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Evaluation of Critical Shear Strength of Soils in Nebraska Based on Revised CPT

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16. Abstract

This study evaluated the "Fully Softened Shear Strength (FSSS)" of soils in Nebraska based on a revised Cone Penetration Test (CPT) method. FSSS is a unique strength condition that soils show substantially reduced strength during rainy seasons, eventually causing failure of slopes and other structures.

The revised CPT was adapted to inject water at targeted depths, inducing controlled moisture infiltration and capturing real-time soil softening over designated intervals of 30, 60, or 90 minutes. Through tip resistance (q_c) and sleeve friction (f_s) measurements under "in-situ" and "wet" conditions, the test effectively identified fully disturbed/low strength zones around the sleeve. Simultaneous sampling at test locations facilitated laboratory characterization of soil index properties. Subsequent direct shear testing under field-moisture conditions and 48 hour submerged conditions confirmed that high-plasticity soils exhibited similar strength loss. Conversely, lower-plasticity soils demonstrated limited magnitude of shear strength reductions.

The correlation proposed by Robertson (2009) was employed to estimate undisturbed strength (s_u) and disturbed strength ($s_{u(d)}$) for the revised CPT parameters, and these correlations were compared to laboratory direct shear strength test data. Marked decreases in peak shear strength similar to FSSS were observed at most sites. These outcomes well aligned with Skempton's (1970) concept of fully softened shear strength, validating the measurement scheme developed in this study. Overall, the revised CPT procedure offers a reliable, efficient in-situ method to evaluate fully softened shear strength, facilitating improved assessments of slope stability and soil behavior.

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Abstract

This study evaluated the “Fully Softened Shear Strength (FSSS)” of soils in Nebraska based on a revised Cone Penetration Test (CPT). FSSS is a unique strength condition that soils show substantially reduced strength during rainy seasons, eventually causing failure of slopes and other structures.

The revised CPT was adapted to inject water at targeted depths, inducing controlled moisture infiltration and capturing real-time soil softening over designated intervals of 30, 60, and 90 minutes. Through tip resistance (q_c) and sleeve friction (f_s) measurements under “in-situ” and “wet” conditions, the test effectively identified fully disturbed/low strength zones around the sleeve. Simultaneous sampling at test locations facilitated laboratory characterization of soil index properties. Subsequent direct shear testing under field-moisture conditions and 48 hour submerged conditions confirmed that high-plasticity soils exhibited pronounced strength loss. Conversely, lower-plasticity soils demonstrated limited magnitude of shear strength reductions.

The correlation proposed by Robertson (2009) was employed to estimate undisturbed strength (s_u) and disturbed strength ($s_{u(d)}$) for the revised CPT parameters, and these correlations were compared to laboratory direct shear strength test data. Marked decreases in peak shear strength were observed at certain sites. These outcomes well aligned with Skempton’s (1970) concept of fully softened shear strength. Overall, the revised CPT procedure offers a reliable, efficient in-situ method to evaluate fully softened shear strength, facilitating improved assessments of slope stability and soil behavior.

Chapter 1 Introduction

1.1 Background and Significance of Work

Nebraska has experienced gradual movement and failure in several roadside slopes, as identified in different research studies conducted by the Nebraska Department of Transportation (NDOT) and illustrated in Figure 1.1 (Bitar, 2020; Song et al., 2019, 2021). Back-calculated soil strengths for these failed slopes were significantly lower than those measured during the design phase (Song et al., 2019, 2021). Notably, the strength of soils at the failed slopes was reduced by 22–90% of their initial values under conditions of soil saturation and reduced effective overburden pressure, as detailed in Table 1.1.

Table 1.1 Strength Reduction Ratio of Failed Slopes

Slope Name	**% Strength Reduction**
UNL East Campus	55%
Bristow	68%
Santee Spur	90%
US-75	40%
Verdigre East	24%
Highway 84	22%

Figure 1.1 Failed slope at US-75

Further investigations revealed that this behavior was linked to the region's unique geological and geotechnical history. During the Cretaceous Period (145 to 60 million years ago), the area was submerged under seawater, leading to the formation of fine marine deposits, as shown in Figure 1.2(a). These deposits were later overlain by glacial tills during the Last Glacial Maximum, approximately 19–20 million years ago, as depicted in Figure 1.2(b). Both soil types exhibit extremely slippery and low-strength characteristics when saturated, contributing to the gradual failure of slopes. This condition is referred to as wet-drained-fully softened shear strength (Skempton, 1977, 1985; Stark et al., 2005; Terzaghi et al., 1996), reflecting the state of wet soils that have had sufficient time to drain water and undergo significant deformation, resulting in strength reduction. Similar strength conditions have been identified as a cause of numerous slope failures in Northern European countries, as (Skempton et al., 1969) reported.

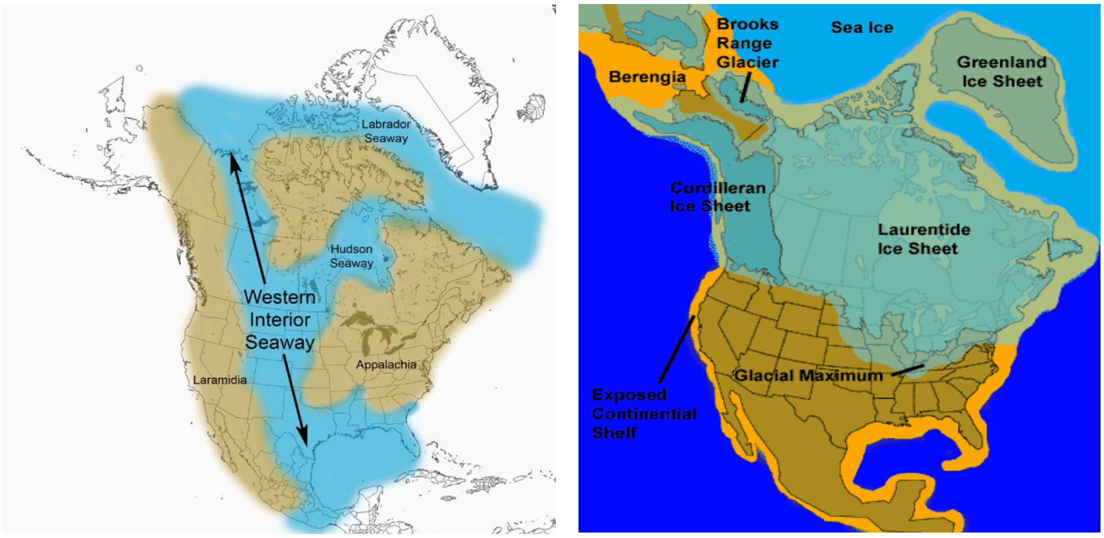

Figure 1.2 (a) Map of North America During Cretaceous Era (cretaceousatlas.org, 2025) (b) Map of North America During Last Glacial Period (*Digital Atlas, 2025*)

Methods for evaluating the shear strength of wet-drained-fully softened soil through laboratory testing have been extensively studied, such as Direct shear test (Castellanos, 2014; Skempton, 1977), ring shear test ((Stark & Eid, 1993), and triaxial test (Wright et al., 2007). However, these methods often require specialized laboratory equipment, such as a ring shear device, and involve lengthy testing durations.

On the other hand, research has investigated field testing techniques, such as the T-bar penetrometer, to measure the strength of disturbed (fully softened) soil (J. DeJong et al., 2010, 2011; J. T. DeJong et al., 2011; Lunne et al., 2005; Randolph et al., 1998; Stewart and Randolph, 1994). These methods have produced successful results for soils located below the groundwater table. Although the T-bar method is effective for testing below the water table, it cannot simulate artificial "wet" conditions for soils situated above the groundwater table.

This study proposes a Cone Penetration Test CPT-based field testing technique to create artificial "wet" conditions and directly utilize it in the field to evaluate the wet-drained-fully softened shear strength of soils, as follows:

- The water injection feature will be added to the existing CPT equipment to artificially create the wet condition for soils above the groundwater table.
- The friction resistance is directly related to the disturbed strength of soils with shear strain higher than 100% (Lunne et al., 2002a).

The piezocone penetration test (PCPT) is recognized as one of the most versatile and precise in-situ testing methods (Lunne et al., 2002a). This technique measures tip resistance (q_c), sleeve friction (f_s), and pore water pressure (u_1 or u_2) by inserting an instrumented probe into the ground and recording these parameters. The data obtained from PCPT are essential for evaluating the strength, deformation, and hydraulic properties of soils. Notably, the sleeve friction component of the test induces extremely high shear strain, often reaching several hundred percent (Song et al., 2019; Song et al., 2019, 2022; Song and Voyiadjis, 2005; Voyiadjis and Song, 2003).

Due to this significant shear strain, the soil surrounding the sleeve friction area becomes highly disturbed, approaching a fully softened state. Based on this principle, Powell and Lunne (2005) demonstrated that the undrained shear strength of disturbed cohesive soils could be effectively estimated using CPT. Additionally, Quyang and Mayne (2018) derived the effective stress-based shear strength of saturated soils from PCPT by utilizing tip resistance (q_c), pore water pressure at the cone shoulder (u_2), and a limit plasticity solution. By integrating the concept of a fully softened condition near the cone sleeve with the ability to determine strength parameters, it becomes feasible to assess the disturbed (nearly fully softened) shear strength of saturated soils using CPT. Furthermore, this technique could potentially be extended to partially wet soils if additional water is introduced in-situ to increase their moisture content to the required level.

Therefore, this research aims to directly obtain the wet-drained-fully softened shear strength of soils by measuring the friction resistance of the CPT, significantly advancing the engineering of obtaining the accurate critical strength of soils for transportation geo-infrastructures such as slopes and subgrades.

1.2 Problem Statement

The variation between the wet-drained-fully softened shear strength and the typical shear strength measured during the dry season is a critical factor influencing the stability and performance of geotechnical structures, particularly in regions with pronounced seasonal fluctuations in ground moisture content. In states such as Nebraska and other Midwestern areas, soil moisture content varies significantly between wet and dry seasons, leading to substantial changes in soil strength. This seasonal variability poses challenges for designing foundations, slopes, and embankments that must remain stable under both extreme dry and wet conditions. Conversely, in states such as Louisiana where soils predominantly exist in a wet state, conventional testing methods often capture the wet-drained-fully softened strength, providing a more accurate representation of soil behavior. This contrast underscores the pressing need for a direct, field-applicable method to measure the critical shear strength of soils in Nebraska and other regions with similar geotechnical conditions.

One approach to address this technical challenge is conducting field sampling or testing during wet seasons; however, this is often prohibited due to safety concerns. This study proposes an innovative application of the Cone Penetration Test (CPT) to recreate wet-drained-fully softened shear strength conditions in the field, enabling convenient and accurate measurement of this critical parameter.

Several studies, summarized in Table 1.2, have proposed methods to determine the wet-drained-fully softened shear strength of soils. However, as previously noted, these techniques are indirect and not fully developed for direct field application. Moreover, they often fail to provide site-specific and depth-specific measurements of the wet-drained-fully softened shear strength.

Table 1.2 Methods to Obtain Wet-Drained-Fully Softened Strength of Soils

Technique	**Reference**
Ring Shear Test	(T. Stark & Eid, 1993)
Triaxial Test	(Wright et al., 2007)
Repeated Direct Shear	(Skempton et al., 1969)
Artificial Intelligence	(Kaya, 2009)

1.3 Objectives of the Study

This study aims to develop a method to accurately and conveniently determine the wet-drained-fully softened shear strength of soils through the following steps:

1) Develop a systematic wetting technique for field soils based on the revised CPT.
    a. The study investigates the relationship between water injection pressure, injection duration, and the degree of saturation for typical soils such as shales, glacial tills, and Loess.
    b. Expected results are that the data shows the correlations between the soil type, injection pressure, and injection duration.

2) Develop a CPT testing mechanism to obtain "wet-drained-fully softened strength".
    a. Revised CPT technique: Use the revised CPT system to wet the soils and test soils.
    b. Expected output is the detailed testing procedure of revised CPT.
3) Verify the new testing method by comparing field revised CPT test results to other reference strengths.
    a. Comparison of revised CPT test results to back-calculated shear strength parameters for failed slopes, repeated direct shear test results (Skempton, 1977), and other published results.
    b. Output is a comparison chart of obtained "wet-drained-fully softened strength" with other testing techniques verifying the accuracy.
4) Develop a test data reduction procedure to obtain revised CPT "wet-drained-fully softened strength".
    a. Documentation of the test procedure for revised CPT-based technique for evaluating wet-drained-fully softened strength.

1.4 Work Procedure/Tasks

The principal investigator was Dr. Chung Song, and the co-principal investigators were Dr. Seunghgee Kim and Dr. Jongwan Eun at the University of Nebraska-Lincoln. The research assistants were Bashar Al-Nimri, Basil Abulshar and Dhurba Pandey. The laboratory, field tests, as well as numerical simulations, were primarily conducted at UNL's Geotechnical Engineering Lab and the designated field sites for this study. The project team has ample experience in conducting CPT in the state. The testing sites were either on the UNL campus or in locations that the PI and Co-PIs were familiar with from previous research. The project team coordinated closely with the TAC members during the research work and held several meetings throughout the duration of the project until its completion.

Chapter 2 Literature Review

A literature review was conducted on fully softened shear strength and what field and laboratory techniques are used to evaluate it.

2.1 Fully Softened Shear Strength

Section 2.1 provides a literature review on fully softened shear strength, starting with significant early cases of progressive slope failures and the initial hypothesis regarding fissure development. The section then explores observed mechanisms of strength reduction and the justifications proposed in the literature. It also examines the behavior of over consolidated and normally consolidated clays, concluding with the integration of fully softened shear strength into the design process.

2.1.1 Progressive Slope Failures and Fissures Development Hypothesis

The earliest known study documenting a long-term slope failure in stiff-fissured clay was conducted by Charles Gregory and published in 1844 in the Minutes of the Proceedings (Gregory, 1844). In this study, Gregory described a landslide that occurred during excavation of the London and Croydon railway. Initially, the slope remained stable; however, it was later inferred that an external event disrupted this equilibrium. The excavation profile consisted of a yellow clay layer, characterized by numerous "breaks", lying above a stiff, homogeneous, and impermeable blue clay. The term "breaks", as used by Gregory, corresponds to what is now commonly referred to as fissures in geotechnical engineering. The yellow and blue clay formations described in his study are now recognized as Brown and Blue London Clay, respectively (Gregory, 1844).

To explain the mechanism of failure, Gregory proposed that water infiltrates the fissures in the clay until saturation is reached, causing swelling within the soil mass. During the summer, the clay dries and shrinks; however, it does not return to its original volume, resulting in permanent deformation and the formation of additional cracks. These newly formed fissures allow further water infiltration, progressively increasing probability of slope failure over time. Gregory also noted that while the clay is highly stable when dry, it transforms into a semi-fluid state when wet, making it incapable of supporting itself.

