists at the NHDOT. Note that the fill material was identified as man-placed fill from when the current RT 16 SB On-Ramp was constructed and was not encountered in areas extending further from edge of pavement.

A general soil profile depicting Table 4.3 is shown in Figure 4.3; however, soil stratigraphy varies throughout the site. For a more detailed representation of subsurface conditions for a particular segment of the proposed embankment, the reader should refer to Figure 4.4 where individual test borings are plotted against the recorded elevation and along a baseline oriented with the proposed SB On-Ramp. Note that the
numbers located aside each boring indicate uncorrected SPT N values. The standard penetration testing confirmed the extent of the soft marine clay. The very soft silty clay was designated as a primary concern in regards to consolidation and settlement; therefore, the next phase of testing focused on determining properties required to predict the engineering soil behavior.

Figure 4.2: DTE SPT borehole locus map (Blair, 2012)
Table 4.3: Initial subsurface conditions as described by NHDOT Geotechnical Test Embankment Recommendations report (2012)

<table>
<thead>
<tr>
<th>Soil Strata</th>
<th>Approximate Thickness (ft)</th>
<th>Soil Density/Consistency (ft)</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fill</td>
<td>2-10</td>
<td>loose-dense</td>
<td>loamy topsoil, fine sand, medium to fine sand, and sandy silt</td>
</tr>
<tr>
<td>Alluvium</td>
<td>4.5-10</td>
<td>loose-medium dense</td>
<td>fine sand with variable amounts of silt</td>
</tr>
<tr>
<td>Upper Marine Deposit</td>
<td>~10</td>
<td>soft-medium stiff</td>
<td>predominantly silty clay, silt with clay, and occasional sand lens</td>
</tr>
<tr>
<td>Lower Marine Deposit</td>
<td>~50</td>
<td>very soft</td>
<td>predominantly silty clay, silt with clay, and occasional sand lens</td>
</tr>
<tr>
<td>Glacial Outwash</td>
<td>4.9-28.5</td>
<td>loose-dense</td>
<td>fine sand with variable amounts of silt, silt layers, trace gravel</td>
</tr>
<tr>
<td>Glacial Till</td>
<td>0.6-7.3</td>
<td>medium dense-very dense</td>
<td>silty fine sand or fine sandy silt, trace medium coarse sand and gravel, cobbles and boulders expected</td>
</tr>
</tbody>
</table>
Figure 4.3: Soil profile of initial conditions as described by NHDOT Geotechnical Test Embankment Recommendations report (2012)
Figure 4.4: DTE subsurface fence diagram of borehole profiles (Blair, 2012)
4.1.3 Piezocone

Piezocone penetration data in this report was initially analyzed using CPeT-IT software and later corrected using site specific correlations determined by Getchell (2013). This section shows the predicted soil stratigraphy based on Normalized Soil Behavior Type (SBTn) from CPTu data collected at the DTE. All CPTu data presented herein, unless otherwise mentioned, reflects data prior to site specific correlations and is therefore used as an estimate for determining the thickness of soil strata.

Robertson (2010) recommends that the SBT chart (Figure 3.1) is best suited for soundings extending up to 60 ft in depth and suggests using the SBTn chart (Figure 3.2) for deeper soundings. Though not by much, all CPT soundings performed as a part of Phase 1 testing exceeded 60 ft; therefore, the soil behavior type for each sounding presented in this report is normalized.

Two CPTu soundings were completed in Segment 1 as part of Phase 1 testing (Q-B220 and Q-B221). Using CPeT-IT, the SBTn is plotted versus depth for both of these soundings and shown in Figure 4.5 and Figure 4.6. Additionally the SBTn index ($I_c$) is shown. Comparing these plots with the soil stratigraphy gathered from standard penetration testing, the SBTn classification from CPT can be directly correlated with the material observed. Due to variation of SBTn during the transition of layers, CPT soil layers were correlated by first identifying the major SBTn present and second, by recognizing the minor normalized soil behavior types that may be present at varying degrees (Table 4.4). All SBTn profiles may be found in Appendix A.
Figure 4.5: Segment 1 Q-B220 SBn generated by CPeT-IT (Sta. 603+37, RT 16)
Figure 4.6: Segment 1 Q-B221 SBTn generated by CPeT-IT (Sta. 603+47, RT 11)
Table 4.4: Typical soil layer correlation between SPT and CPT

<table>
<thead>
<tr>
<th>SPT Material</th>
<th>CPT Major SBTn</th>
<th>CPT Minor SBTn</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>Sand &amp; silty sand</td>
<td>Silty sand &amp; sandy silt</td>
</tr>
<tr>
<td>Upper Marine (soft)</td>
<td>Clay</td>
<td>Clay &amp; silty clay and/or Silty sand &amp; sandy silt</td>
</tr>
<tr>
<td>Lower Marine (very soft)</td>
<td>Sensitive fine grained</td>
<td>Clay &amp; silty clay and/or Clay</td>
</tr>
<tr>
<td>Glacial Outwash</td>
<td>Sand &amp; silty sand</td>
<td>Silty sand &amp; sandy silt</td>
</tr>
</tbody>
</table>

While it is apparent the CPT soundings consist of different soil layers made up of varying SBTn, distinguishing where one layer ends and the next starts is not so obvious. Also note the SBTn difference between Q-B220 (Figure 4.5) and Q-B221 (Figure 4.6). Both soundings were completed in Segment 1, approximately 10 ft from one another; however, Q-B220 SBTn indicates the clay layer behaves like a clay & silty clay where in Q-B221 the clay behaves like a sensitive fine grained material. Despite these differences, the SBTn index profile indicates the two soundings are similar. It is assumed there is a fine line between the two and that the difference merely indicates a slightly less sensitive silty marine layer in Q-B220 causing the SBTn to plot as clay & silty clay. The soil layer thicknesses as determined by this methodology are compared with those obtained from the NHDOT standard penetration test stratigraphy and presented in Table 4.5. The CPTu soundings were assumed to be terminated within the glacial outwash or on the glacial till margin; therefore glacial till was not recorded as a soil layer using SBTn. It is important to distinguish that standard penetration test (SPT)
soil classification is an actual classification of soil material based on grain size and soil type observed during testing while the normalized soil behavior index (SBTn) from CPTu is a soil classification based on soil behavior as determined by cone and friction sleeve resistance.

Table 4.5: Estimates of Segment 1 soil layer thicknesses based on CPTu and SPT borings

<table>
<thead>
<tr>
<th>Test Method</th>
<th>SPT</th>
<th>CPTu</th>
<th>SPT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole Name</td>
<td>Q-B201</td>
<td>Q-B220</td>
<td>Q-B221</td>
</tr>
<tr>
<td>Station (ft)</td>
<td>602+00</td>
<td>603+37</td>
<td>603+47</td>
</tr>
<tr>
<td>Offset (ft)</td>
<td>LT 50</td>
<td>RT 16</td>
<td>RT 11</td>
</tr>
<tr>
<td>Surface Elevation (ft)</td>
<td>15.3</td>
<td>11.9</td>
<td>12.4</td>
</tr>
</tbody>
</table>

Thickness of Layer (ft)

<table>
<thead>
<tr>
<th>Thickness of Layer (ft)</th>
<th>Alluvium</th>
<th>Upper Marine (soft)</th>
<th>Lower Marine (very soft)</th>
<th>Glacial Outwash</th>
<th>Glacial Till</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10.0</td>
<td>9.0</td>
<td>49.0</td>
<td>10.5</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td>10.0</td>
<td>8.0</td>
<td>48.0</td>
<td>5.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>10.0</td>
<td>9.0</td>
<td>47.0</td>
<td>3.0</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5.0</td>
<td>46.0</td>
<td>47.0</td>
<td>11.0</td>
<td>6.2</td>
</tr>
</tbody>
</table>

4.1.4 Flat Plate Dilatometer

Three flat plate dilatometer profiles were performed within Segment 1 prior to construction of the embankment (Q-B213, Q-B215, and Q-B218). DMT test borings Q-B213 and Q-B215 were analyzed for soil stratigraphy based on the DMT material index parameter ($I_D$) and correlated with material layers identified by standard penetration testing, similar to methods performed when analyzing CPTu SBTn. Material index given by DMT ranges from sand to silty sand to very soft material described as mud and/or peat and provides a general guideline for determining layer thicknesses and composition for correlation. Figure 4.7 and Figure 4.8 show DMT instrument readings,
material index and estimated soil layers for Q-B213 and Q-B215, respectively. Table 4.6 presents estimated soil layer thicknesses for Segment 1 as found using DMT material index and compares to thicknesses found using the SPT method. Borehole Q-B218 was terminated within the soft marine clay at a depth of 42 ft below the ground surface. Since the profile does not capture a complete dataset of the clay layer, the soil stratigraphy of Q-B218 was not analyzed for comparison. Again, the glacial outwash and glacial till layers are not easily distinguished from one another; therefore the DMT profiles were assumed to be terminated within the glacial outwash or on the glacial till margin; therefore glacial till was not recorded as a soil layer using the material index.
Figure 4.7: DTE Segment 1 Q-B213 DMT results with material index and estimated soil layers
Figure 4.8: DTE Segment 1 Q-B215 DMT results with material index and estimated soil layers
Table 4.6: Estimates of Segment 1 soil layer thicknesses based on DMT and SPT borings

<table>
<thead>
<tr>
<th>Test Method</th>
<th>SPT</th>
<th>DMT</th>
<th>SPT</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole Name</td>
<td>Q-B201</td>
<td>Q-B213</td>
<td>Q-B215</td>
</tr>
<tr>
<td>Station (ft)</td>
<td>602+00</td>
<td>603+28</td>
<td>603+40</td>
</tr>
<tr>
<td>Offset (ft)</td>
<td>LT 50</td>
<td>LT 14</td>
<td>LT 4</td>
</tr>
<tr>
<td>Surface Elevation (ft)</td>
<td>15.3</td>
<td>12.4</td>
<td>12.0</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Thickness of Layer (ft)</th>
<th>Alluvium</th>
<th>Upper Marine (soft)</th>
<th>Lower Marine (very soft)</th>
<th>Glacial Outwash</th>
<th>Glacial Till</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>10.0</td>
<td>9.0</td>
<td>49.0</td>
<td>10.5</td>
<td>4.7</td>
</tr>
<tr>
<td></td>
<td>10.4</td>
<td>10.1</td>
<td>43.0</td>
<td>12.3</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>10.0</td>
<td>8.0</td>
<td>47.5</td>
<td>7.5</td>
<td>-</td>
</tr>
<tr>
<td></td>
<td>5.0</td>
<td>3.0</td>
<td>46.0</td>
<td>11.0</td>
<td>6.2</td>
</tr>
</tbody>
</table>

4.1.5 Summary

The methods described in previous sections for estimating soil stratigraphy based on SPT, CPTu, and DMT results were implemented to all segments of the Dover Test Embankment and reflect in situ soil conditions prior to construction of the embankment. Soil layer thicknesses for each test and segment are shown for comparison in Table 4.7. As stated previously, the upper and lower marine deposits consisting of soft and very soft silty clay are of highest concern in regards to settlement. Figure 4.9 shows a subsurface fence diagram of selected CPTu soundings from each segment. Sections appearing in black represent soil where SBTn classification was not viable. From this figure it can be noted that the very soft lower marine deposit gradually increases in thickness from Segment 1 to Segment 4 and then decreases in thickness leading into Segment 5. The increased dominance of the very soft marine layer can also be observed in Table 4.7 which summarizes all tests and approximate soil layers. Note that
SPT profiles are not grouped within segments since they were performed at transition points where either one segment starts and/or the next segment ends. More detailed CPTu SBTn profiles may be observed in the appendices corresponding to each individual segment. The soil layers determined in this section will serve as a basis for collecting average soil properties per layer to later be used in settlement calculations.

Figure 4.9: CPTu subsurface fence diagram of borehole profiles
Table 4.7: Summary of DTE testing and corresponding soil layer estimates per segment

<table>
<thead>
<tr>
<th>Test Method</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Borehole Name</td>
<td>SPT</td>
<td>DMT</td>
<td>CPTu</td>
<td>SPT</td>
<td>SPT</td>
</tr>
<tr>
<td>Q-B201</td>
<td>Q-B213</td>
<td>Q-B215</td>
<td>Q-B220</td>
<td>Q-B221</td>
<td>Q-B204</td>
</tr>
<tr>
<td>Station (ft)</td>
<td>602+00</td>
<td>603+28</td>
<td>603+40</td>
<td>603+37</td>
<td>603+47</td>
</tr>
<tr>
<td>Offset (ft)</td>
<td>LT 50</td>
<td>LT 14</td>
<td>LT 4</td>
<td>RT 16</td>
<td>RT 11</td>
</tr>
<tr>
<td>Surface Elev. (ft)</td>
<td>15.3</td>
<td>12.4</td>
<td>12.0</td>
<td>11.9</td>
<td>12.4</td>
</tr>
<tr>
<td>Thickness of Layer (ft)</td>
<td>Alluvium</td>
<td>10.0</td>
<td>10</td>
<td>10</td>
<td>10</td>
</tr>
<tr>
<td></td>
<td>Upper Marine (soft)</td>
<td>9.0</td>
<td>10</td>
<td>8</td>
<td>8</td>
</tr>
<tr>
<td></td>
<td>Lower Marine (very soft)</td>
<td>49.0</td>
<td>43</td>
<td>48</td>
<td>48</td>
</tr>
<tr>
<td></td>
<td>Glacial Outwash</td>
<td>10.5</td>
<td>12</td>
<td>8</td>
<td>5</td>
</tr>
<tr>
<td></td>
<td>Glacial Till</td>
<td>4.7</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
</tbody>
</table>
4.2 Analysis of Laboratory and Field Test Results

The data collected during Phase 1 was used to establish soil parameters of the subsurface material. Getchell (2013) compared parameters from all tests (CPTu, DMT, and FVT) with parameters found from laboratory testing, such as consolidation and Atterberg limit testing. Using the test specific parameters, Getchell (2013) was able to determine site specific correlations for certain DMT and CPTu parameters such as total unit weight \((\gamma_T)\) and overconsolidation ratio \((OCR)\). Getchell (2013) then compared marine clay data with previous research findings from the area including Ladd (1972), Findlay (1991), and research conducted by UNH class CIE 961 (1997). This section provides relevant information from Getchell (2013) in regards to defining soil properties required for settlement calculation using PLAXIS 2D. For detailed data reduction, refer to Getchell (2013). To establish uniform material properties, test specific stratigraphy established in Chapter 4.1 was used to determine average properties found within each soil layer (alluvium, upper marine, lower marine, and glacial outwash/till).