As research on slope failures in stiff-fissured clays continued, attention turned to broader classifications, leading to Terzaghi’s findings. Terzaghi (1936) classified clays into three categories: soft intact, stiff intact, and stiff-fissured clays. Terzaghi examined five cases of initial slope failures in stiff-fissured clays and noted that the mobilized shear strength in the field was lower than the strength values obtained from laboratory tests. While it was not explicitly stated whether the reference was to drained or undrained shear strength, Terzaghi referred to undrained conditions. The case studies lacked specific details about the soil’s index properties or sliding depth (Terzaghi, 1936).

To explain the observed difference in shear strength, Terzaghi hypothesized that in their natural state, stiff-fissured clays experience high lateral pressures that keep fissures closed. After excavation, stress relief causes lateral expansion, leading to the opening of fissures. During rainfall, water infiltrates these fissures, causing the clay to swell under zero pressure, which weakens the material and creates additional cracks. As displacement occurs along these weakened zones, the trapped water prevents drainage, reducing shear strength to levels comparable to zero-pressure laboratory conditions. Furthermore, Terzaghi noted that in dry conditions, these fissures can remain open to a depth determined by the clay's compressive shear strength ratio to its moist unit weight.

In parallel with Terzaghi’s classification and hypotheses, other researchers sought more detailed investigations of joint and fissure characteristics in clays. Skempton et al. (1969)

investigated the characteristics of joints and fissures in London Clay through field studies at Wraysbury in Buckinghamshire and near Edgware in Middlesex. The study focuses on their orientation, mechanical behavior, and influence on the clay's strength and stability. The findings reveal that joints in London Clay are predominantly vertical, with a moderate preferred orientation, while fissures are smaller, planar, or conchoidal fractures concentrated near the upper clay layers. These discontinuities reduce the overall strength of the clay mass, and strength specifically along joints and fissures exhibiting a peak value followed by a drop to residual strength. Stress relief and weathering are critical factors in fissures' formation and weakening effects. The study underscores the importance of considering these discontinuities in geotechnical design, as they significantly affect the stability and mechanical behavior of London Clay.

2.1.2 Suggested Failure Mechanisms and Strength Reduction Justifications

Skempton (1970) examined the strength characteristics of over-consolidated clays in the context of first-time slope failures. The fully softened shear strength was defined as the drained peak shear strength of clay in a normally consolidated state, representing the strength after the clay's structure has been destroyed but before the development of a continuous shear surface as shown in Figure 2.1.

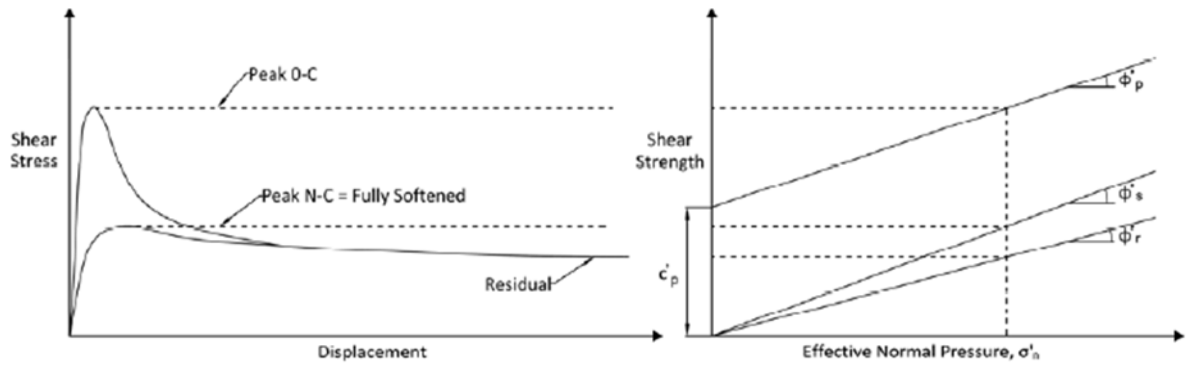

Figure 2.1 Shear Characteristics of Clays (Skempton, 1970)

Skempton (1970) argued that the fully softened strength is a more realistic and appropriate limit than the residual strength in such conditions. Researchers described the post-peak behavior of over-consolidated clays in two stages: first, dilatancy and fissure opening lead to increased water content and a drop in strength to the fully softened value, forming a softened shear zone; second, principal shears develop, eventually linking to form a continuous shear surface at residual strength. The findings emphasize the role of fissures as stress concentrators and their contribution to clay softening, highlighting that the fully softened strength should be used in designing first-time slides, especially in London Clay.

Subsequent research built upon these observations led to a more nuanced understanding of delayed failures in similarly over-consolidated clays. Skempton (1977) addressed delayed failures in cuttings within the stiff and fissured brown London Clay. Skempton identified the slow equilibration of pore pressures, typically taking 40 to 50 years, as the primary cause of these delayed failures. Back analysis of slope failures demonstrates that the clay's strength at failure corresponds closely to the fully softened or fissure strength, which is greater than the residual strength but lower than the peak strength, even when measured on large samples. The study highlights shear strength parameters for first-time slides in brown London Clay as

c'=1 kN/m^2 and ϕ'=20, and references historical observations that slopes steeper than 3:1 were generally unstable. These findings emphasize the importance of considering fully softened strength in long-term slope stability analysis and the role of pore pressure dynamics in influencing failure.

In parallel, Skempton expanded his investigation and examined the behavior of clays transitioning from fully softened to residual strength conditions. In the 1985 paper, "Residual Strength of Clays in Landslides, Folded Strata, and the Laboratory", Skempton (1985) comprehensively analyzed the residual strength in clays across various geological settings. The post-peak drop in strength was described as a two-stage process: first, a decrease to the fully softened or critical state strength due to increased water content, and second, a further drop to residual strength caused by the reorientation of platy clay minerals parallel to the shear direction. The study highlighted that residual strength is predominantly governed by the clay fraction, with clays containing less than 25% clay fraction behaving as silts or sands, while those with around 50% clay fraction exhibit residual strength dominated by sliding friction of clay minerals. Residual strength angles for kaolinite, illite, and montmorillonite are approximately 15°, 10°, and 5°, respectively. Laboratory tests and back analysis of reactivated landslides yield comparable residual strength values, although ring shear tests tend to report slightly lower values. The study underscored the significance of residual strength in engineering design, particularly for landslide analysis and stability in folded strata, while emphasizing the critical role of clay fraction, water content, and particle reorientation.

Alongside Skempton's extensive work, other researchers also explored over consolidated clays, focusing on how historical overburden pressures influenced slope failures. Chandler (1974) analyzed 12 slope failures in cuts through over consolidated Upper Lias Clay, a significantly over consolidated type of clay with a history of overburden pressures exceeding 3,000 feet. The clay was brecciated near the surface and exhibited variable shear strength due to weathering. By employing back-analyses based on the Morgenstern and Price's (1965) limit equilibrium method, Chandler determined a lower bound for the drained shear strength with $c' = 0$ and $\phi' = 23° \pm 1.5°$. The stabilization of pore pressure in the clay varied, taking up to 60 years in unbrecciated layers (Chandler, 1974). However, Chandler and Skempton (1974) indicated that the estimate of $c' = 0$ is conservative and argued that small effective stress cohesion values are significant for initial sliding events due to progressive failure mechanisms.

Further extending these findings, later studies investigated how time and fissure permeability influenced pore pressure stabilization in saturated clays. Chandler, (1984a, b) concluded that the time required for pore pressure stabilization in saturated clays varies depending on factors such as drainage path length and fissure permeability. High-plasticity clays exhibit greater susceptibility to progressive failure due to their brittle behavior and significant strength losses after the peak. Larger triaxial test specimens showed lower shear strengths than smaller ones, but both overestimated field strengths, attributed to mechanisms such as softening, creep, and progressive failure.

Concurrently, the impact of weathering on clay properties gained attention, reinforcing the importance of both geological and environmental factors in slope stability. Chandler and Apted (1988) examined weathering effects on London Clay, finding that weathering reduces the pre-consolidation pressure and effective stress cohesion. Despite minimal changes in mineralogy or index properties, weathering led to distinct stress paths. Triaxial tests confirmed lower cohesion in more weathered samples, consistent with field observations.

2.1.3 Over consolidated and Normally Consolidated Clays Behavior

Over consolidated clays are more susceptible to a decrease in shear strength in-situ compared to normally consolidated clays. Climatic conditions can further reduce the shear strength of both over consolidated and compacted clays. These weather patterns, characterized by cycles of wetting and drying or freezing and thawing, have been shown to diminish the shear strength of these types of clay, with the lowest limit of this decreased shear strength being the fully softened shear strength (BOTTS, 1986; Graham and Au, 1985; Kayyal and Wright, 1991; Wright, 2005; Wright et al., 2007).

In Eastern Nebraska, the cycles of wetting and drying occur due to seasonal variations between dry and wet periods, exacerbated by the region's high to extremely high rainfall. This precipitation can act as a contributing factor to slope failures. Additionally, Nebraska experiences freeze and thaw cycles. This weather pattern allows water to infiltrate rocks through cracks and joints. When the water freezes, it creates significant expansion pressure, which can lead to the formation of new cracks or the widening of existing ones. The repeated freeze and thaw cycles are one of the potential causes of slope sliding (Song et al., 2019).

Therefore, given the influence of wetting-drying and freeze-thaw cycles on slope stability, it is critical to distinguish and understand the differing mechanical responses of over consolidated and normally consolidated clays. Stark and Duncan (1991) observed that the strength of stiff fissured clays decreases from peak to fully softened conditions over time. However, they also noted that intense rainfall can suddenly and significantly reduce strength. This phenomenon may be significant for swelling clays in Nebraska, which readily absorb water and lose strength when exposed to prolonged precipitation (Song et al., 2019).

Building on the idea that clay properties depend on both plasticity and consolidation state, Mesri and Abdel-Ghaffar (1993) further explored how soil plasticity affects the mobilized shear strength. Researchers conducted a comparative study on the shear strength of soft and stiff clays by analyzing 35 different slopes composed of both soft and stiff soil types. Their research involved back-analysis to determine the shear strength parameters at the point of initial failure. A reduction factor was calculated through back-analysis to quantify the ratio between mobilized shear strength and peak shear strength. The findings indicated that clays with low plasticity tend to exhibit mobilized shear resistance close to their peak strength. However, they observed that the shear strength could be significantly reduced in over consolidated clays with a plasticity index of 20% or higher.

Extending these findings, Wright (2005) investigated compacted clay embankments. Wright (2005) summarized research conducted by the Center for Transportation Research for the Texas Department of Transportation regarding the stability of compacted clay embankments in Texas. His findings indicated that the results from consolidated undrained (CU) and consolidated drained (CD) triaxial tests produced nearly identical effective shear strength failure envelopes. Further investigation into embankment failures involving compacted high-plasticity clays was presented by Wright et al. (2007). Their study focused on characterizing the shear strength of Eagle Ford Shale that shares a similar geological history of shales in Nebraska, a soil type frequently associated with such failures (Wright, 2005; Wright et al., 2007).

To understand how different sample preparation techniques affect shear strength, Wright (2005) carried out an extensive laboratory testing program. The laboratory testing program examined specimens prepared using three methods: (1) compacted, (2) compacted and subjected to wetting and drying cycles, and (3) normally consolidated from a slurry. CU triaxial tests were performed to determine the effective stress shear strength parameters, revealing a curved failure envelope across all sample preparation techniques.

The study found that even after 20 cycles of wetting and drying, the shear strength envelope of Eagle Ford Shale did not reach a normally consolidated state. Results indicated that normally consolidated specimens had a higher void ratio than those subjected to wetting and drying cycles, which was identified as a key factor influencing the measured shear strength. Through back-analysis of a failed slope in Eagle Ford Shale, Wright recommended using fully softened shear strength along with a pore pressure ratio ranging from 0.43 to 0.53 to better reflect realistic field conditions.

2.1.4 Incorporating Fully Softened Shear Strength in the Design Process

Chandler (1988) suggested using fully softened shear strength as an empirical design parameter for slopes, especially in high-plasticity clays. Residual shear strength, often mobilized in slopes with prior movement or tectonic activity, was found to be higher in the field than in laboratory tests, due to variations in stress distribution and movement dynamics during failure.

Further refining these insights, Stark et al. (2005) developed a framework that highlighted the importance of both residual and fully softened shear strength in landslide analysis. Researchers provided a comprehensive framework for selecting drained shear strength parameters essential for landslide analysis. Their study highlighted the significance of stress-dependent failure envelopes and updated empirical relationships for residual and fully softened shear strengths. Torsional ring shear tests conducted on 36 specimens—including clays, mudstones, claystones, and shales—demonstrate how soil composition and pre-existing shear surfaces influence shear strength behavior. The findings reveal that residual shear strength is governed by the frictional resistance of clay particles arranged in a face-to-face orientation and that any strength gain due to healing is lost after minor shear displacement. Additionally, the study emphasizes that the effective stress cohesion component should be set to zero when considering residual and fully softened strength conditions, reinforcing the importance of stress-dependent modeling in slope stability analysis.

Stark et al. (2005) also identified key limitations, primarily its focus on drained shear strength parameters, assuming sufficient hydraulic conductivity to dissipate pore-water pressures before instability occurs. Despite the abovementioned limitations, the research provides valuable empirical relationships that improve residual and fully softened strength predictions, particularly when incorporating liquid limit and clay-size fraction data. Practical applications include using updated strength parameters in slope stability analyses and considering pre-existing shear surfaces in slope design.

Building on these empirical relationships, Kaya (2009) used an artificial intelligence approach to estimate secant fully softened and residual friction angles. In 2009, Kaya utilized a back-propagation artificial neural network (ANN) model to estimate the secant fully softened and residual friction angles of soils. The back-propagation method improves the ANN's prediction ability by continuously learning and adjusting the model based on new data through a corrective feedback system (Tariq et al., 2024). ANNs are computational models that identify both linear and nonlinear relationships between input and output variables. For model development, Kaya used 75% of the dataset from Stark et al. (2005) for training, with the remaining 25% reserved for validation. A sensitivity analysis was performed to evaluate the effects of liquid limit, plastic limit, clay-sized fraction, activity, and effective normal stress on the secant fully softened friction angle. The results indicated that the plastic limit and activity had minimal impact on the fully softened secant friction angle (Kaya, 2009).

The validation of the ANN model, using an independent dataset, highlighted its potential as a predictive tool, achieving a coefficient of determination (R^2) of 0.88 in estimating the residual friction angle of soils, which is significantly high considering the typical variations of natural materials. The ANN model's predictions for the fully softened friction angle were compared with those obtained using the empirical curves from Stark et al. (2005) and the equation from Wright (2005). It was found that both the ANN model and the Stark et al. (2005) curves provided reasonable predictions of the measured fully softened friction angle, with an R^2 value of 0.80. In contrast, the Wright (2005) equation showed lower predictive accuracy with an R^2 of 0.45, leading Kaya to conclude that Wright (2005) empirical formula was not suitable for estimating the fully softened secant friction angle (Kaya, 2009).

2.2 Laboratory Tests for Evaluating Fully Softened Shear Strength

Section 2.2 presents a literature review on the laboratory tests utilized to evaluate the fully softened shear strength, including direct shear test, triaxial test, and ring shear test.

2.2.1 Direct Shear Test and Triaxial Test

Skempton's (1985) study focused on the residual strength of clays, particularly in the context of landslides, folded strata, and laboratory conditions. The laboratory component of the research involved direct shear and triaxial tests conducted on undisturbed and remolded clay samples. The samples were prepared to simulate field conditions, including pre-shearing to mimic the effects of prior failure. The tests were performed under controlled moisture content and strain rates to replicate the stress conditions that clays experience in nature. The goal was to understand how clays behave when subjected to significant deformation. Notably, the clay reached a residual shear strength state without further deformation as shear stress increased.

The results of the laboratory tests revealed that the residual strength of clays is much lower than the peak strength reached during initial shearing. Higher plasticity clays, such as CH, generally exhibited lower residual strengths compared to less plastic clays, such as CL and SC. The residual strength remained relatively constant after reaching a threshold, indicating that once a certain amount of deformation occurred, the material behaved as a residual shear strength material.

Extending these findings to real-world settings, recent studies have highlighted the role of local geology. According to Al-Nimri et al. (2025) and Song et al. (2019) Nebraska is covered by glacial tills, loess/sands, and shales with expansive and high plasticity clay minerals, the combination of these materials made Nebraska more vulnerable to landslide risk.

While these insights underscored the importance of residual strength, Skempton's study also acknowledged several constraints. One key limitation is the difference between laboratory and field conditions, as the tests could not fully replicate the scale, complexity, and variability of natural landslides and geological history. Laboratory samples were also subjected to rapid shearing rates, whereas landslides typically occur over a longer period, which can lead to discrepancies in strength measurements. Additionally, the disturbance of samples during retrieval and preparation could affect the accuracy of the results, particularly in terms of moisture content and structure.

Building upon such foundational insights, Wright, Zornberg, and Aguettant (2007) conducted further investigations using both triaxial and direct shear tests. Wright, Zornberg, and Aguettant (2007) conducted a laboratory study to evaluate the fully softened shear strength of high-plasticity clays using consolidated undrained (CU) triaxial tests and direct shear tests. Soil specimens were prepared using three different methods: compacted at optimum moisture content, compacted and subjected to wetting-drying cycles, and normally consolidated from a slurry to simulate long-term softened conditions. The results indicated that normally consolidated specimens exhibited the lowest shear strength, consistent with fully softened behavior. However, compacted specimens subjected to wetting and drying cycles did not fully reach the shear strength of normally consolidated soils, suggesting that repeated moisture fluctuations alone do not always lead to complete softening. Additionally, variations in void ratio played a significant role in determining the measured shear strength, with normally consolidated specimens displaying higher void ratios than compacted samples.

While the study provided valuable insights into the shear strength behavior of high-plasticity clays, several limitations were noted. The controlled laboratory environment could not fully replicate long-term field conditions, where aging, prolonged moisture infiltration, and stress redistribution may further affect soil strength. Additionally, the number of wetting and drying cycles was limited, making it unclear whether extended cycling would lead to further reductions in strength. Soil variability, including differences in mineralogy and organic content, could also influence the general applicability of the findings. Despite these limitations, the study's results offer practical guidance for assessing slope stability and foundation performance, emphasizing the need for further field validation to confirm the laboratory findings in real-world geotechnical applications.

2.2.2 Ring Shear Test

The study conducted by Stark and Eid in 1993 examined the residual strength of bentonitic tuff from the Portuguese Bend Landslide. The study utilized a modified Bromhead ring shear apparatus, which featured a newly designed specimen container. This innovative container facilitates over consolidation and the precutting of remolded specimens, thereby improving the simulation of real field conditions and significantly reducing the duration of tests. By addressing the limitations of traditional testing methods, the study enhanced the accuracy of residual strength measurements.

One of the key findings is that wall friction along the inner and outer circumferences of the confined specimen affects the measured drained residual strength. The new specimen container minimized wall friction effects, leading to more reliable results. Additionally, the precut specimens demonstrated a lower peak strength and required less horizontal displacement to reach a residual condition compared to intact specimens, making the testing process more efficient.

The study estimated the effective shear strength parameters for the nonhorizontal slide surface as c' = 0.0 kPa and ϕ' = 20°. Using the residual envelope obtained from intact specimens and the "flush" test procedure, a 1.13 factor of safety was determined for the coastal subslides. Furthermore, the modified container reduced vertical displacement to less than 0.75 mm, which allows for multistage testing and the efficient development of drained residual failure envelopes.

By incorporating over consolidated and precut remolded specimens, the new procedure shortened the time required to obtain a residual failure envelope to approximately five days. The findings confirm that the new method provided a better representation of field conditions, capturing the large post-peak decrease in drained strength often observed in clay shales, claystones, and mudstones. Most importantly, the residual strengths measured using this technique are in excellent agreement with field case histories.

However, the original Bromhead ring shear apparatus has several limitations. Excessive settlement and wall friction impact the measured residual strength, leading to potential inaccuracies. Additionally, the study notes that the difference in the residual failure envelopes measured using intact and precut specimens increased with soil plasticity and clay fraction, indicating a dependency on soil composition. Another drawback of the original specimen container was the time-consuming process required to obtain a drained residual failure envelope, as repeated soil additions and reconsolidation were necessary, taking 16 to 18 days.

2.3 Field Tests for Evaluating Fully Softened Shear Strength

Several researchers have investigated the use of field-testing techniques, particularly the T-bar penetrometer, to determine the disturbed (fully softened) strength of soils below the groundwater table. These studies have demonstrated the effectiveness of the T-bar penetrometer in profiling soft clay sediments and estimating undrained shear strength.

Stewart and Randolph (1994) were among the first to introduce the T-bar penetrometer as a tool for in-situ testing of soft clays. Their research demonstrated that the T-bar could provide continuous profiles of undrained shear strength, offering a reliable alternative to traditional methods. Building on this work, Randolph et al. (1998) refined the methodology, focusing on soil-penetrometer interaction and providing guidelines for interpreting T-bar test results to enhance the reliability of undrained shear strength estimations.

Extending these foundational studies, DeJong et al. (2010, 2011b) further advanced full-flow penetrometer applications by developing correlations for estimating undrained shear strength and soil sensitivity. Their 2010 study evaluated undrained shear strength using full-flow penetrometers across multiple soft clay sites, establishing correlations with field vane shear data. In 2011, they proposed recommended practices for full-flow penetrometer testing and analysis, providing standardized procedures for field investigations. Their 2012 research focused on evaluating remolded shear strength and soil sensitivity, contributing to a deeper understanding of soil behavior under different stress conditions.

Simultaneously, Lunne et al. (2005) applied T-bar testing methods to offshore environments, confirming the technique's broader utility. Lunne et al. (2005) analyzed test results from multiple subsea soft soil sites, confirming the reliability of the T-bar penetrometer in measuring the undrained shear strength of in-situ and remolded soft soils on the seabed. Their work provided valuable insights into the applicability of T-bar testing in offshore geotechnical investigations.

Despite its advantages, several limitations have been identified in using the T-bar penetrometer for geotechnical investigations. One major limitation is its dependence on soil type, as the test is most effective in soft clays but less reliable in stratified or mixed soils with varying sand, silt, or organic content (Randolph et al., 1998; Stewart and Randolph, 1994).

Additionally, empirical correlations used for interpreting T-bar results may not be universally applicable, requiring site-specific calibration for accurate strength estimation (J. DeJong et al., 2010, 2011a, 2011b). Another concern is strain rate sensitivity, as the penetration rate can significantly impact measured resistance, leading to potential variations in results for highly sensitive soils (Lunne et al., 2005).

Furthermore, while the T-bar penetrometer is valuable for estimating remolded shear strength, it may not fully replicate in-situ field softening processes. Laboratory-based reconsolidation and shearing can differ from natural long-term strength degradation, leading to discrepancies in engineering assessments (DeJong et al., 2011a)

Other challenges include depth limitations and equipment constraints, particularly in deep seabed investigations, where rod bending and difficulty in maintaining vertical alignment become significant concerns (Lunne et al., 2005). Additionally, T-bar results may not always align with other in-situ testing methods, such as the field vane shear test or cone penetration test (CPT), due to differences in stress conditions, drainage behavior, and test execution (Randolph et al., 1998; Stewart and Randolph, 1994).

Recently, The piezocone penetration test (CPT) has been gaining popularity worldwide as an efficient and cost-effective method for delineating soil stratigraphy and interpreting geotechnical parameters at various depths (Ouyang & Mayne, 2018). However, the conventional CPT does not control the saturation condition of the soil in a way that represents the wet condition which may trigger the softening phenomenon and result in a lower shear strength. The next Chapter proposes a new innovative CPT-based field-testing technique to create an artificial "wet" condition to obtain the wet-drained-fully softened shear strength in the field indirectly.

Chapter 3 Evaluation of Critical Shear Strength of Soils Based on Revised CPT

Chapter 3 provides comprehensive information about the selected testing locations, including soil type, failure shape, design strength, and back-calculated strength where applicable. Additionally, the chapter details the design and fabrication of the testing equipment, followed by the testing procedure and the sequence of the traditional geotechnical tests conducted.

Figure 3.1 Tested Sites in Nebraska

3.1 Testing Locations

In this study, eleven different sites were selected to conduct the revised CPT. The selection of these sites was based on previous research conducted by Song et al. (2019, 2021) on previously failed slopes, with the exception of the UNL East campus. The UNL East campus was not a failed slope but was included as a preliminary test site during the testing procedure development. The map in Figure 3.1 shows all the test locations. Detailed descriptions of the test locations are provided in Sections 3.1.1 to 3.1.11.

3.1.1 UNL East Campus

The UNL East Campus is a preliminary test site because it was easily accessible for the research team to develop the testing procedure for the revised CPT. Also, UNL East campus is known for its stiff clayey soil during the dry seasons and soft clayey soil during the wet seasons. The UNL East Campus test location is shown in Figure 3.2.

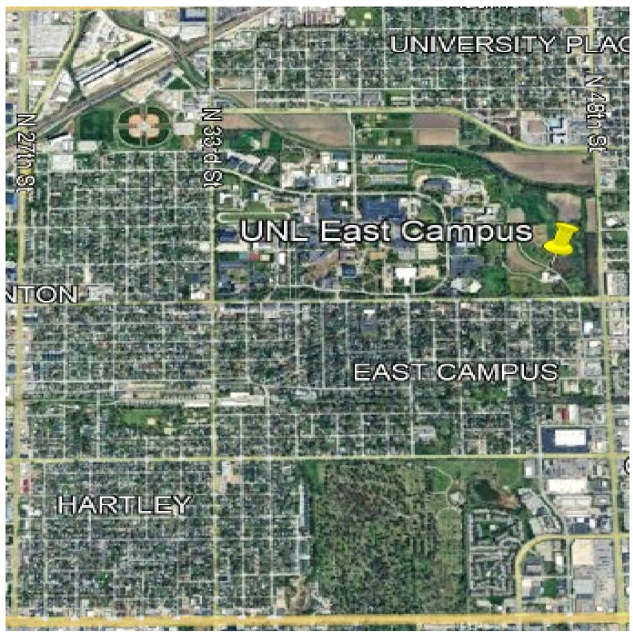

Figure 3.2 UNL East Campus Test Location

3.1.2 Lincoln I-180

The second testing slope is in northern Lincoln, NE at the intersection of Interstate-180 (I-180) and Superior Street. The slope, constructed in 1964 from a cut section, initially had a 3:1 gradient with an estimated height of 8.8 m and a width of 26.4 m, based on back-analyzed data from LiDAR and field surveys conducted in 2012 and 2018. The failure, which began around 1996–1997, indicates a long-term stability issue, taking 32–33 years to manifest (Bekele et al., 2021). The Lincoln I-180 test location is shown in Figure 3.3.

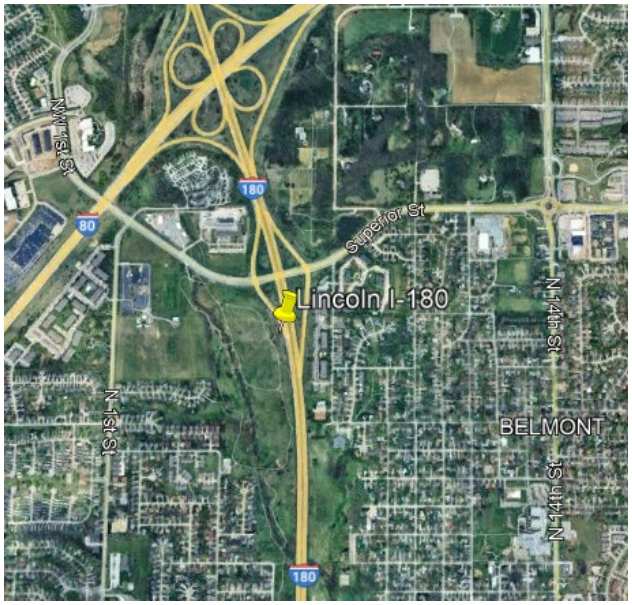

Figure 3.3 Lincoln I-180 Test Location

The failed zone spans approximately 40 m transversely and 22 m longitudinally, with a crown forming partway down the slope, originating from an initial tension crack. The stable zone above the crown remains largely intact, showing minimal deformation. Visual inspections and deformation patterns from LiDAR and field surveys confirm these observations. Continued slope movement led to the formation of a scarp and exposed soil, yet no visible distress was observed on the pavement adjacent to the slope toe (Bekele et al., 2021).

3.1.3 Lincoln Hwy-2

The next test location is approximately seven miles east of Hwy-77 along Hwy-2 near Lincoln, NE as shown in Figure 3.4. The location was recommended by NDOT engineers due to previous historic failure. Based on information provided by NDOT, the soils are clay and sand layers. Further information regarding soil profile can be found in Appendix A.