4.2.1 Total Unit Weight

Total unit weight \((\gamma_T)\) for the marine clay deposit was found by using undisturbed clay specimens from Shelby tube piston samples that were used in consolidation testing (Getchell, 2013). Additionally, the unit weight for the subsoil at the DTE site was determined empirically using data collected from CPTu and DMT. Using the laboratory unit weights results as the baseline, Getchell (2013) correlated the DMT and CPTu estimates of unit weight by shifting the conventional correlations as shown in Equations 4.1 and 4.2, respectively. Note that these correlations are based solely on correcting for
the unit weight in the upper and lower marine layers and may not reflect a good correlation with the alluvium and glacial layers. A comparison of the laboratory and site specific unit weights are shown in Figure 4.10. Note that the site specific correlation for unit weight is based on data collected in Segment 1, but is assumed to apply to the entire site and therefore applied to all DMT and CPTu data collected in all embankment segments. DMT and CPTu unit weight results are shown in Table 4.8 and Table 4.9 respectively.

\[ \gamma_{T_{Dover\ (DMT)}} = \gamma_{T_{DMT}} + 10pcf \]  \hspace{1cm} 4.1

\[ \gamma_{T_{Dover\ (CPTu)}} = \gamma_{T_{CPTu}} + 20pcf \]  \hspace{1cm} 4.2
Figure 4.10: Site specific correlations for total unit weight in Segment 1 (Getchell, 2014)
### Table 4.8: DMT site specific unit weights per soil layer for Segment 1

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Q-B213</th>
<th>Q-B215</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>122</td>
<td>123</td>
<td>123</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>108</td>
<td>115</td>
<td>111</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>107</td>
<td>111</td>
<td>109</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>125</td>
<td>130</td>
<td>127</td>
</tr>
</tbody>
</table>

### Table 4.9: CPTu site specific unit weights per soil layer

<table>
<thead>
<tr>
<th>Soil layer</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q-B220</td>
<td>Q-B221</td>
<td>Q-B222</td>
<td>Q-B223</td>
<td>Q-B224</td>
<td>Q-B225</td>
</tr>
<tr>
<td>Alluvium</td>
<td>136</td>
<td>136</td>
<td>136</td>
<td>134</td>
<td>137</td>
<td>134</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>122</td>
<td>121</td>
<td>117</td>
<td>120</td>
<td>118</td>
<td>118</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>110</td>
<td>111</td>
<td>111</td>
<td>110</td>
<td>110</td>
<td>110</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>135</td>
<td>132</td>
<td>139</td>
<td>141</td>
<td>139</td>
<td>140</td>
</tr>
</tbody>
</table>
4.2.2 Void Ratio
The initial void ratio, as determined by Getchell (2013), is shown in Figure 4.11 along with void ratios determined by Ladd (1972) and Findlay (1991). The average initial void ratio for the upper and lower marine deposits was found to be 1.08 and 1.17 respectively. Getchell (2014) reports the average void ratio of the lower marine deposit to be 1.2. From Figure 4.11, a similar void ratio for the alluvium and glacial soil layers is shown to be less than 0.8. Based on the data available immediately above the upper marine deposit, a void ratio of 0.8 is assumed for the alluvium. Assuming a mixed grained glacial outwash/till a void ratio value of 0.5 was assumed.
Figure 4.11: Initial void ratio for Segment 1 from consolidation testing and comparison with other findings (Getchell, 2014)
4.2.3 Compression Parameters

Compression parameters such as the compression index \((C_c)\) and recompression index \((C_r)\) are useful in predicting the consolidation behavior of soft material. Compression indices are reported by Getchell (2013) from consolidation testing. Getchell (2014) reports the average compression and recompression indices for the lower marine clay to be 0.31 and 0.05, respectively. Note that recompression and swelling index \((C_s)\) as used by PLAXIS are used interchangeably.

Since the consolidation test is actually a reloading test, even a high-quality sample will exhibit a recompression curve with a slope less than that of the field virgin compression curve (Holtz et al., 2011). Schmertmann (1955) suggests a graphical correction to evaluate a more realistic field behavior as shown in Figure 4.12. The average recompression index and Schmertmann corrected compression index used in the settlement evaluation on this project are shown in Table 4.10. One can compare the difference between the corrected \(C_c\) values and the uncorrected values using Figure 4.13. Since the \(C_c\) values found from the Schmertmann method are higher, a larger degree of consolidation and settlement will be found.
Figure 4.12: Typical consolidation curve with Schmertmann (1955) correction

Table 4.10: Summary of compression indices for DTE Segment 1 marine layers

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>$C_c$</th>
<th>$C_r$</th>
<th>$C_c$ (Schmertmann, 1955)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Marine</td>
<td>0.28</td>
<td>0.05</td>
<td>0.33</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>0.31</td>
<td>0.05</td>
<td>0.36</td>
</tr>
</tbody>
</table>
Figure 4.13: Compression parameters from consolidation testing
4.2.4 Hydraulic Conductivity

The hydraulic conductivity or coefficient of permeability indicates the ability of fluids, typically water, to flow through soil or rock and is measured in units of length per time. Permeability is a function of the soil properties (density and void ratio) and properties of the liquid (density and viscosity). In this case permeability was empirically calculated using CPTu, DMT, and consolidation testing. A general list of formulas used in CPeT-IT is given in Appendix C; however, please refer to ASTM standards or Getchell (2013) for a detailed data reduction.

The permeability profiles for all methods of calculation are shown in Figure 4.14. The CPTu permeability profiles show higher permeability compared to those found using DMT and consolidation testing methods, especially with increasing depth. This trend is best explained by the empirical table developed by Lunne et al. (1997) used to find permeability based on SBTn (Table 4.11). While this may be an acceptable estimate, DMT and consolidation testing use consolidation coefficients to calculate permeability and may be regarded as more accurate. Note that the DMT method uses the horizontal coefficient of consolidation \((c_h)\), found from dissipation testing, to calculate horizontal permeability while the consolidation method uses vertical coefficient of consolidation \((c_v)\) to find vertical permeability. As is often the case, the vertical permeability found from consolidation testing is lower than the horizontal permeability in the soft marine clay layer.

Table 4.12, Table 4.13, and Table 4.14 show tabulated results of permeability estimates for the soil layers at the DTE site based on CPTu, DMT, and consolidation testing,
respectively. These values will be used in settlement calculations. Although CPTu permeability will not be used, note the consistency between segments for each soil layer proves the permeability is relatively constant throughout.

Figure 4.14: Permeability comparison for DTE Segment 1
Table 4.11: Estimated permeability based on SBTn (Lunne et al., 1997)

<table>
<thead>
<tr>
<th>$SBT_n$</th>
<th>Permeability (ft/sec)</th>
<th>(m/sec)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>$3 \times 10^{-8}$</td>
<td>$1 \times 10^{-8}$</td>
</tr>
<tr>
<td>2</td>
<td>$3 \times 10^{-7}$</td>
<td>$1 \times 10^{-7}$</td>
</tr>
<tr>
<td>3</td>
<td>$1 \times 10^{-9}$</td>
<td>$3 \times 10^{-10}$</td>
</tr>
<tr>
<td>4</td>
<td>$3 \times 10^{-8}$</td>
<td>$1 \times 10^{-8}$</td>
</tr>
<tr>
<td>5</td>
<td>$3 \times 10^{-6}$</td>
<td>$1 \times 10^{-6}$</td>
</tr>
<tr>
<td>6</td>
<td>$3 \times 10^{-4}$</td>
<td>$1 \times 10^{-4}$</td>
</tr>
<tr>
<td>7</td>
<td>$3 \times 10^{-2}$</td>
<td>$1 \times 10^{-2}$</td>
</tr>
<tr>
<td>8</td>
<td>$3 \times 10^{-6}$</td>
<td>$1 \times 10^{-6}$</td>
</tr>
<tr>
<td>9</td>
<td>$1 \times 10^{-8}$</td>
<td>$3 \times 10^{-9}$</td>
</tr>
</tbody>
</table>
Table 4.12: DTE permeability determined by CPTu for all segments

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Q-B220</th>
<th>Q-B221</th>
<th>Q-B222</th>
<th>Q-B223</th>
<th>Q-B224</th>
<th>Q-B225</th>
<th>Q-B226</th>
<th>Q-B227</th>
<th>Q-B228</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>7.0E+00</td>
<td>1.3E+01</td>
<td>5.4E+00</td>
<td>6.4E+00</td>
<td>1.0E+01</td>
<td>5.2E+00</td>
<td>1.7E+01</td>
<td>9.4E+00</td>
<td>2.0E+01</td>
<td>1.0E+01</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>3.8E-01</td>
<td>3.3E-01</td>
<td>4.7E-02</td>
<td>3.8E-01</td>
<td>1.6E-02</td>
<td>2.8E-02</td>
<td>2.4E-01</td>
<td>3.9E-01</td>
<td>5.7E-01</td>
<td>2.6E-01</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>6.7E-03</td>
<td>5.0E-03</td>
<td>1.1E-02</td>
<td>7.6E-03</td>
<td>9.0E-03</td>
<td>1.1E-02</td>
<td>7.0E-03</td>
<td>3.6E-03</td>
<td>6.3E-03</td>
<td>7.5E-03</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>6.2E-01</td>
<td>2.5E-02</td>
<td>1.4E+00</td>
<td>1.8E+00</td>
<td>9.3E-01</td>
<td>1.9E+00</td>
<td>8.3E-01</td>
<td>9.3E-01</td>
<td>5.3E+00</td>
<td>1.5E+00</td>
</tr>
</tbody>
</table>

Table 4.13: DTE permeability determined by DMT in Segment 1

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Q-B215</th>
<th>Q-B218</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>6.2E-04</td>
<td>2.7E-04</td>
<td>4.5E-04</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>2.9E-04</td>
<td>1.9E-04</td>
<td>2.4E-04</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>1.3E-02</td>
<td>-</td>
<td>1.3E-02</td>
</tr>
</tbody>
</table>

Table 4.14: DTE permeability determined by consolidation testing of Segment 1 soil samples

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Q-B212</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>-</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>7.0E-04</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>1.5E-04</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>-</td>
</tr>
</tbody>
</table>
4.2.5 Overconsolidation Ratio

Getchell (2013) plotted OCR values for all tests performed within Segment 1 and compared with values found by Findlay (1993) near Pease Air Force Base. As in section 4.2.1, a similar site specific correlation for OCR was made by Getchell according to Equations 4.3 and 4.4. Note that these correlations are based solely on correcting for the OCR in the upper and lower marine layers and may not reflect a good correlation with the alluvium and glacial layers.

\[
OCR_{Dover(\text{DMT})} = OCR_{\text{DMT}} - 1.0
\]

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

From Figure 4.15, one can discern that the alluvium layer is highly overconsolidated, the upper marine layer is overconsolidated, the lower marine deposit is normally to near normally consolidated, and the glacial material is overconsolidated to a degree similar to the alluvium. Table 4.15, Table 4.16, and Table 4.17 give estimates of OCR values per soil layer as determined by CPTu, DMT, and consolidation tests, respectively. All methods confirm a similar trend of OCR values for the upper and lower marine layers. The CPTu profile is the only profile that includes data for the alluvium and glacial layers, reporting average OCR values of 10.5 and 6.2, respectively. The glacial layer is likely overconsolidated from glacial loading in the past. These values are based on empirical estimates performed by the CPeT-IT program. A general list of formulas used in CPeT-IT is given in 0; however, please refer to ASTM standards or Getchell (2013) for detailed data reduction.
Figure 4.15: Site Specific Correlations for Overconsolidation Ratio in Segment 1 (Getchell, 2014)
Table 4.15: CPTu site specific OCR values

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q-B220</td>
<td>Q-B221</td>
<td>Q-B222</td>
<td>Q-B223</td>
<td>Q-B224</td>
<td>Q-B225</td>
</tr>
<tr>
<td>Alluvium</td>
<td>6.1</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>12.8</td>
<td>10.8</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>3.8</td>
<td>3.3</td>
<td>2.0</td>
<td>2.2</td>
<td>2.3</td>
<td>2.6</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>1.4</td>
<td>1.3</td>
<td>1.4</td>
<td>1.6</td>
<td>1.4</td>
<td>1.5</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>1.9</td>
<td>4.3</td>
<td>12.3</td>
<td>0.0</td>
<td>8.6</td>
<td>8.7</td>
</tr>
</tbody>
</table>

Table 4.16: DMT site specific OCR values from Segment 1

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Segment 1</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q-B213</td>
<td>Q-B215</td>
<td>Average</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Alluvium</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Upper Marine</td>
<td>3.3</td>
<td>4.4</td>
<td>3.8</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Lower Marine</td>
<td>1.2</td>
<td>0.8</td>
<td>1.0</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.17: Consolidation test OCR values from Segment 1

<table>
<thead>
<tr>
<th>OCR</th>
<th>Segment 1</th>
</tr>
</thead>
<tbody>
<tr>
<td>Soil Layer</td>
<td>Q-B212</td>
</tr>
<tr>
<td>Alluvium</td>
<td>-</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>4.4</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>1.3</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>-</td>
</tr>
</tbody>
</table>
4.2.6 Coefficient of Lateral Earth Pressure at Rest

The coefficient of lateral earth pressure at rest \( (K_0) \) expresses the stress conditions in the ground in terms of effective stress. In general terms, \( K_0 \) is the ratio of effective horizontal stress to the effective vertical stress. Using DMT and CPTu data, \( K_0 \) was empirically estimated as shown in Figure 4.16. Refer to ASTM standards and CPeT-IT manual for data reduction methods.