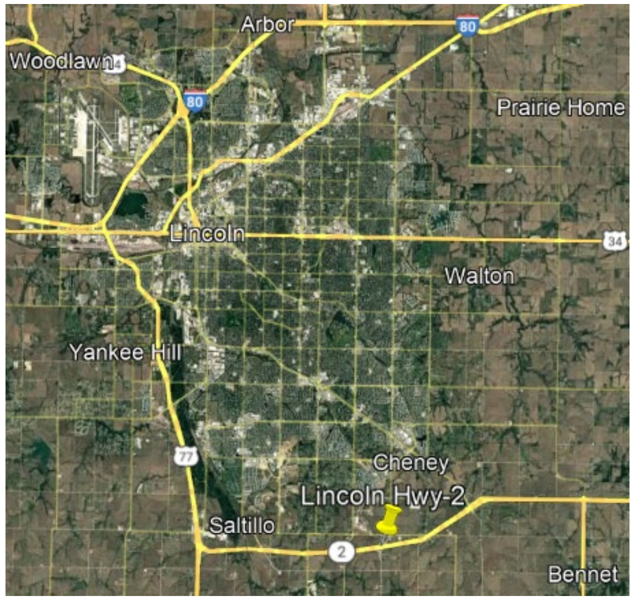

Figure 3.4 Lincoln Hwy-2 Test Location

3.1.4 Verdigre South, Highway 14

The slope failure on Highway 14 occurred south of Verdigre, NE approximately 1.5 to 2 miles from the Verdigre Creek River. The slide was approximately 350 feet in length. The slope was designed following Nebraska's road construction standard with a vertical-to-horizontal ratio of 1:3. The slope location is shown in Figure 3.5.

According to Song, et al. (2019), the slope resulted in a 1.368 factor of safety. The slope comprises of twelve shale layers with varying strength characteristics. The presence of multiple weak layers with slight differences in shear strength can lead to progressive failure. Given the configuration of these adjacent weak layers, the potential slip surface cuts through all layers. To achieve a factor of safety equal to 1.0, the back-analysis indicated that the strength parameters needed to be reduced by approximately 25% from their original values.

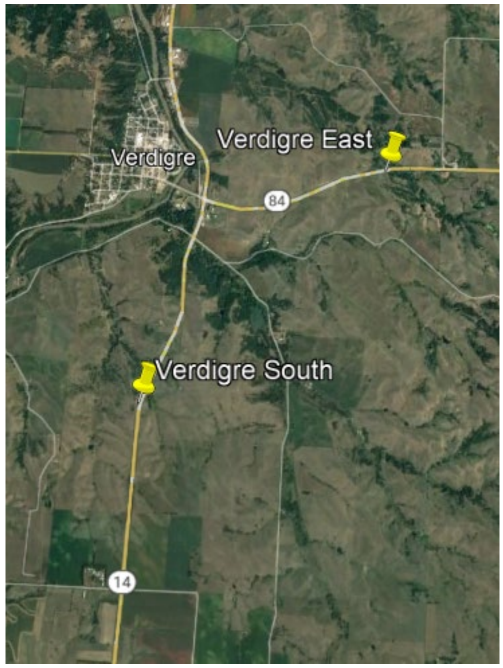

Figure 3.5 Verdigre East and South Test Locations

3.1.5 Verdigre East, Highway 84

This slope is located along Highway 84, approximately 1.5 miles east of the Highway 14 and Highway 84 intersection near Verdigre, NE. The slides on Highway 84 were approximately 100 feet and 250 feet long, respectively (Lindemann, 2010). The slope was designed following Nebraska's road construction standard with a vertical-to-horizontal ratio of 1:3.

The slope failure occurred due to the factor of safety being below unity, indicating lower actual strength parameters than those derived from bore logs. The slope consisted of fill and reworked shale layers, with reduced strength observed in saturated shale below the groundwater table. Two bore logs, taken at different distances from the road centerline, highlighted this reduction. Back-calculated analysis showed that increasing the strength parameters by approximately 22% was necessary to achieve a factor of safety of 1.0, informing the retrofitting design (C. R. Song et al., 2019).

This site was used to show the slow downward movement of the slope, and NDOT decided to remove and replace the materials from the slope with compacted geogrid reinforced soil (C. R. Song et al., 2021). Approximate locations of both slopes tested in Verdigre are shown in Figure 3.5.

3.1.6 Bristow

The Bristow, NE slope is located along Highway 12 and consists of a mix of fill materials, including fill shale, which overlays undisturbed shale layers to a total depth of 25 feet (Song et al., 2019). Natural shale is found between 16 and 20 feet below the surface. This area experiences relatively high precipitation and features a combination of shale and loess layers. The Bristow slope location is shown in Figure 3.6.

The initial factor of safety calculated using the design parameters is 3.573, indicating the design is highly conservative. The slip surface is located within the fill materials, beginning near the boundary of the asphalt and the fill at the top, and extending down to the toe of the slope. The back-calculated slip surface, which has a safety factor of 1.0, closely resembles the initial slip surface. A comparison of the strength parameters shows that they have been reduced by an average of 68% from their original values, due to the weakening of the shale layers during wet seasons.

Figure 3.6 Bristow Test Location

3.1.7 Santee Spur

The Santee Spur slope failure occurred in northeastern Nebraska near Santee, NE with the Missouri River, approximately 1.2 miles away, as shown in Figure 3.7. About 125 feet long, the slide caused a longitudinal crack along the roadway's centerline. The soil profile included asphalt milling and multiple layers of both weathered and unweathered shales with varying strengths. The thickness of the weathered shale layers was nearly equivalent to the total slope height (Song et al., 2019). The slope was designed with a 1:3 angle, adhering to Nebraska road construction standards.

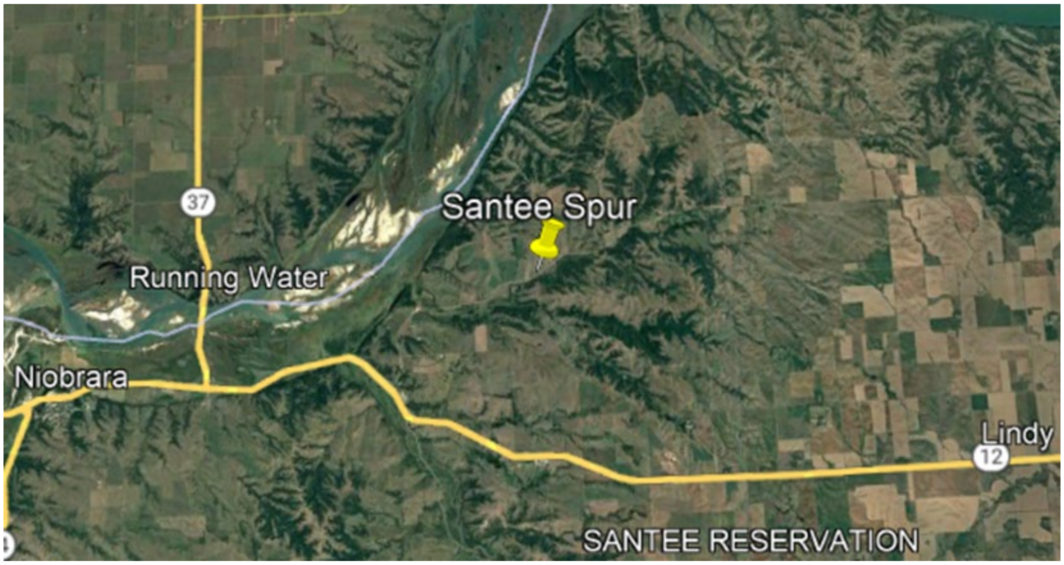

Figure 3.7 Santee Spur Test Location

Initially, the calculated factor of safety for the slope was 9.267, indicating a highly conservative design. The failure, however, occurred along a slip surface passing through the weathered shale layers. Back analysis revealed that the slip surface for the failure aligned closely with the initial slip surface. The analysis also determined that the strength of the weathered shales had to be reduced by approximately 85% of their original values to achieve a factor of safety of 1.0 (Song et al., 2019).

3.1.8 US-75

The next slope lies along Highway 75, approximately 1.7 miles southwest of Plattsmouth, NE, 2.5 miles west of the Missouri River, and 1.2 miles south of the Platte River, as shown in Figure 3.8. The failure occurred in the southeastern section of the flyover image. The slope consists of a thick layer of fill material, predominantly Peoria, resting atop a layer of slightly clayey silt with low strength. Beneath the silt is a 20-foot layer of fine silty sand that becomes prone to failure during rainy seasons (Song et al., 2019).

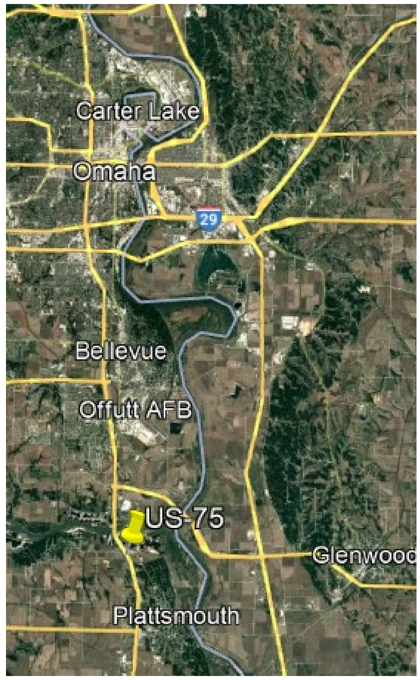

Figure 3.8 US-75 Test Location

Based on the design soil parameters, a factor of safety of 1.603 was calculated, indicating that the slope is stable. The potential slip surface with the lowest factor of safety began near the top edge of the slope, passed through the silt layer, and terminated near the toe of the slope. For the back analysis, the internal friction angle of the soil at failure was assumed to be zero, reflecting the cohesive nature of the soils. Using a trial-and-error approach, the shear strength of the soil was adjusted to achieve a factor of safety of 1.0. The slip surface location remained consistent with the original analysis but corresponded to a factor of safety of unity (Song et al., 2019).

3.1.9 Waverly

The Waverly, NE slope is located approximately 10 miles east of Lincoln along Cornhusker Highway 6, as shown in Figure 3.9. Based on information provided from NDOT, the soil layer is on top of silt layers. Further information regarding soil profile can be found in Appendix A.

Figure 3.9 Waverly Slope Test Location

3.1.10 Greenwood

The Greenwood, NE slope is located approximately 20 miles east of Lincoln along Highway I-80 on the ramp exiting I-80 to Hwy-63, as shown in Figure 3.10. Based on information provided by NDOT, the top layer is a fill, then a layer of silt, then clay. Further information regarding soil profile can be found in Appendix A.

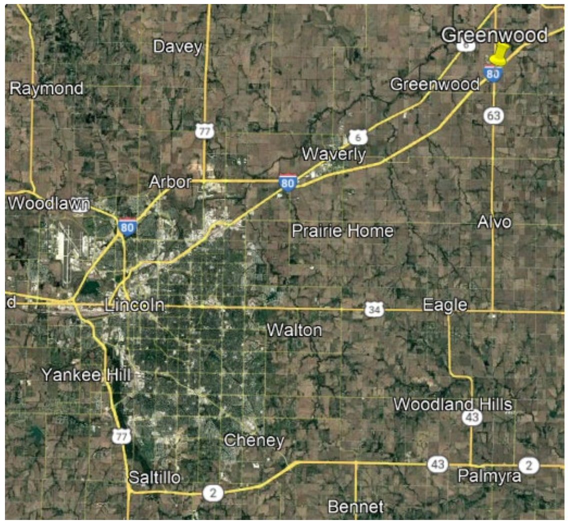

Figure 3.10 Greenwood Test Location

3.1.11 Norfolk, Hwy-275

The final location is approximately 10 miles east of Norfolk, NE, along Highway 275 and 2.5 miles from the T intersection of Hwy 275 with Hwy 57, as shown in Figure 3.11. Based on information provided by NDOT, the top layer is 4 ft of Lean clay, (dark gray; black, soft, and moist), then a layer of Peoria Loess described as Light brown, brown lean clay with some silty layers, soft, and moist (peorian), which is from 4 to 26 ft. Further information regarding soil profile can be found in Appendix A.

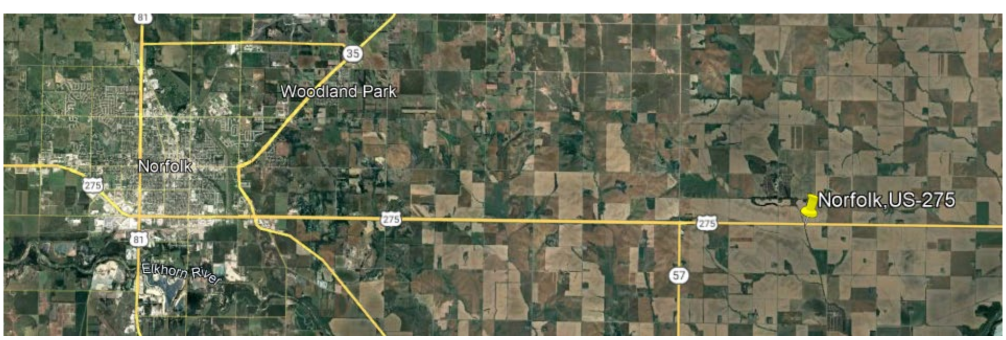

Figure 3.11 Norfolk Hwy-275 Test Location

3.2 Piezocone Penetration Test (CPT)-Field Testing Procedure

3.2.1 General

Section 3.2 provides the detailed design, fabrication, and testing procedure utilized in this research. A Nova CPT system mounted on a Geoprobe 7822DT was utilized to conduct in-situ measurements of tip resistance, friction resistance, and pore pressure. The data was taken in 0.8-inch (2-cm) depth increments during penetration and were conducted for both in-situ and wet conditions to evaluate soil behavior under varying moisture conditions.

3.2.2 Design and fabrication of revised CPT component

The research employs a special design of a wetting cone penetrometer to inject water into the subsurface in the field. The cone has a geometry similar to that of traditional CPT probes. Figure 3.12 illustrates the wetting cone components.

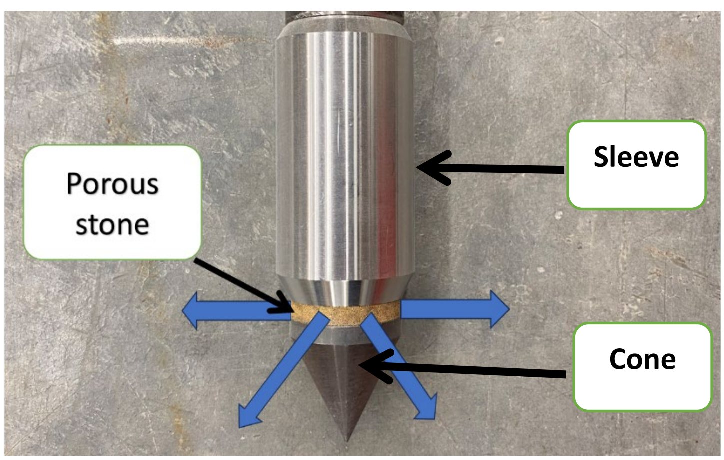

Figure 3.12 Wetting Cone

The injection system utilizes the Geoprobe DI HPT flow module (K6300 series) to inject water from a 15-gallon plastic drum reservoir to the cone through a 1/8" inner diameter tube connected to the specialized wetting cone. The flow module and plastic drum are shown in Figure 3.13.

Figure 3.13 (a) Geoprobe DI HPT Flow Module K6300 Series). (b) ULINE 15-Gallon Plastic Drum

A schematic diagram of connections for the power supply and water source utilized in the revised CPT method is shown in Figure 3.14. This schematic illustrates the water-injection system used in the revised cone penetration testing (CPT) procedure. Water from an external source is drawn into a pump, which pressurizes and pumps the water through a supply line. The injected water exits from the porous stone above the cone tip, enabling targeted wetting or softening of the surrounding soil at the desired depths.

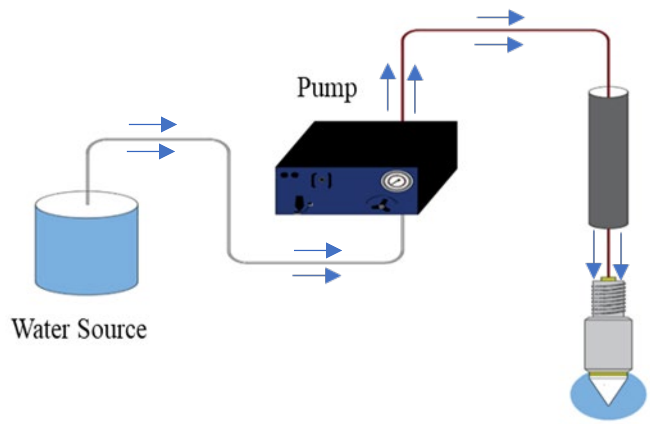

Figure 3.14 Schematic of Revised CPT System

3.2.3 Revised CPT Test Procedure

This study proposed the following procedure to wet the soils and obtain the wet-drained-fully softened shear strength:

1. Use standard Cone Penetration Test (CPT) equipment and conduct a conventional CPT at the predetermined location. Record the "in-situ" measurements for tip resistance, sleeve friction, and pore water pressure at various depths of the target soils.
2. Withdraw the standard CPT cone.
3. Move the CPT rig approximately 1 foot (0.3 m) away from the initial borehole.
4. Attach the specialized wetting cone to the rig. Connect the cone to the Geoprobe flow module (K6300 Series HPT Controller) using a 1/8-inch tube.
5. Push the wetting cone down to the desired depth.
6. Open the flow valve on the pump and initiate water injection. Maintain a flow pressure of 50 psi for one hour to allow sufficient wetting of the surrounding soil.
7. Withdraw the wetting cone and then reattach the standard CPT cone to the rig.

8. Lower the CPT system to the wet depth, conduct another standard Cone Penetration Test (CPT), and record the readings for "wet" tip resistance, sleeve friction, and pore water pressure.

3.3 Laboratory Tests

At each location, two Shelby tube samples were collected for laboratory testing. The samples were taken approximately one foot (0.3 m) away from the corresponding CPT holes. The laboratory tests conducted on these samples included determinations of unit weight (ASTM D-7263), moisture content (ASTM D-2216), particle size distribution (ASTM D-6913 and D-7928), Atterberg limits (ASTM D-4318), and shear strength based on the direct shear test (ASTM D-3080). Details about the direct shear test method used in this research are shared in the following sections.

3.3.1 Direct Shear Apparatus

The direct shear test was conducted following ASTM D-3080 to evaluate the shear strength of the soils under two conditions: at their natural field moisture content and after being submerged in water for 48 hours. This method was employed to replicate the field conditions present during the Cone Penetration Tests (CPT). DigiShear Automated Shear System was used to find the shear strength of the soil, the device is shown in Figure 3.15.

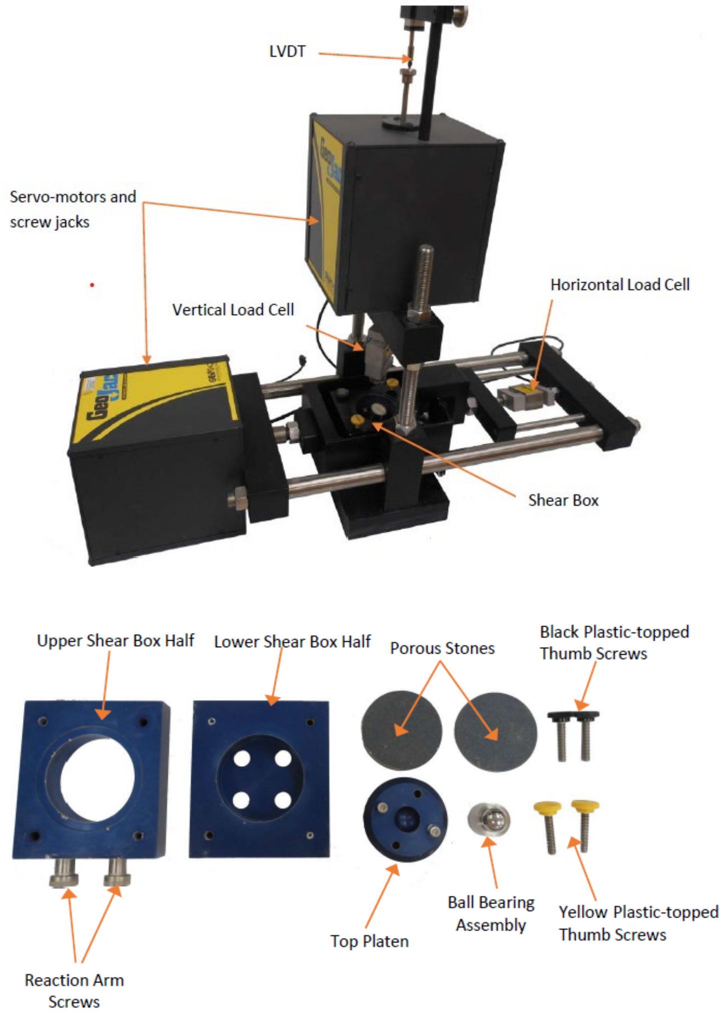

Figure 3.15 DigiShear Automated Direct Shear System and Shear Box Assembly (Castellanos, 2014)

3.3.2 Sample preparation

The samples for field moisture condition testing were extracted from the Shelby tube using steel cutting rings, 2.5 inches in diameter and 1.3 inches in height, which corresponded to the final sample size. The sample was then gently placed into the shear box using a porous stone. The process of preparing the field moisture condition samples is illustrated in Figure 3.16.

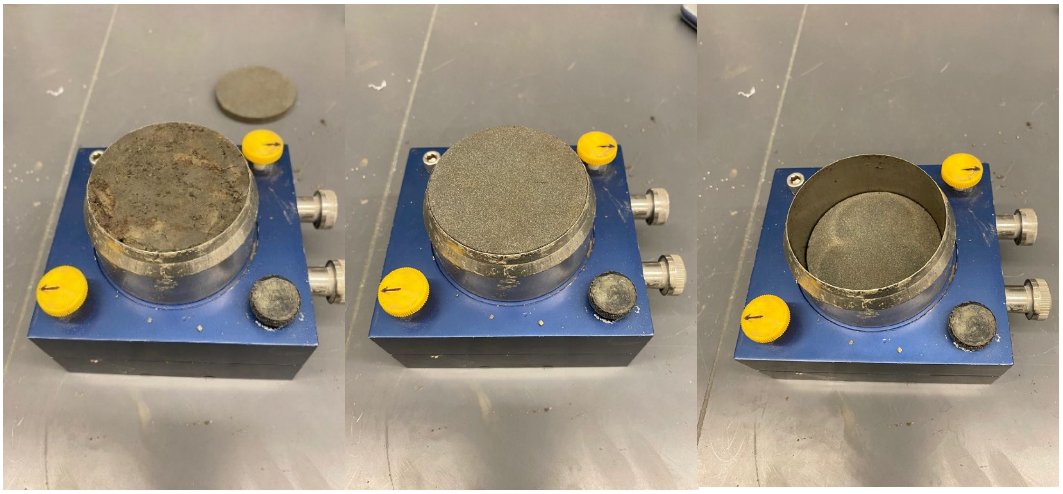

Figure 3.16 Field Moisture Condition Sample Preparation for Direct Shear Test

The samples prepared for testing under laboratory wet conditions were trimmed using the same method as the samples for field moisture conditions. They were then submerged in water for 48 hours while remaining inside the cutting ring. The cutting ring stayed in place during the submersion, ensuring that the sample was fully submerged and could be easily transferred to the shear box without any disturbance. Submerged samples are shown in Figure 3.17.

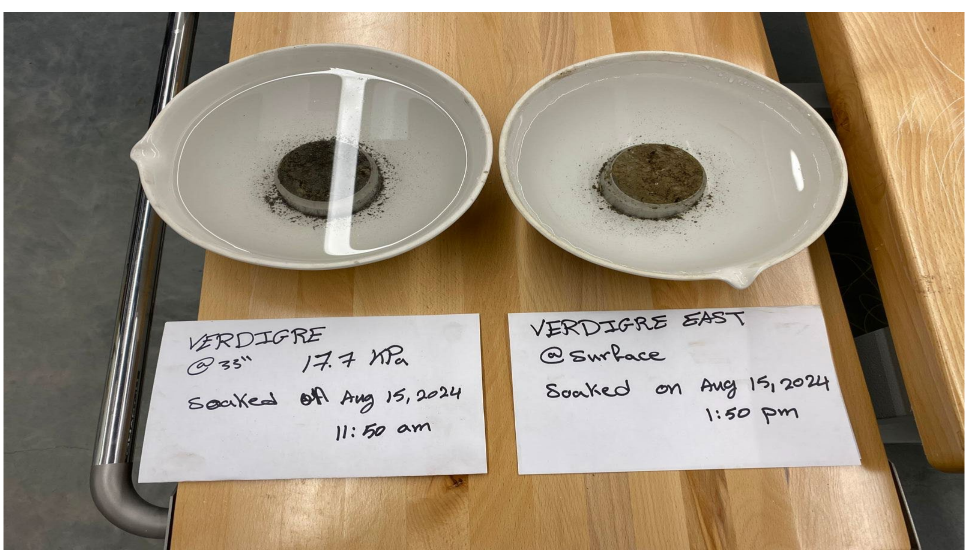

Figure 3.17 Submerged Direct Shear Samples

The Direct Shear Test was conducted on three separate samples under field moisture conditions at each location. Each test was performed at varying normal stress levels: 14.5 psi (100 kPa), 7.3 psi (50 kPa), and approximately 3 psi (20 kPa). The normal stress of 3 psi (20 kPa) was selected to simulate the overburden pressure at a depth of 3 ft (1 m), based on unit weight calculations. Additionally, the test was repeated on three different samples under laboratory wet conditions after they had been submerged in water for 48 hours, using the same normal stress levels. To mimic the field conditions in which the revised Cone Penetration Test (CPT) was carried out where no time was allowed for consolidation, the consolidation phase of the Direct Shear Test was omitted. A displacement rate of 0.02 inches per minute (0.51 mm/min) was applied, which falls within the range recommended by ASTM D-3080.

Chapter 4 Test Results and Discussion

Chapter 4 summarizes the unit weight, moisture content, gradation analysis, Atterberg limits, direct shear, and revised piezocone penetration test (CPT) results for the tested locations except for Santee spur, as the soils were sandy and sampling was not easy using Shelby tubes.

4.1 Unit Weight and Moisture Content.

The results of the unit weight (ASTM D-7263) and moisture content (ASTM D-2216) tests for all tested locations are summarized in Table 4.1.

Table 4.1 Unit Weight and Moisture Content.

Location	**Moist Unit Weight (lb/ft^3)**	**Moisture Content (ω%)**
East Campus	100.6	18.6
Verdigre West	112.7	28.5
Verdigre East	129.1	19.3
Bristow	108.9	28.3
Lincoln I-180	120.9	8.0
Lincoln Hwy-2	121.3	20.9
Greenwood	135.2	20.3
US-75	96.7	17.4
Waverly	90.1	19.3
Norfolk Hwy-275	115.1	12.3

The unit weight ranges from 90.1 lb/ft^3 at Waverly to 135.2 lb/ft^3 at Greenwood, indicating considerable soil compaction and density variability across the sites. The moisture content ranged from 8.03% at Lincoln I-180 to 28.5% at Verdigre West, higher moisture content typically reduces shear strength. While higher unit weights, such as those observed in Greenwood (135.2 lb/ft^3), suggest denser soils with potentially higher shear strength, moisture content can still play a significant role in weakening the soil, especially when it leads to higher pore water pressures or reduced friction between particles. In contrast, locations such as Waverly (90.1 lb/ft^3, 19.3% moisture) with lower unit weight and moderate moisture content may indicate looser, less compacted soils that are more susceptible to shear failure under loading.

4.2 Gradation Analysis and Atterberg Limits

The gradation curves for all tested samples were obtained based on ASTM D-6913 sieve analysis and ASTM D-7928 hydrometer analysis, as shown in Figure 4.1. Additionally, Atterberg limits including liquid limit (LL), and plastic limit (PL) tests were conducted based on ASTM D-4318, as presented in Table 4.2, along with the clay size fraction from the particle size distribution.

According to a unified soil classification system (USCS), the tested locations were categorized into three primary groups: clayey sand (SC), fat clay (CH), and lean clay (CL). The data reveals that different soil types shared a similar clay size fraction, approximately 10%. This includes Lincoln I-180 (SC), US-75 (CL), Norfolk (CL), and Waverly (CH). In contrast, Verdigre South and East (SC), Lincoln Hwy-2 (SC), and UNL East Campus (CH) had a clay size fraction closer to 20%. Greenwood (CL) contained a higher proportion of finer particles compared to the other locations. Notably, the Bristow gradation exhibited a gap-graded distribution, which is uncommon in natural soil.