All \( K_0 \) values are presented per soil layer in Table 4.18 and Table 4.19 for the CPTu and DMT methods, respectively. The DMT \( K_0 \) values are consistently higher than those found using CPTu. From both methods, the \( K_0 \) value for the marine deposit is less than the alluvium and glacial layers. It is unclear which method should be regarded as more accurate, but such values presented may be used as an estimate when selecting \( K_0 \) values for settlement calculations. Findings from Getchell (2013) indicate the DMT is a more dependable method for estimating \( K_0 \), as she compared her findings from the DTE site with Findlay (1991) who used a self-boring pressuremeter (SBPM), which is regarded as a superior method for determining \( K_0 \).
NHDOT Geotechnical Test Embankment
Dover, NH

Figure 4.16: DTE Segment 1 $K_0$ profiles
**Table 4.18: CPTu $K_0$ values from DTE site**

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q-B220</td>
<td>Q-B221</td>
<td>Q-B222</td>
<td>Q-B223</td>
<td>Q-B224</td>
</tr>
<tr>
<td>Alluvium</td>
<td>1.25</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>1.59</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>0.88</td>
<td>0.74</td>
<td>0.75</td>
<td>0.78</td>
<td>0.81</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>0.61</td>
<td>0.62</td>
<td>0.65</td>
<td>0.67</td>
<td>0.68</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>0.98</td>
<td>1.01</td>
<td>1.57</td>
<td>0.00</td>
<td>1.40</td>
</tr>
</tbody>
</table>

**Table 4.19: DMT $K_0$ values from Segment 1**

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Segment 1</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q-B213</td>
</tr>
<tr>
<td>Alluvium</td>
<td>-</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>1.16</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>0.89</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>-</td>
</tr>
</tbody>
</table>
4.2.7 Effective Friction Angle

The effective friction angle ($\varphi'$) of a soil material describes the frictional strength of the soil and is affected by factors such as mineralogy, shape of particles, gradation, and void ratio. Using CPTu data, Ricceri et al. (2002) proposed Equation 4.5 to calculate the effective friction of soil with classifications of ML and SP-SM (Das, 2011). Equation 4.5 was used to estimate the effective friction angle for each CPTu sounding and averaged for each soil layer. Further, the overall site averages are reported in Table 4.20.

$$\varphi' = \tan^{-1} \left[ 0.38 + 0.27 \log \left( \frac{q_c}{\sigma'_{vo}} \right) \right]$$  \hspace{1cm} 4.5

where:

$q_c = $ cone resistance

$\sigma'_{vo} = $ effective overburden stress

The values reported for alluvium and the glacial deposits are typical of what might be expected of those materials; however, the $\varphi'$ values for the marine clay layers are higher than expected. Based on Atterberg limits, the marine layers at the DTE site classify as a low plasticity clay (CL) with some specimens falling into the silty clay of low plasticity classification (ML) (Getchell, 2014). Therefore, Equation 4.5 does not fully apply for the clay layers. From a similar project located in Portsmouth, NH, Ladd (1972) reports values of 25° and 20° to 25° for the upper and lower clay deposits, respectively. These values are more consistent with soft clays and will be used for settlement calculations.
Table 4.20: CPTu effective friction angle values from DTE site

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
<th>Average</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Q-B220</td>
<td>Q-B221</td>
<td>Q-B222</td>
<td>Q-B223</td>
<td>Q-B224</td>
<td>Q-B225</td>
</tr>
<tr>
<td>Alluvium</td>
<td>45</td>
<td>45</td>
<td>45</td>
<td>45</td>
<td>44</td>
<td>44</td>
</tr>
<tr>
<td>Marine 1</td>
<td>35</td>
<td>35</td>
<td>33</td>
<td>35</td>
<td>33</td>
<td>34</td>
</tr>
<tr>
<td>Marine 2</td>
<td>31</td>
<td>31</td>
<td>31</td>
<td>31</td>
<td>31</td>
<td>31</td>
</tr>
<tr>
<td>Glacial Outwash/Till</td>
<td>38</td>
<td>35</td>
<td>41</td>
<td>42</td>
<td>40</td>
<td>41</td>
</tr>
</tbody>
</table>
4.2.8 Summary

The purpose of this site investigation was to establish soil properties required for settlement predictions. Table 4.21 summarizes the suggested properties to be used when calculating settlement. Note that these suggested parameters are median estimates of all tests when applicable. In addition to the soil properties established by in situ and laboratory testing, other soil properties are required by PLAXIS including apparent cohesion \((c')\), dilatancy angle \((\psi)\) and change of permeability \((c_k)\).

Table 4.21: Summary of suggested material specific parameters for DTE site

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>Density (pcf)</th>
<th>(e)‡</th>
<th>(C_{cl})‡</th>
<th>(k_v)¥</th>
<th>(OCR)§</th>
<th>(K_0)*</th>
<th>(\phi')**</th>
</tr>
</thead>
<tbody>
<tr>
<td>Alluvium</td>
<td>130 ± 7</td>
<td>0.8</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Upper Marine</td>
<td>116 ± 4</td>
<td>1.08</td>
<td>0.33</td>
<td>0.05</td>
<td>4.46E-04</td>
<td>7.05E-04</td>
<td>3.65 ± 0.8</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>110 ± 1</td>
<td>1.17</td>
<td>0.36</td>
<td>0.05</td>
<td>2.38E-04</td>
<td>1.50E-04</td>
<td>1.2 ± 0.2</td>
</tr>
<tr>
<td>Glacial Deposit</td>
<td>133 ± 5</td>
<td>0.5</td>
<td>-</td>
<td>-</td>
<td>1.34E-02</td>
<td>-</td>
<td>6.2</td>
</tr>
</tbody>
</table>

* Parameter based on DMT and CPTu  
‡ Parameter based on Oedometer  
† Parameter based on DMT  
‡ Parameter based on oedometer  
§ Parameter Based on DMT, CPTu, and oedometer  
** Parameter based on CPTu  
# Parameter from Ladd (1972)

4.3 Suggested Parameters

In situ and laboratory testing methods provided a majority of the soil properties for the Soft Soil Creep model; however, additional properties are required. This section discusses how those additional properties were obtained.
4.3.1 Cohesion

Cohesion (c) is a strength parameter which describes plasticity of a material which can be obtained in the laboratory from direct shear or triaxial testing. When using the Soft Soil or Soft Soil Creep models, PLAXIS requires a value be entered for apparent cohesion (c'). Not to be confused with c, apparent cohesion for sands and normally consolidated clays is taken as equal to zero, while for overconsolidated clays c' > 0 (Das, 2011).

4.3.2 Hardening Soil Parameters

Since all in situ and laboratory testing was primarily concerned with the soft marine clay, properties for the embankment, sand drainage blanket, alluvium, and glacial layers are limited. Therefore, the arbitrary Hardening Soil parameters suggested by PLAXIS will be used to provide sufficient information for modeling (Table 4.22). The embankment was assumed to be dense due to the layered lift compaction implemented during construction. The glacial layers were also assumed to be dense based on in situ results including SPT blow counts. From DMT and CPTu results, the alluvium layer exhibited a density slightly less than the glacial layers and was therefore considered a medium-dense material where parameter values were interpolated accordingly. SPT blow counts for the alluvium are also in agreement. The drainage blanket was assumed to have a medium density.
Table 4.22: Arbitrary Hardening Soil parameters for sands of different densities converted to English units (Brinkgreve et al., 2014)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Loose</th>
<th>Medium</th>
<th>Dense</th>
<th>Unit</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E_{50}^{\text{ref}}$ (for $p_{\text{ref}} = 1$ bar)</td>
<td>417709</td>
<td>626563</td>
<td>835417</td>
<td>psf</td>
</tr>
<tr>
<td>$E_{ur}^{\text{ref}}$ (for $p_{\text{ref}} = 1$ bar)</td>
<td>1253126</td>
<td>1879689</td>
<td>2506252</td>
<td>psf</td>
</tr>
<tr>
<td>$E_{oor}^{\text{ref}}$ (for $p_{\text{ref}} = 1$ bar)</td>
<td>417709</td>
<td>626563</td>
<td>835417</td>
<td>psf</td>
</tr>
<tr>
<td>Cohesion $c$</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>psf</td>
</tr>
<tr>
<td>Friction angle $\varphi$</td>
<td>30</td>
<td>35</td>
<td>40</td>
<td>°</td>
</tr>
<tr>
<td>Dilatancy angle $\psi$</td>
<td>0</td>
<td>5</td>
<td>10</td>
<td>°</td>
</tr>
<tr>
<td>Poisson's ratio $\nu_{ur}$</td>
<td>0.2</td>
<td>0.2</td>
<td>0.2</td>
<td>-</td>
</tr>
<tr>
<td>Power $m$</td>
<td>0.5</td>
<td>0.5</td>
<td>0.5</td>
<td>-</td>
</tr>
<tr>
<td>$K_0^{nc}$ (using cap)</td>
<td>0.5</td>
<td>0.43</td>
<td>0.36</td>
<td>-</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>psf</td>
</tr>
<tr>
<td>Failure ratio</td>
<td>0.9</td>
<td>0.9</td>
<td>0.9</td>
<td>-</td>
</tr>
</tbody>
</table>

4.3.3 Change of Permeability

Change of permeability ($c_k$) is an advanced parameter within PLAXIS which accounts for the change in permeability during consolidation and will influence the rate of consolidation. The permeability will change according to the relationship shown in Equation 4.6 (Wong, 2013) and may be approximated by Equation 4.7.

$$ \log \left( \frac{k}{k_0} \right) = \frac{\Delta e}{c_k} \quad 4.6 $$

where:

$\Delta e$ = the change in void ratio

$k_0$ = the input permeability

$k$ = the new permeability

$c_k$ = change of permeability
Thus, the change of permeability may be approximated as:

\[ c_k \approx 0.007(LL - 10) \]

where:

LL is the liquid limit

Using Equation 4.7 and the liquid limit (LL) for test specimens from Segment 1, the change in permeability was estimated for the entire marine deposit (Table 4.23). The upper and lower marine deposits were determined to have an average \( c_k \) value of 0.17 and 0.18, respectively. These values are comparable with a value of 0.2 obtained from predefined soft clay material within PLAXIS and determined to be a reasonable estimate for the DTE model.

*Table 4.23: Summary of marine deposit Atterberg Limit and change of permeability results for DTE Segment 1*

<table>
<thead>
<tr>
<th>Depth (ft)</th>
<th>LL(_{avg})</th>
<th>PL(_{avg})</th>
<th>PL(_{avg})</th>
<th>w(_{avg}) (%)</th>
<th>( c_k )</th>
</tr>
</thead>
<tbody>
<tr>
<td>11</td>
<td>40</td>
<td>23</td>
<td>17</td>
<td>38</td>
<td>0.21</td>
</tr>
<tr>
<td>13</td>
<td>36</td>
<td>21</td>
<td>15</td>
<td>38</td>
<td>0.18</td>
</tr>
<tr>
<td>15</td>
<td>34</td>
<td>22</td>
<td>12</td>
<td>42</td>
<td>0.17</td>
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<td>16</td>
<td>32</td>
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<td>11</td>
<td>40</td>
<td>0.15</td>
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<td>21</td>
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<td>25</td>
<td>13</td>
<td>46</td>
<td>0.19</td>
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<td>31</td>
<td>42</td>
<td>25</td>
<td>17</td>
<td>49</td>
<td>0.22</td>
</tr>
<tr>
<td>36</td>
<td>42</td>
<td>25</td>
<td>17</td>
<td>45</td>
<td>0.22</td>
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<tr>
<td>41</td>
<td>41</td>
<td>25</td>
<td>16</td>
<td>44</td>
<td>0.22</td>
</tr>
<tr>
<td>46</td>
<td>39</td>
<td>23</td>
<td>16</td>
<td>45</td>
<td>0.20</td>
</tr>
<tr>
<td>46.5</td>
<td>30</td>
<td>21</td>
<td>9</td>
<td>31</td>
<td>0.14</td>
</tr>
<tr>
<td>51</td>
<td>37</td>
<td>23</td>
<td>14</td>
<td>43</td>
<td>0.19</td>
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<td>56</td>
<td>35</td>
<td>23</td>
<td>12</td>
<td>41</td>
<td>0.18</td>
</tr>
<tr>
<td>61</td>
<td>29</td>
<td>22</td>
<td>7</td>
<td>34</td>
<td>0.13</td>
</tr>
<tr>
<td>66</td>
<td>28</td>
<td>19</td>
<td>8</td>
<td>32</td>
<td>0.12</td>
</tr>
</tbody>
</table>
4.3.4 Secondary Compression Index

For settlement calculations which include secondary settlement, the secondary compression index \((C_α)\) is required. \(C_α\) is defined as the strain per log cycle of time after primary consolidation is completed and may be determined by plotting consolidation results on a semi-log graph of void ratio vs. time. The consolidation testing performed by Getchell (2013) for the Dover test embankment does not provide any data after primary consolidation; therefore, \(C_α\) was not able to be obtained. Ladd (1972) discusses settlement prediction of an embankment with sand drains located in Portsmouth, NH and encountered similar soil stratigraphy including soft marine clay. Based on lab data, Ladd (1972) reports a maximum rate of secondary compression to be 1.5% per log cycle of time at stress just beyond the maximum past pressure. At higher stresses \(C_α\) for normally consolidated clay was estimated to be 0.005 to 0.01 while overconsolidated clay was estimated to be 0.001.