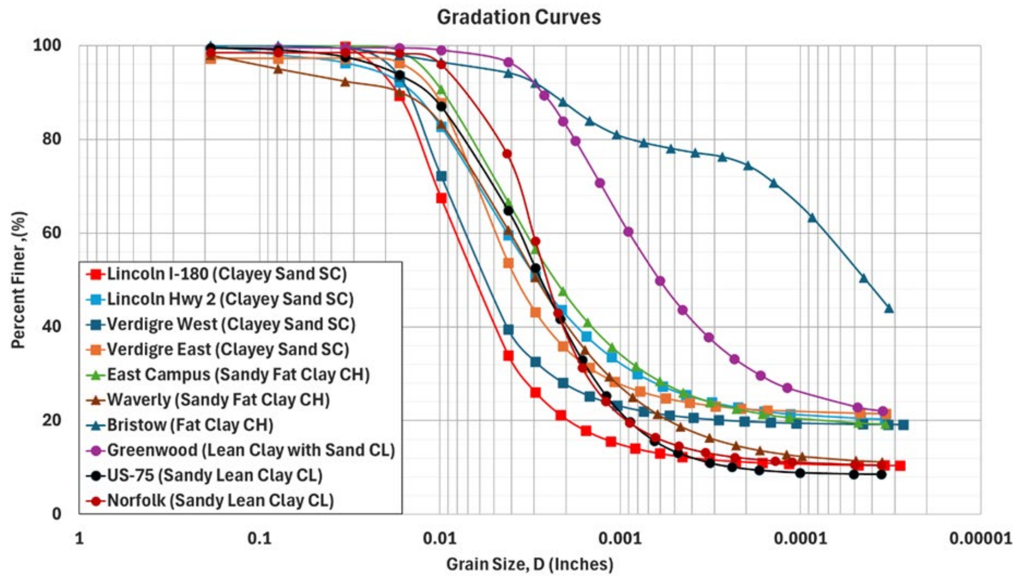

Figure 4.1 Gradation Curves for All Samples

Soil classification and the obtained categories provide valuable insight into the behavior of soils at each location, particularly regarding shear strength and moisture sensitivity. Cohesive soils, such as CH (Sandy Fat Clay) and CL (Lean Clay with Sand), exhibit high plasticity and are notably sensitive to changes in moisture content. For example, Bristow (CH), with a high Plasticity Index (PI) of 40.7 and a clay fraction of 62%, is supposed to be prone to volumetric changes.

Table 4.2 Atterberg Limits and Clay Size Fraction

Location \ Atterberg Limit	Liquid limit (LL%)	Plastic Limit (PL%)	Plasticity Index (PI%)	% Clay Fraction
Lincoln I-180	44.3	21.1	23.1	10.5
Lincoln Hwy 2	58.3	26.2	32.2	21
Verdigre South	69.2	26.5	42.7	19.3
Verdigre East	67.2	27.7	39.6	22
East Campus	55.7	26.4	29.4	20
Waverly	51.9	21.3	30.6	12
Bristow	68.9	28.2	40.7	62
Greenwood	45.8	22.0	23.8	25
US – 75	34.7	18.7	16	9
Norfolk	39.7	22.0	17.7	11

Soils classified as SC (Clayey Sand), such as those at Lincoln I-180, Lincoln Hwy-2, Verdigre West, and Verdigre East, have a moderate to high PI ranging from 23.1 to 42.7, reflecting a mix of sandy and cohesive properties. The clay fraction in these soils ranges from 10.5% to 22%, significantly affecting their behavior. Locations with higher clay fractions, such as Verdigre East (22%) and Lincoln Hwy-2 (21%), exhibit higher cohesion but are more prone to reductions in shear strength when wet, as the retained moisture reduces interparticle friction and cohesion. In contrast, soils with lower clay fractions, such as Lincoln I-180 (10.5%), are less sensitive to moisture and maintain more stable shear strength. However, their lower cohesion may make them more susceptible to deformation under load. The sandy component in these soils provides additional frictional resistance, giving them greater overall stability compared to more cohesive soils, though excessive moisture can still compromise their shear strength.

Lean clays with sand (CL), such as those found at Greenwood, US-75, and Norfolk Hwy-275, exhibit lower plasticity than fat clays (CH). Their PI ranges from 16 to 30.6, and their clay fractions vary between 9% and 25%. Locations such as US-75 (9%) and Norfolk Hwy-275 (11%) have lower clay content, making these soils less prone to shrink-swell behavior and more stable under changing moisture conditions. Greenwood, with a higher clay fraction of 25%, retains more moisture, increasing cohesion but also amplifying susceptibility to swell upon absorbing water. Lean clays generally demonstrate better stability than highly plastic soils. However, their clay content remains an important factor influencing their performance, particularly in wet conditions where higher clay content can lead to increased moisture retention and reduced shear resistance.

To summarize, the tested materials were classified into three categories: clayey sand (SC), fat clay (CH), and lean clay (CL). Each category is expected to show different strength behavior and different responses to moisture content change.

4.3 Revised Piezocone Penetration Test (CPT) and Direct Shear Test Results

The revised CPT test results for various moisture conditions, including in-situ and post-injection conditions, are presented alongside direct shear tests at field moisture and after 48 hours of submersion to validate the revised CPT effectiveness in capturing fully softened shear strength in the field.

4.3.1 UNL East Campus

At the UNL East Campus site, the soil is classified as Sandy Fat Clay (CH) with 20% clay content and a plasticity index of 29.4. Initial CPT data (blue lines in Figure 4.2) show high tip resistance (q_c). However, after 30, 60, and 90 minutes of water injection (red, green, and yellow lines), both q_c and sleeve friction (f_s) drop by roughly 65%, with q_c decreasing from 580 psi to 150 psi and f_s from 15 psi to 5 psi (Figure 4.2).

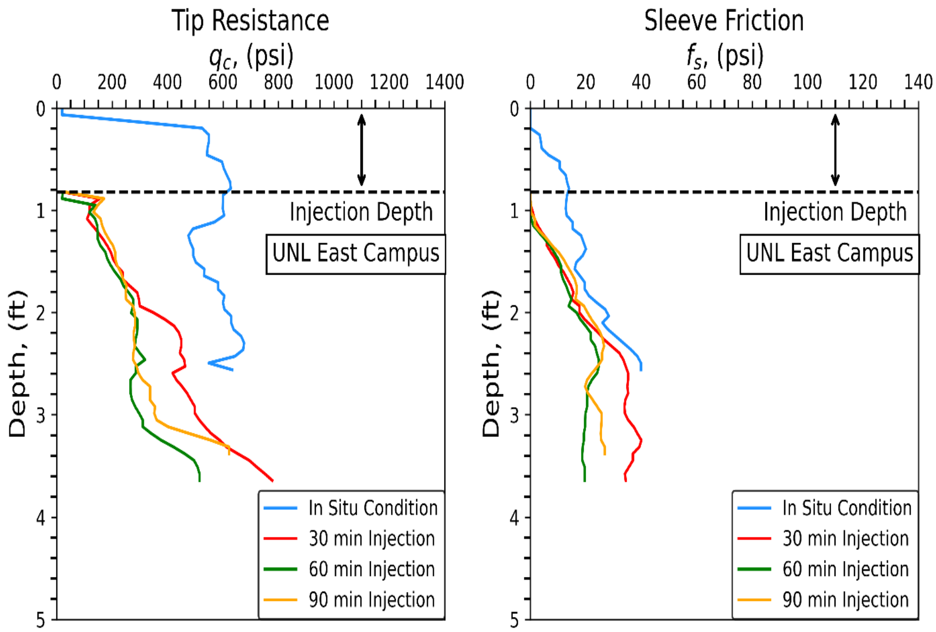

Figure 4.2 UNL East Campus Revised CPT Results

The revised CPT was able to capture the wet softened strength of the soil due to moisture variation, and the disturbance from the sleeve as the generated condition is close to the fully softened condition.

The direct shear test results in Figure 4.3 confirm that moisture reduces shear strength, with a reduction of about 80%, 45%, and 30% under normal stresses of 3 psi, 7.3 psi, and 14.5 psi, respectively. The significant reduction at lower normal stresses highlights the role of cohesion; moisture weakens interparticle bonding and increases pore pressure. As confinement increases, the reduction in shear strength decreases, indicating that higher normal stress helps mitigate moisture effects. This is consistent with CPT findings, indicating a 65% reduction in tip resistance after water injection, particularly at shallow depths (1 – 2.5 ft) due to lower overburden pressure and higher moisture susceptibility.

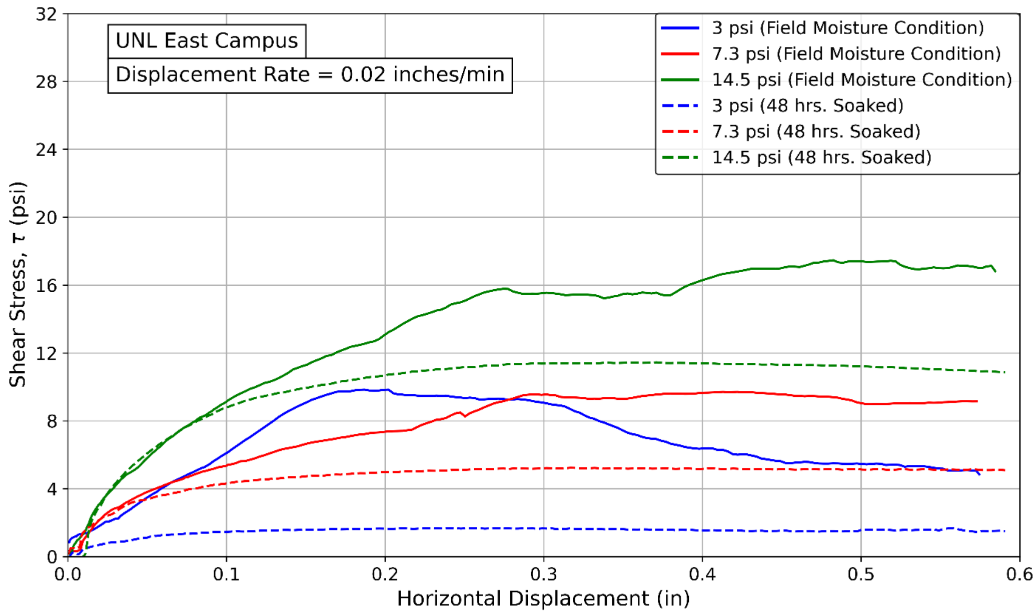

Figure 4.3 UNL East Campus Shear Stress vs. Horizontal Displacement

4.3.2 Lincoln I-180

Lincoln I-180 is classified as Clayey Sand (SC), which contains a 10.5% clay size fraction and has a plasticity index PI of 23.1. Figure 4.4 presents the revised (CPT) results for Lincoln I-180. Under in-situ conditions (blue lines in Figure 4.4), the Clayey Sand (SC) exhibited a tip resistance (q_c) of approximately 500 psi and sleeve friction (f_s) of around 25 psi. After 30 minutes of water injection, q_c decreased to 300 psi, reflecting a 40% reduction, while f_s dropped to 10 psi, indicating a 60% reduction. A similar trend was observed after 60 minutes of water injection, where q_c and f_s stabilized at 300 psi and 10 psi, respectively. However, the effect of water injection extended to a depth of three feet after 60 minutes, whereas after 30 minutes, the impact was limited to just 1.5 feet below the injection depth.

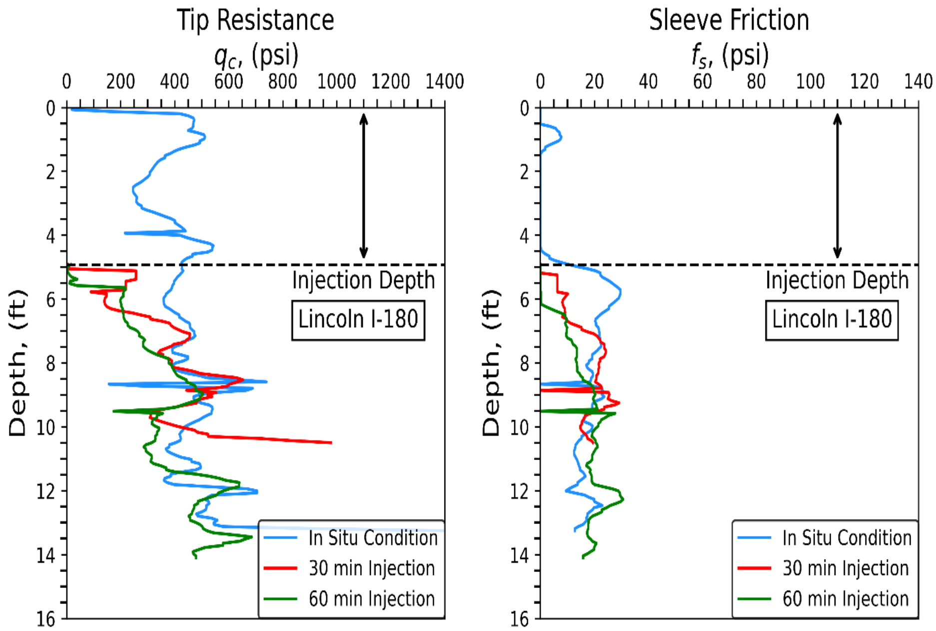

Figure 4.4 Lincoln I-180 Revised CPT Results

The soil's composition primarily governs this response. The sandy fraction facilitates rapid drainage, expediting pore water dissipation and contributing to the limited depth effected by water injection. Conversely, despite its moderate PI, the clay fraction exhibits notable moisture sensitivity, leading to a pronounced reduction in tip resistance (q_c) and sleeve friction (f_s) as interparticle cohesion is weakened.

The direct shear test results further corroborate these findings, revealing reductions in shear stress of 78% at 3 psi, 45% at 7.3 psi, and 48% at 14.5 psi after 48 hours of submersion, as presented in Figure 4.5. These reductions align with the CPT trends, where tip resistance (q_c) was reduced by 40.0% and sleeve friction (f_s) was reduced by 60.0% upon water injection. The more pronounced reduction in f_s and the substantial strength loss at lower normal stresses in the shear test highlight the material's moisture sensitivity.

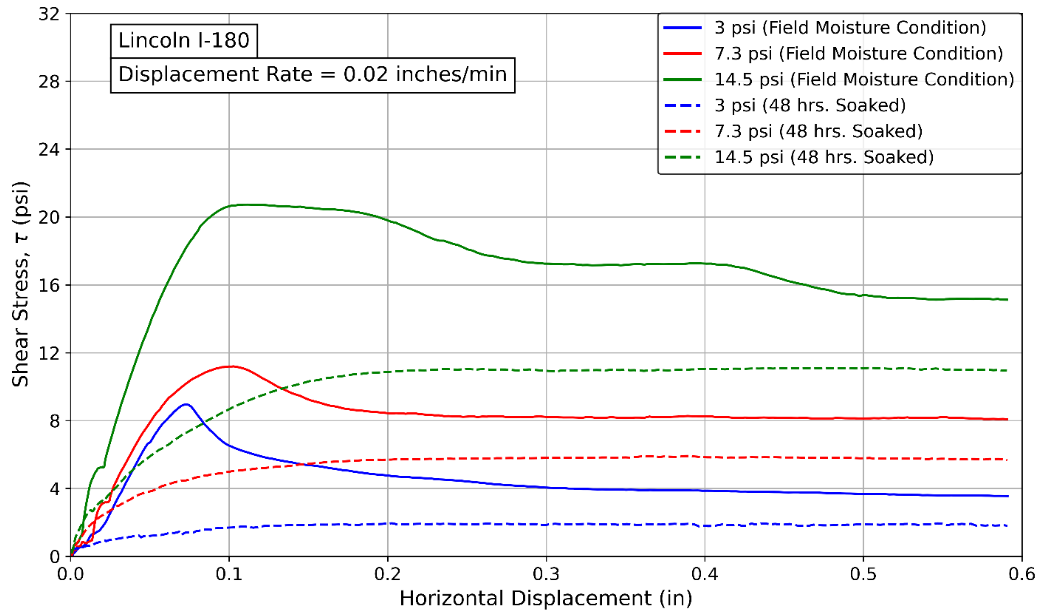

Figure 4.5 Lincoln I-180 Shear Stress vs. Horizontal Displacement

4.3.3 Lincoln Hwy-2

Lincoln I-180 is classified as Clayey Sand (SC), which contains a 21% clay size fraction and has a plasticity index PI of 32.2. Lincoln Hwy-2 soil after drilling is shown in Figure 4.6. The results from the revised Cone Penetration Test (CPT) indicated that there was no reduction in strength after injecting water into the soil for 60 minutes. In fact, both the tip resistance (q_c) and sleeve friction (f_s) values were higher compared to the in-situ condition results, as illustrated in Figure 4.7.

Figure 4.6 Lincoln Hwy-2 Soil After Drilling

Given the high plasticity index (32.2) and clay fraction (21%), a reduction in shear strength upon wetting is anticipated, consistent with observations at previous test locations. However, the unexpected behavior observed in this case may be attributed to variability in soil composition as a natural material. Additionally, the rapid drainage of the sand fraction could have facilitated a decrease in pore water pressures, mitigating the expected strength reduction. Furthermore, the specific clay mineralogy plays a role, as variations in swelling potential and moisture sensitivity can significantly influence the soil's mechanical response.

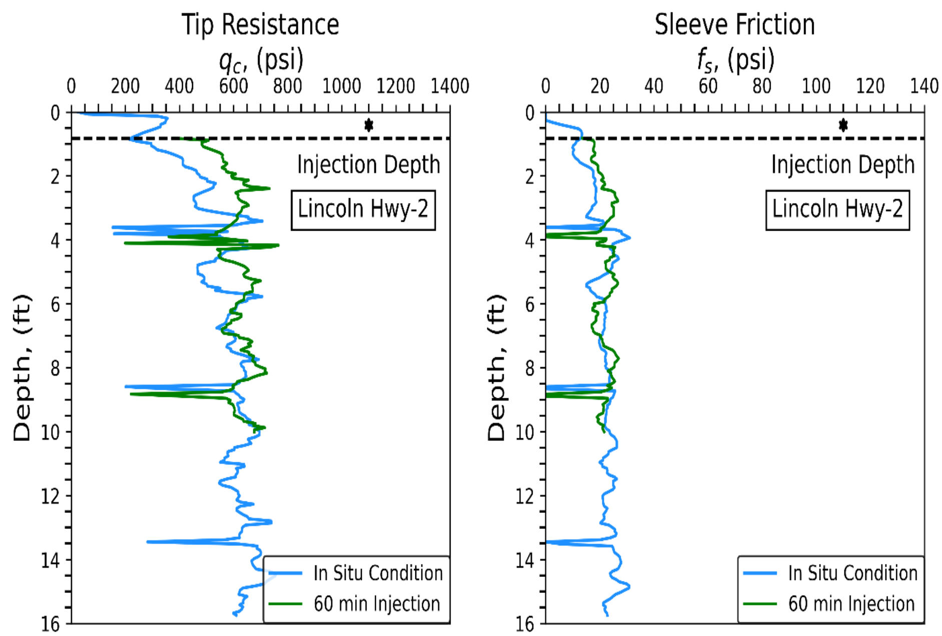

Figure 4.7 Lincoln Hwy-2 Revised CPT

The direct shear test results indicate that the initial peak shear strength under field moisture conditions suggests an over consolidated state, likely influenced by negative pore water pressure, as the field moisture content at Lincoln Hwy-2 was 20.9%. However, after 48 hours of soaking, the soil exhibits a substantial reduction in shear resistance, with shear stress decreasing by 78% at 3 psi, 70% at 7.3 psi, and 43% at 14.5 psi, as illustrated in Figure 4.8.

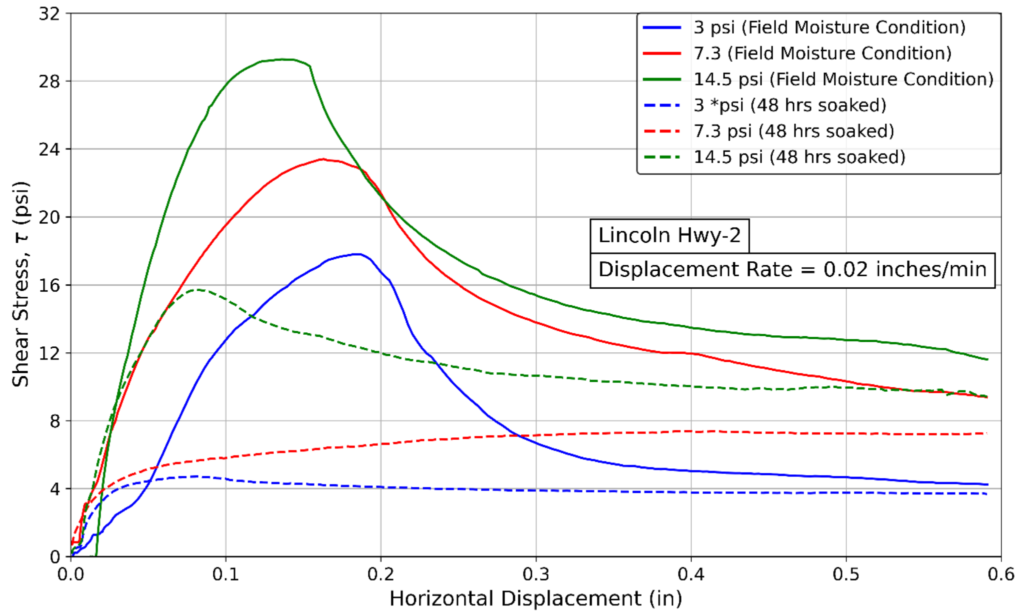

Figure 4.8 Lincoln Hwy-2 Direct Shear Test Results

The observed strain-softening behavior is attributed to the high plasticity index (32) of the Lincoln Hwy-2 soil and prolonged soaking, which leads to the breakdown of interparticle bonds due to swelling and reduced negative pore water pressure. This results in a significant loss of shear strength compared to field moisture conditions. The highest strength reduction occurred at 3 psi normal stress, suggesting that under low confining pressures, the soil experiences greater volume expansion (swelling), reduced effective stress, and a more pronounced loss in shear strength.

4.3.4 Verdigre South

Verdigre South is classified as Clayey Sand (SC) with a clay fraction of 19.3% and a Plasticity Index (PI) of 42.7, indicating a highly plastic and low-permeability soil. The revised CPT results under in-situ conditions show an initial tip resistance (q_c) of approximately 450 psi and sleeve friction (f_s) of 20 psi, as demonstrated in Figure 4.9. Following 60 minutes of water injection, (q_c) experienced an 89% reduction to 50 psi, while (f_s) diminished entirely, reflecting a

100% loss. The depth of influence remained constrained to approximately 1 ft, demonstrating the soil’s low hydraulic conductivity and its tendency to limit vertical moisture migration. The pronounced reduction in strength parameters at shallow depths suggests a moisture-induced degradation of cohesion, driven by the high clay content and plasticity.

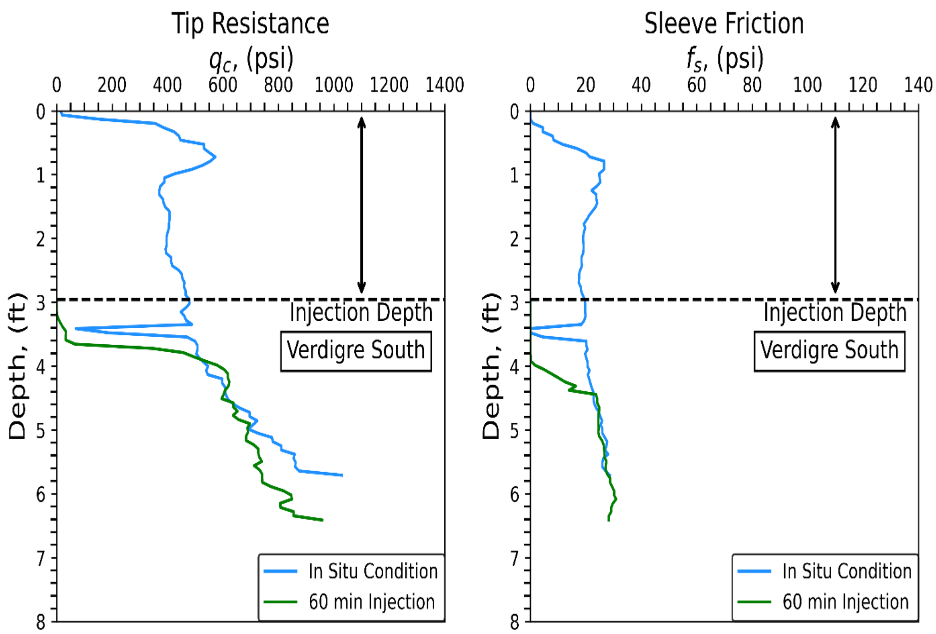

Figure 4.9 Verdigre South Revised CPT Results

The high PI and significant clay fraction contribute to the soil’s low permeability, restricting water movement beyond the immediate injection zone. Coarser-grained soils, where water disperses more uniformly, retains moisture near the injection point and confines the softening effect to the uppermost layer. The complete loss of f_s at the affected depth indicates a breakdown of effective stress due to excess pore water retention, highlighting the soil’s susceptibility to localized strength loss.

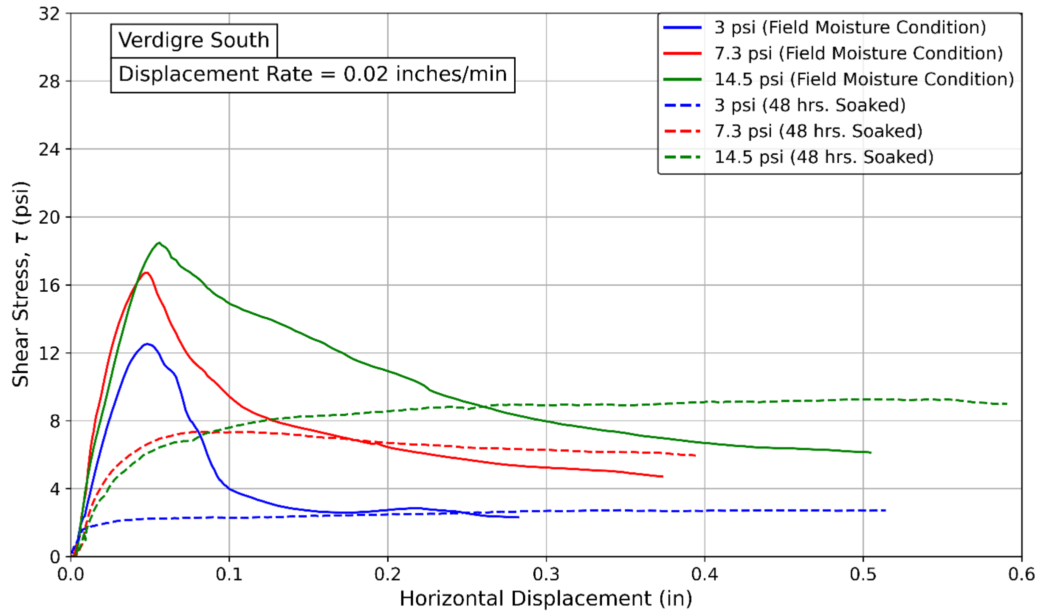

Figure 4.10 Verdigre South Direct Shear Results

Direct shear test results align with CPT findings, showing shear stress reductions of 75% at 3 psi, 65% at 7.3 psi, and 25% at 14.5 psi after soaking, as shown in Figure 4.10. The greater reduction at lower normal stresses suggests cohesion degradation as the primary failure mechanism. However, the limited penetration depth observed in the CPT confirms that while moisture significantly weakens shear strength, low permeability restricts the softening depth, localizing the effect to the injection zone.

4.3.5 Verdigre East

Verdigre East, classified as Clayey Sand (SC) with a clay fraction of 22% and a PI of 39.6, exhibited similar CPT behavior to Verdigre South. The initial tip resistance (q_c; blue line in Figure 4.11) of 300 psi dropped to 50 psi after 30 minutes of water injection (red line in Figure 4.11), an 83% reduction. Sleeve friction (f_s) decreased from 15 psi to 5 psi (66% reduction), with a 100% strength reduction immediately below the injection depth, as presented in Figure 4.11. The depth of influence remained around 0.5 ft, indicating low hydraulic conductivity and restricted vertical moisture dissipation. The response was consistent across both sites, with the only difference being an injection duration of 60 minutes in Verdigre South and 30 minutes in Verdigre East.

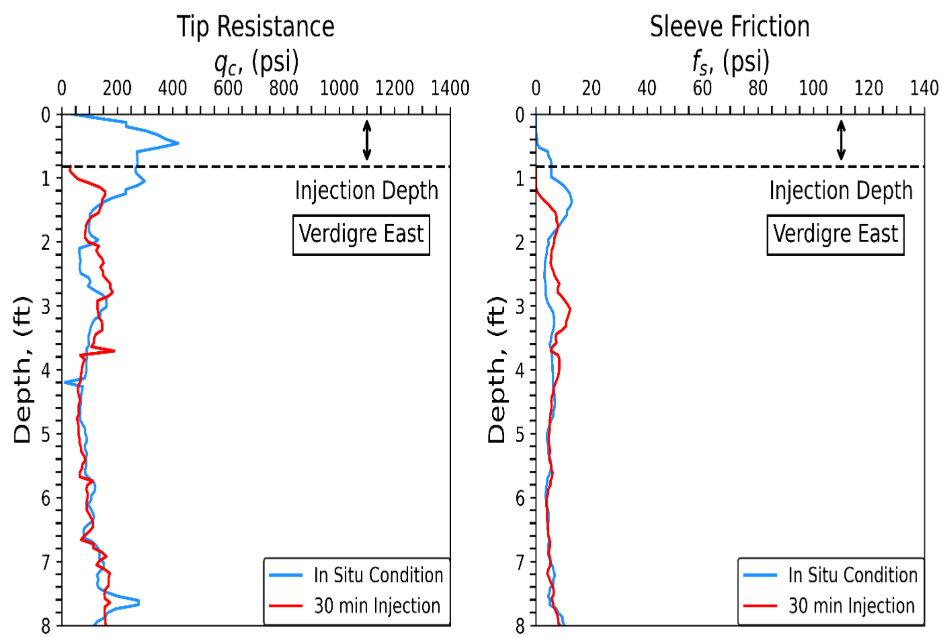

Figure 4.11 Verdigre East Revised CPT Results

The behavior confirms that the clay low permeability localized the strength reduction immediately below the injection depth, since the 30-min injection in Verdigre East impacted 0.5 ft below the injection depth, while 1 ft was impacted in Verdigre South after a 60-min injection.