4.4 Prefabricated Vertical Drains and Mandrel

The prefabricated vertical drains (wick drains) selected for the DTE site are AMERIDRAIN 407. The specifications listed in Table 4.24 indicate a 4 in. width with an approximate thickness of 0.125 inches. The mandrel cross section measures approximately 6 x 3 inches while the driving shoe/anchor plate (Figure 4.18) cross section measures approximately 6 x 5 inches. The dimensions given were used to find the equivalent diameters using Equation 2.13 (Table 4.25).
## Table 4.24: AMERIDRAIN 407 specifications (AWD, 2014)

<table>
<thead>
<tr>
<th>Property Values</th>
<th>ASTM Test Method</th>
<th>Unit of Measure</th>
<th>407</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>FABRIC</strong></td>
<td></td>
<td></td>
<td>MARV</td>
</tr>
<tr>
<td>Material ¹</td>
<td></td>
<td></td>
<td>PP</td>
</tr>
<tr>
<td>Water Flow Rate</td>
<td>D 4491</td>
<td>gpm/ft²</td>
<td>60</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lpm/m²</td>
<td>2,444</td>
</tr>
<tr>
<td>Grab Tensile Strength</td>
<td>D 4632</td>
<td>lbs</td>
<td>130</td>
</tr>
<tr>
<td>CBR Puncture Resistance</td>
<td>D 6241</td>
<td>lbs</td>
<td>290</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>1,290</td>
</tr>
<tr>
<td>Trapezoidal Tear</td>
<td>D 4533</td>
<td>lbs</td>
<td>60</td>
</tr>
<tr>
<td>Apparent Opening Size</td>
<td>D 4751</td>
<td>sec⁻¹</td>
<td>0.8</td>
</tr>
<tr>
<td></td>
<td></td>
<td>micron</td>
<td>210</td>
</tr>
<tr>
<td>Permittivity</td>
<td>D 4491</td>
<td>sec⁻¹</td>
<td>0.8</td>
</tr>
<tr>
<td>Grab Elongation</td>
<td>D 4632</td>
<td>%</td>
<td>50 %</td>
</tr>
<tr>
<td>UV Resistance</td>
<td>D 4355</td>
<td>% / 500 Hrs</td>
<td>70 %</td>
</tr>
<tr>
<td><strong>CORE</strong></td>
<td></td>
<td></td>
<td>Typical Value</td>
</tr>
<tr>
<td>Material ¹</td>
<td></td>
<td></td>
<td>PP</td>
</tr>
<tr>
<td>Tensile Strength</td>
<td>D 4595</td>
<td>lbs</td>
<td>225</td>
</tr>
<tr>
<td></td>
<td></td>
<td>N</td>
<td>1001</td>
</tr>
<tr>
<td><strong>PRODUCT</strong></td>
<td></td>
<td></td>
<td>Typical Value</td>
</tr>
<tr>
<td>Discharge Capacity</td>
<td>D 4716</td>
<td>gpm</td>
<td>1.6</td>
</tr>
<tr>
<td></td>
<td></td>
<td>Lpm</td>
<td>6</td>
</tr>
<tr>
<td>Roll Length</td>
<td></td>
<td>ft</td>
<td>1000</td>
</tr>
<tr>
<td></td>
<td></td>
<td>m</td>
<td>305</td>
</tr>
<tr>
<td>Roll Width</td>
<td></td>
<td>in</td>
<td>4.0</td>
</tr>
<tr>
<td></td>
<td></td>
<td>mm</td>
<td>102</td>
</tr>
</tbody>
</table>

¹ PP = Polypropylene
Figure 4.17: Approximate cross section of AMERIDRAIN 407

Figure 4.18: AMERIDRAIN 407 and driving shoe
Table 4.25: Dimensions and equivalent diameters for drain, mandrel, and mandrel shoe

<table>
<thead>
<tr>
<th>PV Drain Cross Section</th>
<th>Width</th>
<th>Thickness</th>
<th>Equivalent drain cylinder diameter</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>a</td>
<td>4.000</td>
<td>d_w = 2.626</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>0.125</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.333 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.010 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.219 ft</td>
<td></td>
</tr>
<tr>
<td>Mandrel Shoe Cross Section</td>
<td>Width</td>
<td>Thickness</td>
<td>Mandel shoe equivalent diameter</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>6.0</td>
<td>d_ms = 7.003</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>5.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.417 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.584 ft</td>
<td></td>
</tr>
<tr>
<td>Mandrel Cross Section</td>
<td>Width</td>
<td>Thickness</td>
<td>Mandrel equivalent diameter</td>
</tr>
<tr>
<td></td>
<td>a</td>
<td>6.0</td>
<td>d_m = 5.730</td>
</tr>
<tr>
<td></td>
<td>b</td>
<td>3.0</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.50 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.250 ft</td>
<td></td>
</tr>
<tr>
<td></td>
<td></td>
<td>0.477 ft</td>
<td></td>
</tr>
</tbody>
</table>

4.4.1 Plane Strain Hydraulic Conductivity Correction

The configuration of the test embankment, approximately 1000 ft long with a relatively constant cross section, lends itself to plane strain analysis; however, a problem is created when adding vertical drains. Using PLAXIS 2D, the placement of a vertical drain creates an infinitely small seepage boundary within the soil which requires a prescribed external pressure (embankment) to be applied to permit the discharge of water out of the soil. By modeling the test embankment cross section in plane strain, the entire cross section is assumed to extend infinitely in the z-direction (perpendicular to the cross-section), thus generating zero strain in the z-direction. When plane strain analysis is selected in PLAXIS 2D, the vertical drains are modeled as extending infinitely in the z-direction, creating a seepage boundary plane with horizontal drainage. In reality, the vertical drains were placed in a triangular pattern, have cross-sectional dimensions measuring 4 inches wide by approximately 1/8 inch thick, and thus exhibit radial drainage.
Several methods have been developed to address this problem. Since consolidation is highly dependent on flow rate and governed by the permeability of the soil, alternate methods are used for finding equivalent horizontal permeability for plane strain conditions. Hird et al. (1992) developed a solution (Equation 4.8) by implementing geometric matching techniques for a single drain (Wong, 2013) based on the drain spacing ratio \( n \). Indraratna et al. (2005) developed a multi-drain simulation by adjusting the permeability of the soil while maintaining geometric drain spacing and the same rate of consolidation to obtain an equivalent horizontal permeability for the plane strain condition (Equation 4.9). Both equations are similar and both ignore the existence of smear and well resistance; however, Equation 4.9 was selected to be used for this investigation since drain dimensions cannot be defined within PLAXIS and drain spacing was kept constant for ease of comparison between corrected and uncorrected permeability.
\[ k_{h,ps} = k_h \frac{0.67}{\ln(n) - 0.75} \]  
\[ k_{h,ps} = k_{h,ax} \frac{0.67(n-1)^2}{n^2 \ln(n) - 0.75} \]

where:

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

\( d_e \) = the equivalent soil cylinder diameter for triangular spacing = 1.05s

\( s \) = drain spacing

\( d_w \) = the equivalent drain diameter = \( \frac{2(a + b)}{\pi} \)

\( a \) and \( b \) represent the width and thickness of the drain

\( k_{h,ps} \) = plane strain coefficient of horizontal permeability

\( k_h \) = coefficient of horizontal permeability

\( k_{h,ax} \) = axisymmetric coefficient of horizontal permeability

Since a triangular spacing of drains was used, Equation 2.11 was used to find the equivalent soil cylinder diameter for each drain spacing. The equivalent soil cylinder diameter \( (d_e) \), drain spacing ratio \( (n) \), and permeability ratios per Hird et al. (1992) and Indraratna et al. (2005) are presented in Table 4.26. Using Table 4.26, the permeability found from the DMT was corrected for plane strain for each spacing and both the upper and lower marine deposit layers (Table 4.27). Note how the plane strain permeability corrections are applied more heavily (larger reduction) as drain spacing increases. The reduction in permeability for the plane strain condition varies from about 25% for a PV spacing of 6 ft to about 20% for a spacing of 14 ft.
Table 4.26: Equivalent soil diameter, drain spacing ratio, and permeability corrections

<table>
<thead>
<tr>
<th>Triangular spacing</th>
<th>s</th>
<th>6</th>
<th>10</th>
<th>14</th>
<th>ft</th>
</tr>
</thead>
<tbody>
<tr>
<td>Equivalent soil cylinder diameter</td>
<td>d_e</td>
<td>75.6</td>
<td>126</td>
<td>176.4</td>
<td>in</td>
</tr>
<tr>
<td>Equivalent soil cylinder radius</td>
<td>r_e</td>
<td>37.8</td>
<td>63</td>
<td>88.2</td>
<td>in</td>
</tr>
<tr>
<td>Drain spacing ratio</td>
<td>n</td>
<td>28.8</td>
<td>48.0</td>
<td>67.2</td>
<td>-</td>
</tr>
<tr>
<td>Hird et al. (1992)</td>
<td>k_h,pd/k_h</td>
<td>0.257</td>
<td>0.215</td>
<td>0.194</td>
<td>-</td>
</tr>
<tr>
<td>Indraratna et al. (2005)</td>
<td>k_h,ps/k_h,ax</td>
<td>0.239</td>
<td>0.206</td>
<td>0.188</td>
<td>-</td>
</tr>
</tbody>
</table>

Table 4.27: Site specific plane strain permeability correction

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>PV Spacing (ft)</th>
<th>k_h From DMT</th>
<th>Hird et al. (1992)</th>
<th>Indraratna et al. (2005)</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Marine</td>
<td>6</td>
<td>4.46E-04</td>
<td>1.14E-04</td>
<td>1.69E-04</td>
<td>ft/day</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>4.46E-04</td>
<td>9.57E-05</td>
<td>1.45E-04</td>
<td>ft/day</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>4.46E-04</td>
<td>8.64E-05</td>
<td>1.33E-04</td>
<td>ft/day</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>6</td>
<td>2.38E-04</td>
<td>6.11E-05</td>
<td>3.59E-05</td>
<td>ft/day</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>2.38E-04</td>
<td>9.57E-05</td>
<td>3.09E-05</td>
<td>ft/day</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>2.38E-04</td>
<td>8.64E-05</td>
<td>2.82E-05</td>
<td>ft/day</td>
</tr>
</tbody>
</table>

4.4.2 Smear Zone Correction

In addition to correcting permeability for plane strain analysis, a correction was made to consider the decrease in permeability around the drains caused from the installation. Significant smearing of the clay was observed during installation (Figure 4.19) and assumed to reduce the rate of consolidation by inherently reducing permeability.
The permeability of the smear zone is not known and was estimated to be $0.5k_h$ based on ranges given in Table 4.28 by Sathananthan (2005). Many of these values reflect remolded values determined experimentally on a single drain or are based on field observation of projects with consistent drain spacing; however, drains are often installed in large groups and may be installed at variable spacing. Therefore, a relationship is proposed relating an additional reduction in permeability based upon a ratio between the smear zone ratio ($S$) and the drain spacing ratio ($n$) (Equation 4.10). This ratio is effectively a ratio of the equivalent diameter of the smear zone ($d_s$) to the equivalent drain cylinder diameter ($d_e$).
Table 4.28: Proposed smear zone parameters (Sathananthan, 2005)

<table>
<thead>
<tr>
<th>Source</th>
<th>Extent</th>
<th>Permeability</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Barron (1948)</td>
<td>$r_s = 1.6r_m$</td>
<td>$k_s/k_v = 3$</td>
<td>Assumed</td>
</tr>
<tr>
<td>Hansbo (1979)</td>
<td>$r_s = 1.5~3r_m$</td>
<td>Open</td>
<td>Based on available literature at that time</td>
</tr>
<tr>
<td>Hansbo (1981)</td>
<td>$r_s = 1.5r_m$</td>
<td>$k_s/k_v = 3$</td>
<td>Assumed in case study</td>
</tr>
<tr>
<td>Bergado et al. (1991)</td>
<td>$r_s = 2r_m$</td>
<td>$k_s/k_v = 1$</td>
<td>Laboratory investigation and back analysis for Bangkok soft clay</td>
</tr>
<tr>
<td>Onoue (1991)</td>
<td>$r_s = 1.6r_m$</td>
<td>$k_s/k_v = 3$</td>
<td>From test interpretation</td>
</tr>
<tr>
<td>Almeida et al. (1993)</td>
<td>$r_s = 1.5~2r_m$</td>
<td>$k_s/k_v = 3~6$</td>
<td>Based on experiences</td>
</tr>
<tr>
<td>Indraratna et al. (1998)</td>
<td>$r_s = 4~5r_w$</td>
<td>$k_s/k_v = 1.15$</td>
<td>Laboratory investigation (For Sydney clay)</td>
</tr>
<tr>
<td>Chai &amp; Miura (1999)</td>
<td>$r_s = 2~3r_m$</td>
<td>$k_s/k_v = C_f(k_s/k_v)$</td>
<td>$C_f$ the ratio between lab and field values</td>
</tr>
<tr>
<td>Hird et al. (2000)</td>
<td>$r_s = 1.6r_m$</td>
<td>$k_s/k_v = 3$</td>
<td>Recommend for design</td>
</tr>
<tr>
<td>Xiao (2000)</td>
<td>$r_s = 4r_m$</td>
<td>$k_s/k_v = 1.3$</td>
<td>Laboratory investigation (For Kaolin clay)</td>
</tr>
</tbody>
</table>

$r_s$: radius of smear zone, $k_s$: smear zone permeability, and $k_v$: vertical permeability.
\[ k_{r,ps} = k_{h,ps} \left( 1 - \frac{S}{n} \right) \frac{k_r}{k_h} \]

where:

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

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

- \( k_{r,ps} \) is the remolded permeability corrected for plane strain conditions
- \( k_{h,ps} \) is the horizontal permeability corrected for plane strain conditions
- \( \frac{k_r}{k_h} \) is the estimated ratio of remolded to horizontal permeability

As previously discussed, the smear zone is governed by the driving shoe at the end of the mandrel. In addition, Holtz et al. (1991) recommends a conservative estimate of the equivalent smear zone diameter (\( d_s \)) to equal about 2.5-3\( d_{ms} \) where \( d_{ms} \) is the equivalent diameter of the mandrel shoe. Therefore the cross section of the shoe was used to estimate \( d_s \). The smear zone ratio (\( S \)) was then calculated using Equation 4.11. These results are tabulated in Table 4.29. The plane strain remolded permeability (\( k_{r,ps} \)) was then found using the average \( d_s \) values from Table 4.29 and the \( k_{h,ps} \) values from Indraratna et al. (2005). Final site and segment specific correlations for \( k_{r,ps} \) are reported in Table 4.30. Note that the drain group correction (1-\( S/n \)) applies a more significant reduction of permeability for the 6 ft drain spacing and is applied less significantly for increased drain spacing.
\[ S = \frac{d_s}{d_w} \]

Table 4.29: Conservative estimate of equivalent smear zone diameter and smear zone ratio

<table>
<thead>
<tr>
<th></th>
<th>Equivalent diameter of smear zone</th>
<th>Smear zone ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( d_s ) (in.)</td>
<td>( d_s ) (ft)</td>
</tr>
<tr>
<td>Shoe</td>
<td>( d_s = 3d_m )</td>
<td>21.0</td>
</tr>
<tr>
<td></td>
<td>( d_s = 2.5d_m )</td>
<td>17.5</td>
</tr>
<tr>
<td></td>
<td>Average (( d_s = 2.75d_m ))</td>
<td>19.3</td>
</tr>
<tr>
<td>Mandrel</td>
<td>( d_s = 3d_{ms} )</td>
<td>17.2</td>
</tr>
<tr>
<td></td>
<td>( d_s = 2.5d_{ms} )</td>
<td>14.3</td>
</tr>
<tr>
<td></td>
<td>Average (( d_s = 2.75d_{ms} ))</td>
<td>15.8</td>
</tr>
</tbody>
</table>

Table 4.30: Site and segment specific plane strain remolded clay permeability correction

<table>
<thead>
<tr>
<th>Soil Layer</th>
<th>PV Spacing (ft)</th>
<th>( k_{n,ps} ) (ft/day)</th>
<th>( (1-S/n) )</th>
<th>( k_{r,ps} ) (ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Upper Marine</td>
<td>6</td>
<td>1.1E-04</td>
<td>0.37</td>
<td>4.0E-05</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>9.2E-05</td>
<td>0.42</td>
<td>3.9E-05</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>8.4E-05</td>
<td>0.45</td>
<td>3.7E-05</td>
</tr>
<tr>
<td>Lower Marine</td>
<td>6</td>
<td>5.7E-05</td>
<td>0.37</td>
<td>2.1E-05</td>
</tr>
<tr>
<td></td>
<td>10</td>
<td>4.9E-05</td>
<td>0.42</td>
<td>2.1E-05</td>
</tr>
<tr>
<td></td>
<td>14</td>
<td>4.5E-05</td>
<td>0.45</td>
<td>2.0E-05</td>
</tr>
</tbody>
</table>

4.5 DTE Geometry

The Dover Test Embankment consists of five segments with variable geometry, drain spacing, and drain placement depth. Each segment is approximately 200 ft long with transitional segments on either end. The end segments also have vertical drains installed at 8 ft triangular spacing; however, they do not contain any monitoring
equipment. Table 4.31 summarizes the average geometry of each test embankment including PV drain geometry and forms a basis from which settlement models were constructed.
### Table 4.31: Dover Test Embankment section geometry

<table>
<thead>
<tr>
<th>Segment</th>
<th>Fill Height</th>
<th>Triangular PV Spacing</th>
<th>Proposed PV Bottom Elevation</th>
<th>PV Notes:</th>
<th>Station Start (ft)</th>
<th>Station End (ft)</th>
<th>Width of Top of Embankment (ft)</th>
<th>Side Slope</th>
<th>Top Elevation (ft)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>12</td>
<td>6</td>
<td>-54</td>
<td>Penetrate through marine deposit</td>
<td>602+00</td>
<td>604+00</td>
<td>71</td>
<td>2 to 1</td>
<td>24.2</td>
</tr>
<tr>
<td>2</td>
<td>12</td>
<td>10</td>
<td>-54</td>
<td>Penetrate through marine deposit</td>
<td>604+00</td>
<td>606+00</td>
<td>63</td>
<td>2 to 1</td>
<td>24.2</td>
</tr>
<tr>
<td>3</td>
<td>12</td>
<td>10</td>
<td>-47</td>
<td>Terminate 10 ft above glacial outwash</td>
<td>606+00</td>
<td>608+00</td>
<td>54</td>
<td>2 to 1</td>
<td>23.1</td>
</tr>
<tr>
<td>4</td>
<td>18</td>
<td>10</td>
<td>-58</td>
<td>Penetrate through marine deposit</td>
<td>608+00</td>
<td>610+00</td>
<td>32</td>
<td>1.5 to 1</td>
<td>28.5</td>
</tr>
<tr>
<td>5</td>
<td>12</td>
<td>14</td>
<td>-54</td>
<td>Penetrate through marine deposit</td>
<td>610+00</td>
<td>612+00</td>
<td>39</td>
<td>2 to 1</td>
<td>22</td>
</tr>
</tbody>
</table>
4.6 Monitoring

Prior to construction, the test embankment site was extensively instrumented to record vertical deformation during and after the embankment fill placement and to record vertical and lateral deformations adjacent to the embankment in addition to monitoring changes in pore water pressure. The instrumentation includes settlement platforms (SP), surface settlement points (DMP), subsurface settlement points (SSP), vibrating wire piezometers (VWPZ), and inclinometers (INCL). Segments 1 and 4 have the highest concentration of instrumentation and are the primary focus of this investigation. Additional segment monitoring will support the applicability of the settlement models.

Table 4.32 identifies all applicable geotechnical instruments installed and monitored at the embankment site and provides locations and PLAXIS finite element analysis (FEA) coordinates used for comparison. Additionally, Figure 4.20 shows a visual layout of the instrument locations within Segment 1. Plan view segment instrument locations for all segments may be found in Appendix B.
### Table 4.32: DTE field and FEA instrument locations

<table>
<thead>
<tr>
<th>ID</th>
<th>Description</th>
<th>Segment Number</th>
<th>Station (ft)</th>
<th>Offset (ft)</th>
<th>Point Designation</th>
<th>Coordinates x (ft)</th>
<th>Coordinates y (ft)</th>
<th>Analyzed Parameter</th>
</tr>
</thead>
<tbody>
<tr>
<td>SP1</td>
<td>Settlement Platform</td>
<td>1</td>
<td>603+00</td>
<td>RT 5</td>
<td>A</td>
<td>67.5</td>
<td>11.5</td>
<td>P_water</td>
</tr>
<tr>
<td>SP2</td>
<td>Settlement Platform</td>
<td>1</td>
<td>603+25</td>
<td>RT 5</td>
<td>A</td>
<td>67.5</td>
<td>11.5</td>
<td>P_water</td>
</tr>
<tr>
<td>DMP1</td>
<td>Surface Settlement Point</td>
<td>1</td>
<td>603+00</td>
<td>LT 32</td>
<td>B</td>
<td>35.5</td>
<td>25.5</td>
<td>u_y</td>
</tr>
<tr>
<td>DMP2</td>
<td>Surface Settlement Point</td>
<td>1</td>
<td>603+00</td>
<td>LT 56</td>
<td>C</td>
<td>59.5</td>
<td>13.5</td>
<td>u_y</td>
</tr>
<tr>
<td>DMP3</td>
<td>Surface Settlement Point</td>
<td>1</td>
<td>603+00</td>
<td>LT 76</td>
<td>D</td>
<td>79.5</td>
<td>13.5</td>
<td>u_y</td>
</tr>
<tr>
<td>DMP4</td>
<td>Surface Settlement Point</td>
<td>1</td>
<td>603+00</td>
<td>LT 96</td>
<td>E</td>
<td>99.5</td>
<td>13.5</td>
<td>u_y</td>
</tr>
<tr>
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<td>RT 5</td>
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<td>-29</td>
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<td>12</td>
<td>u_y</td>
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<td>611+25</td>
<td>RT 10</td>
<td>A</td>
<td>0</td>
<td>10</td>
<td>u_y</td>
</tr>
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<td>Surface Settlement Point</td>
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<td>611+00</td>
<td>LT 9</td>
<td>B</td>
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<td>23.5</td>
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<td>611+00</td>
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<tr>
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<td>Surface Settlement Point</td>
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<td>611+00</td>
<td>LT 55</td>
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<td>63.5</td>
<td>11.5</td>
<td>u_y</td>
</tr>
<tr>
<td>DMP40</td>
<td>Surface Settlement Point</td>
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<td>611+00</td>
<td>LT 75</td>
<td>E</td>
<td>83.5</td>
<td>11.5</td>
<td>u_y</td>
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</table>
Figure 4.20: DTE Segment 1 instrument locations
4.6.1 Settlement Platforms

Two settlement platforms (SP) were installed within each test embankment segment to monitor total settlement and time rate of settlement. Each settlement platform consists of black pipes affixed to 3 ft square by ¾ in. thick plywood placed just beneath the sand drainage blanket (Figure 4.21). Each platform was placed on the centerline of the embankment to capture maximum settlement. Figure 4.22 shows settlement data collected since installation (October 8, 2012 to December 1, 2014).

Segment 4 (red) indicates the highest degree of settlement (approx.1.8 ft) due to the 18 ft embankment fill height and thick soft clay layer below grade. Segment 1 (black) has the second highest degree of settlement to date, which is mostly attributed to having the smallest drain spacing. In contrast, Segment 5 has the smallest degree of settlement due to having the largest drain spacing. Lastly, the geometry of Segments 2 and 3 are nearly identical with the exception of Segment 3 having shortened drains which terminate approximately 10 feet short of the bottom of the soft marine clay. Despite the shortened drain length and contrary to expectations, Segment 3 exhibits more settlement, mostly attributed to having a thicker soft clay layer compared to Segment 2.
Figure 4.21: Settlement platform specifications (Blair, 2013)
Figure 4.22: DTE settlement platform and fill height data
4.6.2 Surface Settlement Points

Surface settlement points, or deformation monitoring points (DMP’s), were installed within each segment with an array perpendicular to the alignment of the DTE. Prior to construction of the test embankment, DMP’s were installed starting at the approximate toe of the slope at increments of 20 ft and extending to approximately 80 ft from the toe. After the embankment fill had been placed, additional DMP’s were installed at the approximate crest of the slope. All surface settlement points were installed within a 4 in. diameter borehole drilled to an approximate depth of 4 ft. Wooden risers were then placed in the borehole with about 6 inch. of stickup above the ground surface, and clean sand was used for backfill and to provide frost protection.

Elevation data collected by surveying methods was used to calculate the surface settlement at each location. The scope of this investigation includes analysis of the DMP’s located at the crest of slope, toe of slope, 20 ft from the toe, and 40 ft from the toe. Figure 4.23 shows the surface settlements observed in the field for Segment 1 and Segment 4 from date of installation to September 7, 2013. Significant surface settlement was observed at the crest and toe of the slope, but minimal settlement was observed at 20 ft and 40 ft from the toe.

The DMPs located at the crest of the slope were not installed until the embankment fill had been completed for each respective segment. After 125 days from installation, the surface settlement observed at the crest of the slope for Segment 1 (black) and 4 (red) were approximately 0.3 ft and 0.52 ft, respectively. Similar to the settlement platforms, the increase settlement in Segment 4 is due to the increased fill height (1.5 times that of
Segment 1) and the increased thickness of the soft clay layer within Segment 4. Similarly, the surface settlements observed at the toe of the slope in Segment 1 and 4 were approximately 0.16 ft and 0.32 ft, respectively as of September 7, 2013. For additional comparison, Figure 4.24 and Figure 4.25 show the progression of settlement perpendicular to the embankment alignment for Segments 1 and 4, respectively.
Figure 4.23: DTE Segment 1 and 4 field surface settlement points
Figure 4.24: Segment 1 progression of settlement perpendicular to alignment
Figure 4.25: Segment 4 progression of settlement perpendicular to alignment

4.6.3 Subsurface Settlement Points

Borros anchors were used to establish subsurface settlement points (SSP) to estimate subsurface settlement at various depths within Segments 1 and 4. The borros anchor (Figure 4.26) is an anchor with prongs which are extended (anchored) in the soil at a specified depth to establish a borros point. The borros anchor is connected to an
isolated riser pipe which extends above ground level. The riser pipe is then surveyed over time and the data is reduced to find vertical displacement.

Two anchors were installed in each segment with one borros point anchored at the top of the soft to very soft marine deposit and the other anchored at the top of the upper, stiffer marine deposit (Blair, 2013). From Figure 4.27, the degree of settlement was found to decrease with increasing depth. For the first 100 days, Segment 1 has a faster rate of settlement because of the closer PV spacing. After 100 days, an additional fill height of 8 ft was added to Segment 4 which drastically accelerated the rate of settlement. The difference between both segments seems to increase with time, but generally seems to decrease with depth.
Figure 4.26: Typical borros anchor settlement point (Geokon, 2011)
Segment 1 (black) and 4 (red) Subsurface Settlement at Approx. Centerline of Embankment

Figure 4.27: DTE Segment 1 and 4 field subsurface settlement points
4.6.4 Vibrating Wire Piezometers

A total of two vibrating wire piezometers were installed at the DTE site. One piezometer was installed in Segment 1 and the other in Segment 4. The instruments were installed at the approximate midpoint depth of the soft to very soft marine deposit (Blair, 2013). The procedure consisted of casing a borehole, lowering the piezometer, and then carefully extracting the casing in small increments. Bentonite chips were placed at the bottom of the borehole followed by approximately 1 ft of clean sand, the piezometer, more clean sand, then more bentonite chips (Blair, 2013). This procedure isolates the piezometer tip while providing permeable backfill and was repeated at each location (Blair, 2013). The corresponding elevations for the piezometers for Segment 1 and 4 are -29.0 ft and -32.5 ft, respectively. Using these instruments, the pore water pressure was measured during and after placement of the embankment fill (Figure 4.28).

Initial readings are likely high due to loading from construction equipment. Pore water pressure quickly comes to a low point at hydrostatic pressure (approx. 2500 psf) after approximately 1 week then increases sharply as the embankment is constructed. A peak pore water pressure of 3433 psf is reported for Segment 1 after approximately 40 days, corresponding with the completion of the 12 ft fill placement. A gradual decrease in pore pressure then occurs corresponding to consolidation. Segment 4 reaches a peak pore pressure of 3318 psf after placing an additional 6 ft of fill on top of the existing 12 ft embankment and decreases slightly over time.

The first peak for Segment 4 was expected to be similar to the peak in Segment 1 given similar stratigraphy, fill height, and placement depth. Assuming the water table is
similar, the only other difference is the drain spacing. Given Segment 4 has a larger drain spacing, one would expect the pore pressure to be higher. Also note the shallow peak generated by the placement of the 6 ft fill and subsequent very slow dissipation of pore pressure. These inconsistencies are thought to be caused by being within close proximity to a drain; therefore, significantly decreasing the pore pressure reading.

![Figure 4.28: Segment 1 and 4 field piezometer pore water pressure](image)

**4.6.5 Slope Inclinometers**

Five inclinometer locations are available within the DTE site. Segment 1 has two inclinometer locations (INCL1 and INCL2) spaced approximately 8 ft and 33 ft from the toe of slope and perpendicular to the embankment alignment. Segment 4 has three inclinometer locations (INCL3, INCL4, and INCL5) spaced approximately 7 ft, 32 ft, and 57 ft from the toe of slope. Inclinometer guide tube casing was installed at each location extending from the surface to a point within the glacial layers to establish base fixity (Blair, 2013). The base fixity provides a secured fixed point where zero lateral
deformation should occur. A baseline reading of each guide tube was taken by lowering a probe into the guide casing while recording the change in inclination at sequential depth intervals. Repeating this process and determining the change in inclination from the baseline reading allows for horizontal deformation to be found.

Figure 4.29 through Figure 4.33 shows incremental deformation over time for all inclinometer locations. Horizontal deformation was confirmed to occur perpendicular to the embankment alignment, decreasing in magnitude as the inclinometer locations move away from the toe. Note that inclinometers capture changes in inclination and relate horizontal deformation relative to the baseline. Understanding the true deformed shape and horizontal deformation at a specific location may be difficult if the installation yields a highly inclined or deformed guide casing for baseline reading. Horizontal deformations calculated using inclinometers proves to be useful but should still be considered an estimate of total deformation.
Figure 4.29: DTE Segment 1 INCL1 horizontal deformation (NHDOT, 2015)
INCL2, Inclinometer Data
Lateral Deformation
(Sta. 603+00, LT 89)

A Readings / Groove Orientation

Figure 4.30: DTE Segment 1 INCL2 horizontal deformation (NHDOT, 2015)
INCL3, Inclinometer Data
Lateral Deformation
(Sta. 609+00, LT 35)

A Readings / Groove Orientation

Figure 4.31: DTE Segment 4 INCL3 horizontal deformation (NHDOT, 2015)
Figure 4.32: DTE Segment 4 INCL4 horizontal deformation (NHDOT, 2015)
INCL5, Inclinometer Data
Lateral Deformation
(Sta. 609+00, LT 85)

A Readings / Groove Orientation

Figure 4.33: DTE Segment 4 INCL5 horizontal deformation (NHDOT, 2015)
4.6.6 Ground Water Table

The ground water elevation was continuously monitored for approximately a year using an observation well (OW). Based on the data, the average water table elevation is reported to be 10.6 ft (Figure 4.34). Note that prior to construction of the embankment, the water table was much lower (~7.5 ft). Rise and fluctuation in water table are due to both tidal or natural seasonal changes.
Figure 4.34: Q-B109(OW) observation well (Sta. 603+95, RT 149) (Blair, 2013)
5 SETTLEMENT ANALYSIS

5.1 PLAXIS 2D

Deformation analysis of the Dover Test Embankment was performed using PLAXIS 2D AE to predict both horizontal and vertical deformation, and pore pressure of each segment. This chapter will give a detailed description of soil stratigraphy, material properties, and methodology used to create the FEA model. Furthermore, results will be presented as they compare with measurements obtained from the various instrumentation at each test segment.

5.1.1 PLAXIS Soil Stratigraphy

The first step in the settlement analysis is to develop a soil model of the site, for each segment. The soil stratigraphy previously described in Chapter 4 was used to develop a general soil stratigraphy specific for each segment. The actual soil stratigraphy used in the PLAXIS models is shown in . Although it was specified to have all drains penetrate entirely through the soft clay, the length of drains within each segment is defined by the distance between the drainage blanket and the bottom elevation of the PV drains given in Table 4.31. A quick comparison of maximum vertical displacement and time rate of settlement were performed and yielded no apparent difference between full penetration and the elevation prescribed.

It should also be noted that when modeling each embankment segment, drains were placed to satisfy symmetric conditions and maximize the number of drains within the footprint/width of each segment per plan as specified. Actual PV drain installation may
vary and is possible to have extended beyond the toe of embankment. The normalized drain spacing per unit width of the embankment is tabulated in Table 5.2 and is reflective of the section geometry and soil stratigraphy shown in Figure 5.1.

Table 5.1: Soil layer thicknesses used in PLAXIS models

<table>
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<th>Soil Layers</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
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<td>1.5</td>
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<td>1.5</td>
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<td>11</td>
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<td>8</td>
<td>16</td>
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<td>Upper Marine</td>
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<td>7</td>
<td>8</td>
<td>4</td>
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<td>45</td>
<td>54</td>
<td>56</td>
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Table 5.2: Normalized drains modeled per unit width of embankment

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<th>PV Spacing (ft)</th>
<th>Number of Drains</th>
<th>Embankment Width (ft)</th>
<th>Number of drains per unit width of embankment</th>
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Figure 5.1: Comparison of segment stratigraphy and geometry used for PLAXIS models
5.1.2 PLAXIS Material Properties

The site material properties were assumed to be consistent throughout the entire embankment subsurface based on average results; therefore, each segment was modeled using identical soil properties unless otherwise stated. Table 5.3 summarizes the soil properties used for all PLAXIS models. Note the horizontal permeability defined in the PLAXIS model changes depending on drain spacing as discussed in Section 4.4.

5.1.3 PLAXIS Staged Construction

Each segment was constructed using the soil stratigraphy shown in and embankment geometry defined in Table 4.31. Staged construction phases used for each segment are listed in Table 5.4. For the PLAXIS analysis, the deformation control parameters, updated mesh and updated water pressures were selected to avoid unrealistic settlement results.
### Table 5.3: DTE material properties of the test embankment and subsoil

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<th>Parameter</th>
<th>Name</th>
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<th>Alluvium</th>
<th>Upper Marine</th>
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<td>Hardening Soil</td>
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<td>Soft Soil Creep</td>
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<td>Drained</td>
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<td>123</td>
<td>116</td>
<td>110</td>
<td>127</td>
<td>pcf</td>
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<tr>
<td>Soil unit weight below phreatic level</td>
<td>$\gamma_{unsat}$</td>
<td>121</td>
<td>127</td>
<td>123</td>
<td>116</td>
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<td>127</td>
<td>pcf</td>
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<td>0.5</td>
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<td>6.27E+05</td>
<td>7.31E+05</td>
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<td>-</td>
<td>8.35E+05</td>
<td>psf</td>
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<tr>
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<td>6.27E+05</td>
<td>7.31E+05</td>
<td>-</td>
<td>-</td>
<td>8.35E+05</td>
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<td>2.51E+06</td>
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<td>35</td>
<td>35.2</td>
<td>25</td>
<td>22.5</td>
<td>40</td>
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<td>0</td>
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<td>Van Gencich</td>
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<td>Van Gencich</td>
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<td>-</td>
<td>-</td>
<td>4.00E-05</td>
<td>2.00E-05</td>
<td>-</td>
<td>ft/day</td>
</tr>
<tr>
<td>Vertical permeability</td>
<td>$k_{y}$</td>
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<td>-</td>
<td>-</td>
<td>7.05E-04</td>
<td>1.50E-04</td>
<td>-</td>
<td>ft/day</td>
</tr>
<tr>
<td>Change in permeability</td>
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<td>1.00E+15</td>
<td>1.00E+15</td>
<td>1.70E-01</td>
<td>1.00E+15</td>
<td>-</td>
<td>1.00E+15</td>
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<td>Strength reduction factor</td>
<td>$R_{int}$</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
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<td>$K_{0}$ determination</td>
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<td>Automatic</td>
<td>Automatic</td>
<td>Manual</td>
<td>Manual</td>
<td>Automatic</td>
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<td>Over-consolidation ratio</td>
<td>$OCR$</td>
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<td>1</td>
<td>6</td>
<td>3.7</td>
<td>1.2</td>
<td>6.2</td>
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<tr>
<td>Pre-overburden pressure</td>
<td>$POP$</td>
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<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>0</td>
<td>-</td>
</tr>
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### Table 5.4: Segment specific staged construction phases

<table>
<thead>
<tr>
<th>Calculation Phase</th>
<th>Calculation Type</th>
<th>Loading Type</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>Initial Phase</td>
<td>K0 procedure</td>
<td>Staged Construction</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>Place Drainage Blanket</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>2</td>
<td>days</td>
</tr>
<tr>
<td>Install PV Drains</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>1</td>
<td>days</td>
</tr>
<tr>
<td>Waiting Time</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>13</td>
<td>12</td>
<td>11</td>
<td>10</td>
<td>9</td>
<td>days</td>
</tr>
<tr>
<td>Place Embankment (+12 ft Fill Height)</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>40</td>
<td>days</td>
</tr>
<tr>
<td>Waiting Time</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>45</td>
<td>NA</td>
<td>days</td>
</tr>
<tr>
<td>Place Embankment (+6 ft Fill Height)</td>
<td>Consolidation</td>
<td>Staged Construction</td>
<td>NA</td>
<td>NA</td>
<td>NA</td>
<td>7</td>
<td>NA</td>
<td>days</td>
</tr>
<tr>
<td>90 % Consolidation</td>
<td>Consolidation</td>
<td>Minimum Excess Pore Pressure</td>
<td>6728</td>
<td>5589</td>
<td>9543</td>
<td>11201</td>
<td>10782</td>
<td>days</td>
</tr>
<tr>
<td><strong>Total Time</strong></td>
<td></td>
<td></td>
<td>6784</td>
<td>5644</td>
<td>9597</td>
<td>11306</td>
<td>10834</td>
<td>days</td>
</tr>
</tbody>
</table>

- **Units**

- **Total Time**

- **Years**
5.1.4 Discretization

All models were discretized using a medium mesh distribution as shown in Figure 5.2. The medium mesh was chosen as it was found to give satisfactory results without further refinement, especially when used on a symmetric plane strain model as demonstrated later in Section 5.1.8.1. Table 5.5 shows the number of elements and nodes generated by a medium mesh for each segment.

![Figure 5.2: PLAXIS DTE Segment 1 discretization (medium mesh distribution)](image)

Table 5.5: DTE plane strain symmetric segment model elements and nodes

<table>
<thead>
<tr>
<th>Element Distribution</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of Elements</td>
<td>1649</td>
<td>1551</td>
<td>1465</td>
<td>1236</td>
<td>1452</td>
</tr>
<tr>
<td>Number of Nodes</td>
<td>13427</td>
<td>12719</td>
<td>11983</td>
<td>10129</td>
<td>11788</td>
</tr>
</tbody>
</table>
5.1.5 Boundary Conditions

Boundary conditions in Plaxis 2D are concentrated to groundwater flow conditions. There are boundary conditions for static deformation analysis that can be applied to elements in one or two directions but the boundaries are often non-physical and do not affect the surrounding deformation behavior. In other words, the boundaries are kept far away from the model area in question. Plaxis suggests setting large limits for embankment modeling as shown in Figure 5.3. By doing this, the program can essentially ignore any deformation boundary conditions local to the embankment. For groundwater flow, Plaxis offers closed, inflow, outflow, head, infiltration, and seepage boundary conditions that associate certain flow conditions with elements. It is assumed that the seepage boundary condition will be used during analysis. This is a typical method as Plaxis 2D AE sets this condition as a default. Drain boundary conditions act as vertical seepage boundaries that allow water to travel upwards or downwards depending on pore water pressure differential.

All models used the same boundary conditions. The \( y_{\text{min}} \) boundary is fixed, as shown in Figure 5.3, while all other planes are free to deform vertically and horizontally. Similarly the, \( x_{\text{min}} \) boundary was prescribed to have closed groundwater flow conditions while all other boundaries (\( x_{\text{max}}, y_{\text{max}}, \) and \( y_{\text{min}} \)) remained open.
5.1.6 Points for Curves

Before the model can be evaluated, PLAXIS requires the user to select points for curves. Within the output window, the connectivity plot is shown where nodes and/or stress points may be selected near desired locations within the mesh. Figure 5.4 shows the points selected for plots within Segment 1 and indicates the designation to match the field instrumentation. Coordinates used for all PLAXIS models are reported in Table 4.32. Note all x-coordinates are relative to centerline of embankment but do not directly correlate with station offsets, and y- coordinates are reported in elevation. The model was drawn in terms of elevation for ease of comparison.
5.1.7 Degree of Consolidation Calculation

There are three calculation types for consolidation analysis within PLAXIS: *Staged construction*, *Minimum excess pore pressure*, and *Degree of consolidation*. In order to achieve 90% consolidation, typically it would be suggested to use *Degree of consolidation*. An issue was identified where calculation steps continued beyond 90% consolidation, resulting in error messages and a broken mesh.
In order to achieve 90% consolidation, the following was suggested by PLAXIS:

- Run a calculation of the model
- After calculation is complete go to *phases>* edit phases* and view the Start-from-phase preceding the 90% consolidation phase
- Under *Reached values*, view the absolute excess pore pressure value (*P*<sub>max</sub>)
- Change the *Degree of Consolidation* phase to *Minimum excess pore pressure*
- Manually calculate the minimum excess pore pressure to be entered as P-stop using Equation 3.3 while specifying the degree of consolidation (*U*) to be 90

\[ |P - stop| = (100\% - U) \times P_{\text{max, previous phase}} \]

The *P*-stop values determined for FEA calculations are shown in Table 5.6. Note that the *P*<sub>max</sub> value for Segment 4 reflects the maximum pore pressure developed during the second phase of construction (additional 6 ft fill placement).

**Table 5.6: P-stop values used for 90% consolidation calculation**

<table>
<thead>
<tr>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td><em>P</em>&lt;sub&gt;max&lt;/sub&gt;-Reached max p&lt;sub&gt;excess&lt;/sub&gt;</td>
<td>1390</td>
<td>1585</td>
<td>1348</td>
<td>1514</td>
<td>1129</td>
</tr>
<tr>
<td><em>P</em>-stop 90% Consolidation</td>
<td>139.0</td>
<td>158.5</td>
<td>134.8</td>
<td>151.4</td>
<td>112.9</td>
</tr>
</tbody>
</table>

### 5.1.8 Controlled Model Comparison

Before segments are compared to the geotechnical monitoring instrumentation, certain model characteristics such as symmetry, drain spacing, plane strain permeability corrections, and Soft Soil (SS) vs. Soft Soil Creep (SSC) will be considered to show the effects on a baseline model. All properties previously described for Segment 1, unless
otherwise stated, will serve as the control. All analysis reported is with respect to vertical deformation (settlement) which occurs below the sand blanket and directly below the centerline of the embankment. This point is designated Point A. All models are assumed to be Soft Soil Creep models unless labeled otherwise.

5.1.8.1 Symmetric vs. Non-Symmetric

Although the test embankment is not perfectly symmetric and the soil layers are not perfectly horizontal, the model was assumed to be symmetric and thus only half of the embankment and horizontal soil layering were used in developing the finite element model (Figure 5.5). The symmetric model with medium mesh distribution for segment one is comprised of 1649 15-node elements with 13427 nodes. In contrast, the full model with medium mesh distribution is comprised of 1800 elements with 14777 nodes. Therefore, a model half the size having nearly the same number of elements and nodes would be expected to produce a more accurate prediction for a specified point. Figure 5.6 shows vertical displacement for both models at Point A. No apparent difference is observed between symmetric and non-symmetric models. The symmetric model was chosen for all additional analysis since calculation time is significantly reduced and more nodes are available.
Figure 5.5: Symmetric vs. Non-symmetric model
Figure 5.6: PLAXIS settlement of Point A for symmetric and non-symmetric conditions (PV= 6 ft)
5.1.8.2 Soft Soil vs. Soft Soil Creep

Within PLAXIS both Soft Soil (SS) and Soft Soil Creep (SSC) models were used to calculate settlement under the centerline of embankment using the exact same soil properties, staged construction, and geometry. The only difference is that the SSC model requires the input of the secondary compression index in order to estimate creep. In regards to the magnitude and time rate of settlement, Figure 5.7 denotes a clear difference between the two models.

Both models have similar trends; however, the soft soil model yields approximately half the settlement (1.1 ft) observed in the SSC model (2.1 ft) and therefore reaches 90% consolidation at a faster rate. For the cases with PV drains, the SS model predicts the soil will reach 90% consolidation in approximately 2000 days whereas the SSC model predicts 90% consolidation in about 6700 days (shown by the last point on each curve). It may be possible that the SS model accurately predicts the time rate of consolidation to reach 90% but since the embankment has already settled approximately 1.5 ft at Segment 1 it is apparent that the SS model does not accurately predict the degree of settlement in this case.
Figure 5.7: PLAXIS settlement of Point A for SS and SSC models (PV= 6 ft, Symmetric)
5.1.8.3 Corrected vs Uncorrected Permeability

The effect of smear zone and plane strain conditions can be accounted for by correcting the horizontal permeability coefficient \((k_h)\). This section demonstrates how these corrections affect the rate of settlement at Point A below the centerline of the embankment.

Figure 5.8 shows that all models achieve the same amount of settlement at 90% consolidation. The difference strictly relates to the rate of consolidation which in this case is governed by permeability. Decreasing the permeability of the material effectively decreases the rate of consolidation. The uncorrected permeability is reflective of a drain extending infinitely in the \(z\)-direction, thus inducing horizontal flow perpendicular to the \(z\)-plane (as opposed to radial) and effectively increasing the permeability of the soil. Although this is the essential reason for placing drains, flow is truly radial and limited to the equivalent soil cylinder diameter generated by the drain cross-section.

Correcting the drains for plane strain analysis as suggested by Indraratna et al. (2005) yields \(k_{h,ps}\). The plane strain correction increase the time to reach 90% consolidation from the uncorrected value by approximately 1460 days. Applying the smear zone correction for remolded clay in addition to the plane strain correction \((k_{r,ps})\) shifts the curve slightly but increases the time to reach 90% consolidation by an additional 1.5 years. This combination of corrections was applied to all models shown in the following FEA results section.
Figure 5.8: PLAXIS settlement of Point A analysis of permeability corrections (PV= 6 ft, Symmetric)
5.1.8.4 Effect of Drain Spacing

The Dover Test Embankment is composed of 5 segments with two transitional segments at either end. In total, 4 different drain spacings are used (6 ft, 8 ft, 10 ft, and 12 ft); however, the 8 ft drain spacing within the transitional segments are not monitored. This chapter is concerned with the general trend of increasing drain spacing from 6 ft, to 10 ft, to 14 ft, all in a triangular pattern.

As one would expect, the rate of consolidation decreases as drain spacing increases but the total amount of settlement remains the same. An important factor to consider is the efficiency of the drains, where efficiency is considered to be a measure of time rate of consolidation compared to a configuration without drains. Figure 5.9 shows the time rate of consolidation for all drain spacings. All curves show a similar trend but the increase in spacing results in a decreases of the overall rate of consolidation.
Figure 5.9: PLAXIS settlement of Point A analysis of drain spacing
5.1.9 Finite Element Analysis Results

The PLAXIS FEA models generated for each segment were compared to field instrumentation measurements for validation. Differing settlement is observed in each segment as a result of embankment geometry and thickness of compressible soil layers. Each segment was calculated to 90% consolidation and the selected points, previously shown in Table 4.32, were plotted as a function of time. Locations of the vibrating wire piezometers were also considered for comparison with pore water pressure observed in the field. Deformed meshes at 90% consolidation for all segments may be found in Appendix D (Segment 1 shown in Figure 5.10). Vertical deformation contours for each segment are reported in Appendix E (Segment 1 shown in Figure 5.11) and horizontal deformation contours are reported in Appendix F (Segment 1 shown in Figure 5.12). It should be noted that irregular “wavy” horizontal deformation contours were observed in all horizontal deformation results generated by PLAXIS. It is apparent the PV drains interrupt the contours and create the irregularity. Although plane strain corrections were applied to the PV drains, they only correct the permeability and flow conditions. The correction does not change the plane strain condition applied PV drains in 2D space, thus creating a vertical obstacle for horizontal deformation.
Figure 5.10: PLAXIS Segment 1 (Drains) deformed mesh at 90% consolidation
Figure 5.11: PLAXIS Segment 1 (Drains) vertical deformation at 90% consolidation
Figure 5.12: PLAXIS Segment 1 (Drains) horizontal deformation at 90% consolidation
5.1.9.1 Settlement Platforms

Settlement platforms located directly below the centerline of the embankment are designated Point A. Maximum settlement for each segment and the time to reach 90% consolidation is reported in Table 5.7. The settlement platform data available reflect settlement observed in the field from October 8, 2012 to December 1, 2015. Each segment includes two platforms. The settlement analyses using PLAXIS are also plotted alongside the field data as shown in Figure 5.13. Overall, the predictions are in good agreement with the field observations. Segment 1 FEA prediction is considered the most accurate model since the material properties developed are primarily based on testing within Segment 1. The Segment 2 model also agrees well with the field data, while Segments 3, 4, and 5 slightly underpredict the rate of settlement. Difference in rate and degree of settlement in the FEA model predictions is possibly due to change in subsurface soil stratigraphy, including thickness and position of the compressible zones.

Figure 5.14 shows the FEA settlement predictions with drains at the settlement platform locations for reaching 90% consolidation. Based on the PLAXIS model, Segment 1 and 2 are estimated to reach 90% consolidation after 18.4 and 15.3 years, respectively. Segment 4 is estimated to take the longest time to reach 90% consolidation (30.7 years); however, it is also estimated to have the largest settlement (3.0 ft).
Table 5.7: PLAXIS Estimated settlement and time to reach 90% consolidation with and without PV drains

<table>
<thead>
<tr>
<th>Drains</th>
<th>Segment 1</th>
<th>Segment 2</th>
<th>Segment 3</th>
<th>Segment 4</th>
<th>Segment 5</th>
<th>Units</th>
</tr>
</thead>
<tbody>
<tr>
<td>No Drains</td>
<td>46.5</td>
<td>33.6</td>
<td>60.9</td>
<td>51.3</td>
<td>45.2</td>
<td>yrs</td>
</tr>
<tr>
<td>Total Time to reach 90% Consolidation from Start of Fill Placement</td>
<td>18.4</td>
<td>15.3</td>
<td>26.1</td>
<td>30.7</td>
<td>29.5</td>
<td>yrs</td>
</tr>
<tr>
<td>Max Settlement at 90%</td>
<td>2.1</td>
<td>1.8</td>
<td>2.4</td>
<td>3.0</td>
<td>1.7</td>
<td>ft</td>
</tr>
<tr>
<td>No Drains</td>
<td>2.1</td>
<td>1.8</td>
<td>2.4</td>
<td>2.9</td>
<td>1.7</td>
<td>ft</td>
</tr>
</tbody>
</table>
Figure 5.13: DTE PLAXIS FEA predictions
Figure 5.14: DTE PLAXIS FEA predictions to 90% consolidation
5.1.9.2 Pore Water Pressure from Vibrating Wire Piezometers

Two vibrating wire piezometers were installed; one in Segment 1 at an elevation of -29 ft (VWPZ1) and one in Segment 4 at an elevation of -32.5 ft (VWPZ2). Given stations and offsets, analysis points were selected for plots accordingly as reported in Table 4.32. These points were designated as Point H by PLAXIS.

Plots of pore water pressure over time are shown for VWPZ1 and VWPZ2 in Figure 5.15 and Figure 5.16 along with the progression of fill placement and settlement over time. FEA predictions are plotted for comparison. As the test embankment is placed, pore water pressure increases until it reaches its peak simultaneously with the completion of fill placement and then slowly and gradually dissipates over time.

In both cases, the trend of pore water pressure over time matches but the pore water pressures predicted by the FEA models are consistently greater than values observed in the field. The measured pore pressures are likely affected by their proximity to the PV drains.

Consider a vibrating wire piezometer installed at midpoint depth of Segment 1 if no drains were installed (Figure 5.17). Excess pore water pressure ($p_{\text{excess}}$) gradually decreases with increasing distance perpendicular to embankment alignment and even more gradually with depth. Now consider the same configuration but with drains (Figure 5.18). The $p_{\text{excess}}$ still changes gradually with depth; however, the PV drains create a greater variability of $p_{\text{excess}}$ in the lateral direction, decreasing significantly within close proximity to the drains. It is therefore possible that the piezometers installed in both
Segments 1 and 4 are located near a drain thus influencing the pore water pressure response. Figure 5.19 demonstrates PLAXIS predictions of how the proximity of the piezometer to the PV drain affects the pore water pressure readings. The FEA model indicates the point selected for pore water pressure analysis is in good agreement if the piezometer is located at a lateral distance 0.75 ft to 1.5 ft from a PV drain. Plaxis may be able to accurately model pore pressures for the case of PV drains, but it is highly dependent on accurate groundwater table elevation and the location of the point of interest with respect to the instrumentation.

Figure 5.20 and Figure 5.21 show excess pore water pressure contours at 90% consolidation for a configuration without and with PV drains. When viewing figures from PLAXIS, the reader must use caution in viewing the colored scales before direct comparisons can be made between figures. These figures show how the PV drains fully dissipate excess pore pressure below the embankment where the drains are located while away from the toe of the embankment pore pressure dissipates at a slower rate. Without drains, the maximum excess pore water pressure remains below the center of embankment and relies on both horizontal and vertical flow of pore water to allow dissipation. A similar trend is expected in all segments of the DTE but to varying degrees and contours depending on drain spacing, embankment geometry (loading) and soil stratigraphy.
Figure 5.15: Segment 1 fill placement timeline with piezometer and settlement platform response
Figure 5.16: Segment 4 fill placement timeline with piezometer and settlement platform response
Figure 5.17: Segment 1 (no drains) excess pore water pressure generated immediately after placement

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

Maximum value = $0.1987 \times 10^{-3}$ lbf/ft$^2$ (Element 14 at Node 10577)

Minimum value = -1327 lbf/ft$^2$ (Element 764 at Node 12273)
Figure 5.18: Segment 1 (drains) excess pore water pressure generated immediately after placement
Figure 5.19: Effect of piezometer proximity to PV drain
Figure 5.20: Segment 1 (no drains) excess pore water pressure at 90% consolidation
Figure 5.21: Segment 1 (drains) excess pore water pressure at 90% consolidation
5.1.9.3 Subsurface Settlement Points

Subsurface settlement points (SSP) or borros points were only installed in Segments 1 and 4. The borros settlement points were placed to observe the settlement at the approximate top of the upper marine layer (Point F) and the top of the lower marine layer (Point G). Points were selected for analysis based on reported elevations for each SSP (Table 4.32).

The FEA model for both Segment 1 (Figure 5.22) and Segment 4 (Figure 5.23) agree well with the settlement observed in the field. While the settlement platform prediction for Segment 1 proves to be in excellent agreement, the FEA prediction for the subsurface settlement points SSP1 and SSP2 slightly overestimates settlement and settlement rate observed in the field. The FEA prediction for Segment 4 captures the settlement of SP7 and SP8 fairly well and provides an accurate prediction of the field settlement observed for SSP3 and SSP4. In contrast to the Segment 1 FEA model, Segment 4 overestimates early settlement and slightly underestimates settlement observed after approximately 200 days.

While the Segment 1 model slightly overestimated settlement and time rate of consolidation, Segment 4 showed a slight underestimate, but overall the PLAXIS FEA models function well to predict the settlement observed and any variability in accuracy may be due to the assumption of perfectly horizontal and cohesive material layers.
Figure 5.22: Segment 1 settlement below centerline at increasing depth
Segment 4 Subsoil Settlement at Approx. Centerline of Embankment

Figure 5.23: Segment 4 settlement below centerline at increasing depth
5.1.9.4 Surface Settlement Points

Surface settlement points were installed to monitor the settlement at the ground surface. Points of interest include the crest of slope (Point B), toe of slope (Point C), 20 ft from the toe (Point D), and 40 ft from the toe (Point E). The instrumentation continues to provide surface settlement points at 60 ft and 80 ft from the toe of embankment; however, PLAXIS only allows for a maximum of 10 points to be selected for curves per calculation. The other surface settlement points were determined to be less desirable for calibration modeling since field data indicates they captured little to no apparent vertical settlement (Blair, 2013). These points could be used to establish the extent of the influence zone of settlement and may be considered in the future.

Vertical deformations for the Segment 1 FEA model points are plotted along with the field data and against time (Figure 5.24). At the crest of the slope (DMP1), the FEA model for Segment 1 overpredicts settlement and the rate of settlement by approximately 40%. Additionally, the model predicts 60% more settlement at the toe (DMP2); however, the model is in general agreement with DMP3 and DMP4 which are located 20 and 40 ft away from the toe in Segment 1, respectively.

The Segment 4 FEA model provides a better agreement with the settlement observed in the field. The FEA model at the crest of the slope (DMP25) underpredicts settlement and rate of settlement by approximately 20%. Settlement observed at the toe of slope (DMP26) is highly variable; however, the overall trend does correlate with the model prediction. The model is in general agreement with DMP27 and DMP28 which are located 20 and 40 ft away from the toe in Segment 4, respectively.
Supplemental comparisons between segments of each surface settlement point location were generated in the PLAXIS Output window to show the effect of settlement predicted by the FEA model at interval distances perpendicular to the embankment alignment. These plots may be found in Appendix G.
Figure 5.24: Segment 1 relationship of settlement perpendicular to alignment
Figure 5.25: Segment 4 relationship of settlement perpendicular to alignment
5.1.9.5 Horizontal Deformation from Inclinometers

The progression of horizontal displacement was estimated by reading inclinometers at regular time intervals. The PLAXIS output menu allows the user to define cross-sections within the model window to examine stress or displacement over the length of the cross-section. Cross-sections were drawn at the approximate distance from toe to match that of the inclinometer locations. The model view of horizontal displacement was then selected and the data was compared with the inclinometer results for INCL1 and INCL3 (Figure 5.26 and Figure 5.27, respectively).

From the figures it is apparent that the FEA model shows a similar general trend as the field data but overestimates the degree of horizontal displacement. Table 5.8 compares the observed maximum horizontal displacement at INCL1 and INCL3 locations with PLAXIS predictions. INCL 2, 4, and 5 were spaced at increasing increments from the toe of the embankment and exhibited significantly less displacement.

*Table 5.8: Comparison maximum horizontal displacement at INCL1 and INCL3 locations*

<table>
<thead>
<tr>
<th>Days After Start of Fill Placement</th>
<th>Maximum Horizontal Displacement (in.)</th>
<th>INCL1</th>
<th>INCL3</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>Field</td>
<td>FEA</td>
</tr>
<tr>
<td>65</td>
<td></td>
<td>0.4</td>
<td>1.1</td>
</tr>
<tr>
<td>288</td>
<td></td>
<td>0.7</td>
<td>1.7</td>
</tr>
<tr>
<td>884</td>
<td></td>
<td>1.3</td>
<td>2.4</td>
</tr>
</tbody>
</table>
Figure 5.26: Horizontal displacement of Segment 1 INCL1
Figure 5.27: Horizontal displacement of Segment 4 INCL3
6 SUMMARY, CONCLUSIONS, AND RECOMMENDATIONS

6.1 Summary

In 2003, the New Hampshire Department of Transportation began studies for a road network expansion of the Spaulding Turnpike located in Newington and Dover, NH. Previous construction and publications highlighted the presence of a thick layer of soft marine clay known as the Presumpscot Formation. This formation is documented as having low shear strength and bearing capacity in addition to having high compressibility and excessive settlement characteristics. To avoid differential settlement of the new highway alignment, embankment preloads with prefabricated vertical drains were selected to be used for ground improvement prior to constructing the final alignment.

A fully instrumented test embankment was proposed by the NHDOT to evaluate the effectiveness of different embankment geometry, drain length, and drain spacing in regards to settlement. The installed geotechnical monitoring instrumentation includes settlement platforms, borros points, surface settlement points, vibrating wire piezometers, and slope inclinometers. A test embankment consisting of approximately 200 ft segments, each with a different geometry, was constructed between October 2012 and January 2013.

An extensive testing program was developed by the NHDOT and the University of New Hampshire to evaluate the properties and behavior of the clay prior to placing the test
embankment. Testing methods such as DMT, CPTu, FVT, and piston sampling were performed by Getchell (2013) in the summer of 2012, with a majority of the testing actually being performed within the Segment 1 location. Getchell (2013) also performed laboratory consolidation testing on Shelby tube piston samples between November 2012 and February 2013. Data reduction of the in situ testing performed by Getchell (2013) provided several soil properties for the marine clay and gave detailed insight into the soil stratigraphy of the site.

The option of finite element analysis was explored using PLAXIS 2D AE, in which soil stratigraphy and average soil layer properties were used to predict consolidation settlement and the time-rate of consolidation. The model was then compared with data collected from the geotechnical field instrumentation. The vertical displacement observed from settlement platforms, borros points, and surface settlement points were compared with vertical displacement estimated using the Soft Soil Creep model available in PLAXIS 2D. Pore water pressure observed from vibrating wire piezometers was also included for comparison and lastly, horizontal displacements found from inclinometer results were compared with results from the PLAXIS models.

### 6.2 Conclusions

A comparison of FEA results with field data found the Soft Soil Creep predictions of pore water dissipation and vertical displacements to be in good agreement with those observed in the field while FEA results for horizontal displacements were found to overestimate those observed in the field. An inconsistency is apparent from the good
results generated for vertical deformation compared with horizontal deformation and likely caused by a discrepancy created by the assumptions of plane strain.

Settlement platforms within Segment 1 show approximately 1.5 ft of settlement to date and indicate no apparent change in consolidation rate over the past year. The FEA model predicts settlement at 90% consolidation for Segment 1 to be approximately 2 ft which indicates an additional 0.5 ft of settlement will occur over the next 16 years. This is thought to be an overestimate of the time to reach 90% consolidation; however, at this time it can neither be refuted nor confirmed since 90% consolidation has yet to be established from field data.

Subsurface settlement predictions by the FEA model is are also good agreement with the field data of borros points. In addition, surface settlement points at the crest of slope, toe of slope, 20 ft from toe, and 40 ft from toe were compared with model points at said locations and shown to be in good agreement. The PLAXIS model was found to overpredict settlement at the crest of slope in Segment 1 and consistently overpredict settlement at the toe in both Segment 1 and 4; however, this may be explained by variations in thickness of the compressible soil layers across the various segments. The progression of horizontal displacement in the field was captured using inclinometers. FEA predictions of horizontal displacement at approximate inclinometer locations show a general agreement with the shape of deformation, but indicate 50% more deformation than what is being observed.
In conclusion, it was found that the overall deformation of the FEA model correlates well with field observations to date. Settlement points located closer to the centerline of embankment show better agreement while other predictions further from the centerline may be considered valid estimates. It is also recognized that the mesh is less refined as distance from centerline increases and that choosing a finer mesh may be required in order to accurately predict both vertical and horizontal deformations outside the embankment geometry.

In closing, finite element analysis using PLAXIS 2D AE software proves to be a useful tool in understanding the behavior of clay consolidation; however, as with any software extreme care and caution must be used before relying on the results for future applications. This thesis serves as a basis for future development and investigation of modeling a test embankment with PV drains on soft clay and should be regarded as an estimate of soil behavior.

6.3 Recommendations

At this time, the FEA model developed as a part of this thesis is suggested to be used as an initial model for thick silty clay deposits located in the coastal areas of Maine and New Hampshire as defined by the extent of the Presumpscot formation. The work presented in this thesis reflects average soil properties found using several different test methods over the extent of the site. In situ testing such as DMT or CPTu is still recommended to obtain a site specific soil profile and corresponding material properties. Warranted adjustments of soil properties may then made accordingly to more accurately predict soil behavior at future sites. Based on the field experience gained at the DTE
site, the ease, simplicity and ruggedness of the DMT suggests that its use is well-suited for future investigations by the NHDOT where marine deposits are identified in conventional geotechnical test borings.

Secondary compression index was not known and was based on findings from Ladd (1972). The values used are considered to be a good estimate; however, if more sampling of clay within the DTE is performed then it is suggested to run a complete consolidation test past 90% consolidation for the determination of the secondary compression index.

An investigation of remolded permeability is suggested to confirm the assumptions of permeability used to explain the smear effect within the FEA model. Well resistance and discharge capacity were assumed to be negligible when modeling; however, these drain properties may be a contributing factor in the delayed rate of consolidation observed.

Apparent cohesion and effective friction angle, identified by PLAXIS as effective strength parameters, were also assumed and should be confirmed using triaxial testing methods. In addition, permeability was the only parameter corrected for plane strain conditions; however, other parameters may also need to be corrected for plane strain. Correcting the effective friction angle for plane strain conditions could provide enough additional friction resistance to reduce the amount of horizontal displacement observed, thus providing a better agreement with field observations.
Eventually the test embankment will be cut to final grade for the placement and construction of the Exit 6S on-ramp. Future studies should include continuous monitoring of the site, if possible, to investigate the effect of unloading.

Lastly, the Soft Soil model was not extensively explored as part of this thesis; however, the SS model may give new insight into the time-rate and 90% degree of consolidation. Warranted adjustments of soil properties and soil stratigraphy may be made to explore a fit of the Soft Soil model to the field data. Additionally, the advanced NGI-ADP model is equally rated by PLAXIS as an applicable model for soft soil and embankment loading. Parameters required by this model should also be considered for comparison between models.
REFERENCES


APPENDIX A: CPT SBTN PROFILES
Figure A-1: Segment 1 Q-B220 SBTn generated by CPeT-IT (Sta. 603+37, RT 16)
Figure A-2: Segment 1 Q-B221 SBTn generated by CPet-IT (Sta. 603+47, RT 11)
Figure A-3: Segment 2 Q-B222 SBTn generated by CPeT-IT (Sta. 604+92, LT 6)
Figure A-4: Segment 2 Q-B223 SBTn generated by CPeT-IT (Sta. 605+04, LT 6)
Figure A-5: Segment 3 Q-B224 SBTn generated by CPeT-IT (Sta. 606+76, CL)
Figure A-6: Segment 3 Q-B225 SBTn generated by CPeT-IT (Sta. 606+86, CL)
Figure A-7: Segment 4 Q-B226 SBTn generated by CPeT-IT (Sta. 609+05, LT 10)
Figure A-8: Segment 4 Q-B227 SBTn generated by CPeT-IT (Sta. 609+09, CL)
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APPENDIX B: DTE INSTRUMENT LOCATIONS
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Figure B-2: DTE Segment 2 plan view of instrument locations (Blair, 2013)
Test Embankment - Segment 3
(PV-Drain Spacing = 10 feet, Marine deposit not fully penetrated and Approximate Fill Height = 12 feet)

Figure B-3: DTE Segment 3 plan view of instrument locations (Blair, 2013)
Test Embankment - Segment 4
(PV-Drain Spacing = 10 feet and Approximate Fill Height = 18 feet)

Figure B-4: DTE Segment 4 plan view of instrument locations (Blair, 2013)
Figure B-5: DTE Segment 5 plan view of instrument locations (Blair, 2013)
APPENDIX C: CPET-IT DATA REDUCTION
Figure C-1: CPeT-IT data reduction formulas

- Unit Weight, \( g \) (kN/m³) =
  \[
g = g_w \left( 0.27 \cdot \log(R_T) + 0.36 \cdot \log \left( \frac{Q_s}{P_a} \right) + 1.23 \right)
\]
  where \( g_w \) = water unit weight

- Permeability, \( k \) (m/s) =
  \[
  I_c < 3.27 \text{ and } I_c > 1.00 \text{ then } k = 10^{0.065 \cdot 0.94 \cdot I_c}
  
  I_c \leq 4.00 \text{ and } I_c > 3.27 \text{ then } k = 10^{-5.52 \cdot 1.27 \cdot I_c}
  
- \( N_{60} \) (blows per 30 cm) =
  \[
  N_{60} = \left( \frac{Q_s}{P_a} \right) \cdot \frac{1}{10^{0.1128 \cdot 0.23574}}
  
  N_{60} = Q_s \cdot \frac{1}{10^{0.1128 \cdot 0.23574}}
  
- Young's Modulus, \( E_s \) (MPa) =
  \[
  (q_t - \sigma_v) \cdot 0.015 \cdot 10^{0.551 \cdot \log_{10}(F_T)}
  
  \text{ (applicable only to } I_c < I_{c,correct})
  
- Relative Density, \( D_r \) (%):
  \[
  100 \cdot \left( \frac{Q_s}{P_a} \right) \quad \text{ (applicable only to } \text{SPT}\text{n: 5, 6, 7 and 8 or } I_c < I_{c,correct})
  
- State Parameter, \( \psi \) =
  \[
  \psi = 0.56 - 0.33 \cdot \log(Q_s)
  
- Peak drained friction angle, \( \phi^* \) (°) =
  \[
  \phi = 17.60 + 11 \cdot \log(Q_s, F_T)
  
  \text{ (applicable only to } \text{SPT}\text{n: 5, 6, 7 and 8})
  
- 1-D constrained modulus, \( M \) (MPa) =
  \[
  \text{If } I_c > 2.20
  \]
  \[
  \alpha = 14 \text{ for } Q_{ts} > 14
  
  \alpha = Q_{ts} \text{ for } Q_{ts} \leq 14
  
  M_{CPT} = 0.188 \cdot 10^{0.551 \cdot \log_{10}(F_T) + 0.551 \cdot \log_{10}(F_T)}
  
  \text{If } I_c \leq 2.20
  
  M_{CPT} = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.551 \cdot \log_{10}(F_T) + 0.551 \cdot \log_{10}(F_T)}
  
- Small strain shear Modulus, \( G_0 \) (MPa) =
  \[
  G_0 = (q_t - \sigma_v) \cdot 0.0188 \cdot 10^{0.551 \cdot \log_{10}(F_T) + 0.551 \cdot \log_{10}(F_T)}
  
- Shear Wave Velocity, \( V_s \) (m/s) =
  \[
  V_s = \left( \frac{G_0}{\rho} \right)^{0.5}
  
- Undrained peak shear strength, \( S_u \) (kPa) =
  \[
  S_u = \frac{(q_t - \sigma_v)}{N_{60}}
  
  \text{ (applicable only to } \text{SPT}\text{n: 1, 2, 3, 4 and 9 or } I_c > I_{c,correct})
  
- Remolded undrained shear strength, \( S_u(rem) \) (kPa) =
  \[
  S_u(rem) = f_s \quad \text{ (applicable only to } \text{SPT}\text{n: 1, 2, 3, 4 and 9 or } I_c > I_{c,correct})
  
- Overconsolidation Ratio, OCR =
  \[
  k_{OCR} = \left[ \frac{Q_s^{0.25}}{0.25 \cdot 10.5 \cdot 0.7 \cdot \log(F_T)} \right]^{125}
  
  \text{ or user defined}
  
  \text{OCR = } k_{OCR} \cdot Q_{ts}
  
  \text{ (applicable only to } \text{SPT}\text{n: 1, 2, 3, 4 and 9 or } I_c > I_{c,correct})
  
- In situ Stress Ratio, \( K_o \) =
  \[
  K_o = (1 - \sin \theta') \cdot OCR^{1/3}
  
  \text{ (applicable only to } \text{SPT}\text{n: 1, 2, 3, 4 and 9 or } I_c > I_{c,correct})
  
- Soil Sensitivity, \( S_t \) =
  \[
  S_t = \frac{N_{60}}{F_T}
  
  \text{ (applicable only to } \text{SPT}\text{n: 1, 2, 3, 4 and 9 or } I_c > I_{c,correct})
  
- Effective Stress Friction Angle, \( \phi' \) (°) =
  \[
  \phi' = 29.5^\circ \cdot B_T^{0.125} \cdot \left( 0.256 + 0.336 \cdot B_T \cdot \log Q_s \right)
  
  \text{ (applicable for } 0.10 < B_T < 1.00)\]
APPENDIX D: PLAXIS 2D DEFORMED MESH
Figure D-1: PLAXIS Segment 1 (Drains) deformed mesh at 90% consolidation
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Figure E-2: PLAXIS Segment 2 (Drains) vertical deformation at 90% consolidation
Figure E-3: PLAXIS Segment 3 (Drains) vertical deformation at 90% consolidation
Figure E-4: PLAXIS Segment 4 (Drains) vertical deformation at 90% consolidation
Figure E-5: PLAXIS Segment 5 (Drains) vertical deformation at 90% consolidation
Figure F-1: PLAXIS Segment 1 (Drains) horizontal deformation at 90% consolidation
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Figure F-3: PLAXIS Segment 3 (Drains) horizontal deformation at 90% consolidation
Figure F-4: PLAXIS Segment 4 (Drains) horizontal deformation at 90% consolidation
Figure F-5: PLAXIS Segment 5 (Drains) horizontal deformation at 90% consolidation
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Figure G-1: Surface settlement Point B (Approx. Crest of Slope)
Figure G-2: Surface settlement Point C (Approx. Toe of Slope)
Figure G-3: Surface settlement Point D (Approx. 20 ft from toe of slope)
Figure G-4: Surface settlement Point E (Approx. 40 ft from toe of slope)