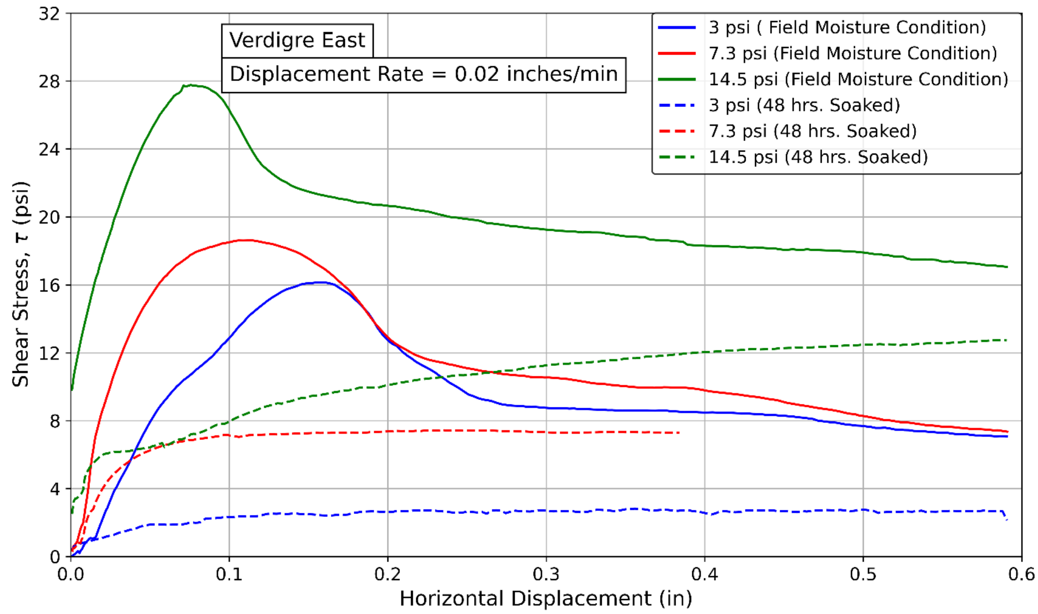

Figure 4.12 Verdigre East Direct Shear Test Result

The results of the direct shear tests align with the findings from the Cone Penetration Test (CPT), showing significant reductions in shear stress: 88% at 3 psi, 61% at 7.3 psi, and 57% at 14.5 psi after soaking, as illustrated in Figure 4.12. The peak shear strength observed in Verdigre South and Verdigre East indicates over consolidated behavior, which reflects the soil's low hydraulic conductivity and limited moisture dissipation.

Notably, Verdigre East exhibited a higher peak shear strength of 28 psi at a normal stress of 14.5 psi, which corresponds to a lower field moisture content of 19.3%. In contrast, Verdigre South showed a reduced peak strength of 18 psi at the same normal stress due to a higher field moisture content of 28.5%. This variation in moisture content affected the consistency of the soil, resulting in Verdigre East being in a semisolid state while Verdigre South remained in a plastic state. These findings further highlight the influence of moisture content on both shear strength and overall soil behavior.

4.3.6 Bristow

Bristow is classified as fat clay (CH) and has a clay content of 62%, a plasticity index (PI) of 40.7, and a field moisture content of 28.3%. The results from the Cone Penetration Test (CPT) conducted at Bristow show a significant decrease in tip resistance (q_c) and sleeve friction (f_s) following water injection, as illustrated in Figure 4.13. Under in-situ conditions, q_c decreased from 350 psi to 150 psi, representing a reduction of approximately 57% after 30 minutes of injection, and similarly at the 90-minute mark. At the same time, sleeve friction (f_s) dropped from 15 psi to 5 psi, indicating a reduction of about 67%. These reductions in q_c and f_s extended up to 1 ft below the injection depth after 30 minutes and up to 1.5 ft after 90 minutes.

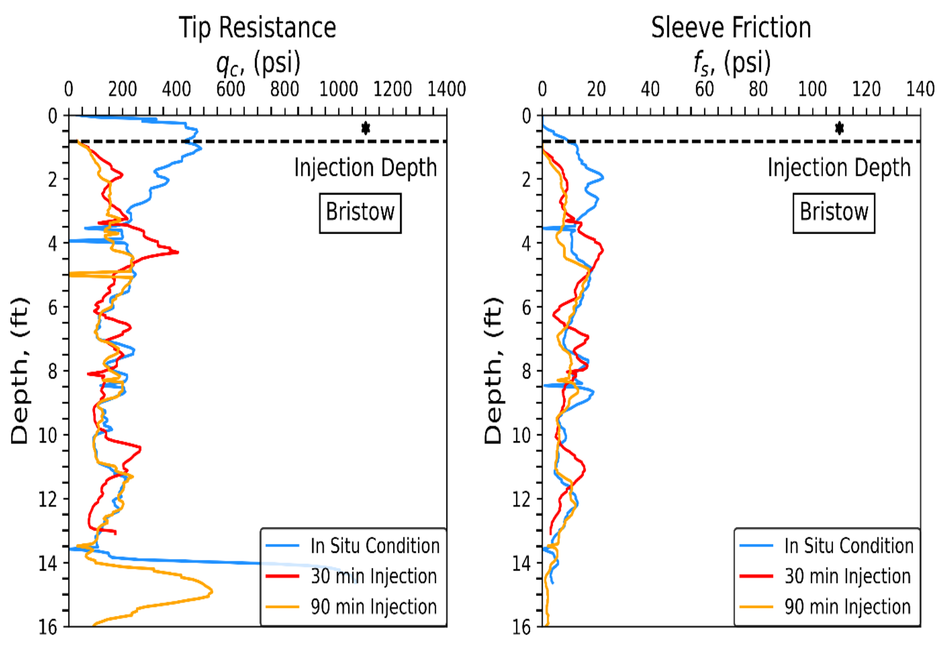

Figure 4.13 Bristow Revised CPT Results

The impacted depth variation between the 30-minute and the 90-minute injection reflects the low permeability of Bristow fat clay, which is expected due to the high clay content (62%).

These reductions align with the direct shear test results, which demonstrate a shear strength decrease of 79% at 3 psi, 75% at 7.3 psi, and 68% at 14.5 psi after 48 hours of soaking, as presented in Figure 4.14. The initial peak shear strength under field moisture conditions indicates an over consolidated state, attributed to low hydraulic conductivity and strong inter-particle bonding. However, post-soaking, the soil exhibits a substantial reduction in shear resistance driven by clay swelling and inter-particle bond weakening.

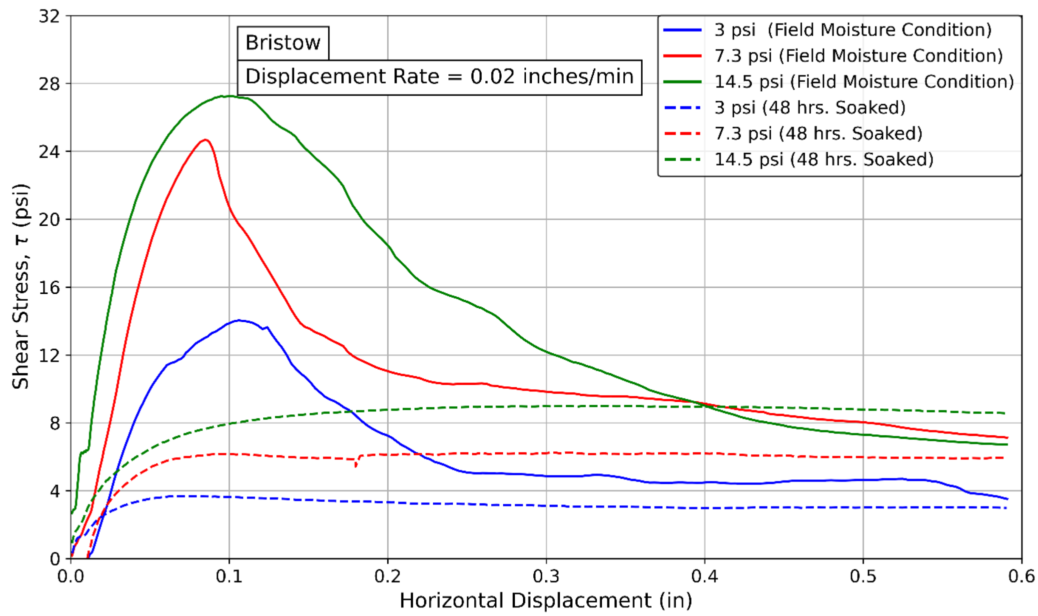

Figure 4.14 Bristow Direct Shear Test Results

4.3.7 US-75

US-75, is classified as sandy lean clay (CL) and has a clay content of 9%, a plasticity index PI of 16, and a field moisture content of 17.4%.The CPT results for US-75 indicate minimal impact from water injection, as both tip resistance (q_c) and sleeve friction (f_s) remain relatively unchanged across the tested depths, as shown in Figure 4.15. The penetration resistance curves for in-situ, 30-minute, and 60-minute injection conditions closely overlap, suggesting that the soil retains its strength despite moisture infiltration. This behavior can be attributed to the low clay fraction (9%) and moderate plasticity index (PI = 16), which limit the soil's susceptibility to significant strength loss upon wetting. Additionally, the lack of noticeable strength reduction below the injection depth implies that the permeability of this soil allows for rapid drainage, preventing pore pressure buildup and subsequent weakening.

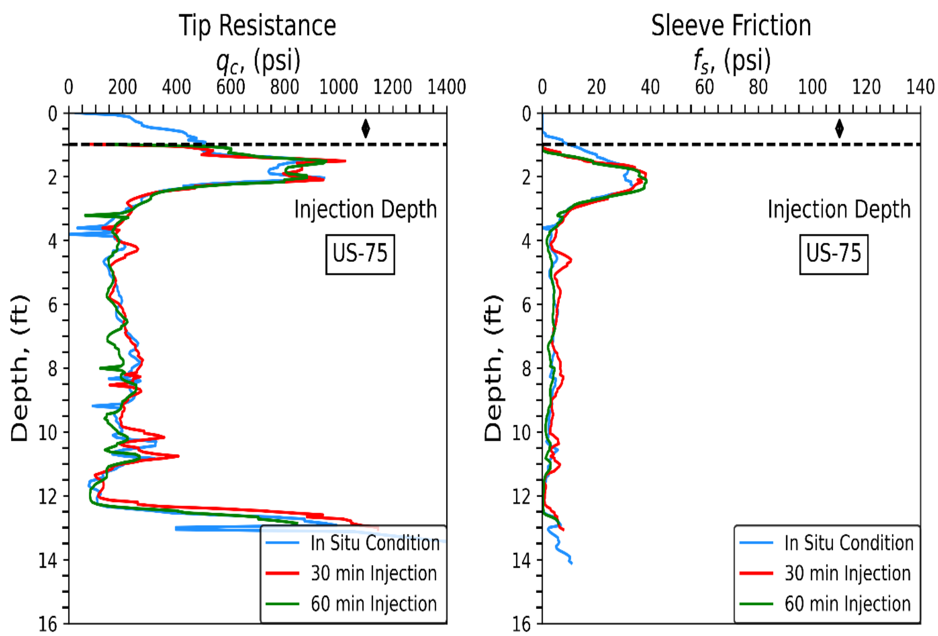

Figure 4.15 US-75 Revised CPT Results

The results of the direct shear test indicate there is no significant reduction in shear strength after eight hours of submersion, as shown in Figure 4.16. While there is a minor reduction in strength under normal stresses of 7.3 psi and 3 psi, the residual shear stresses remain consistent between the field moisture condition (17.4%) and the soaked condition (22.5%). The increase in moisture content after 48 hours of soaking is not substantial. However, when comparing the peak stresses of moisture field samples to the residual stresses of soaked samples, we observe reductions of 38% and 35% under normal stresses of 14.5 psi and 3 psi, respectively. This reinforces the findings from the Cone Penetration Test (CPT), showing that US-75 Sandy Lean Clay retains its structural integrity even under prolonged exposure to water. The minimal loss in strength highlights the stability of this soil type in wet conditions, making it less susceptible to strength degradation-related failures compared to more plastic clays such as CH or CL, which have a higher clay fraction.

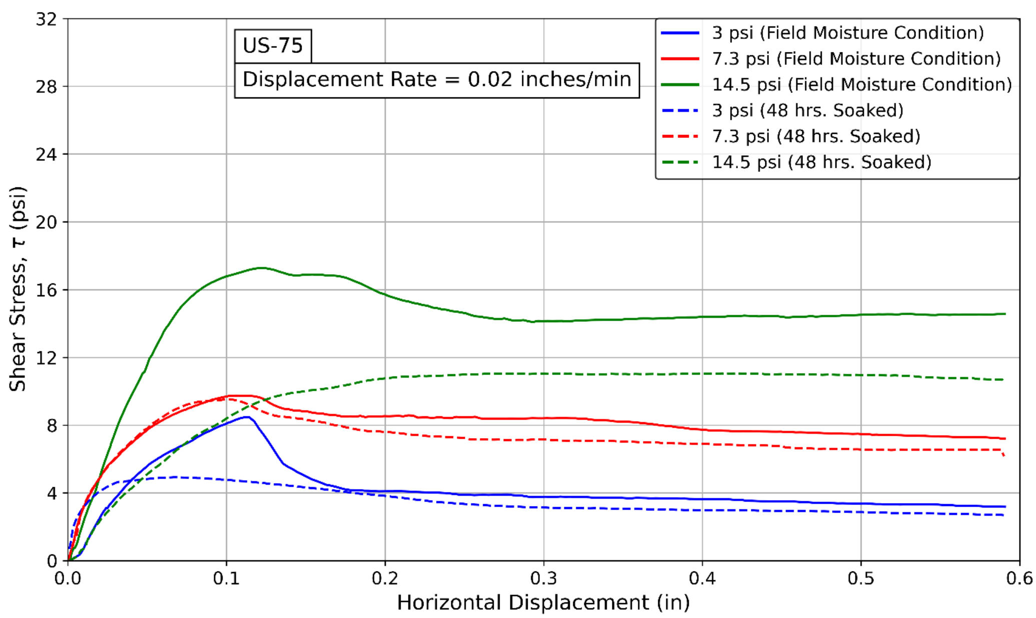

Figure 4.16 US-75 Direct Shear Results

4.3.8 Waverly

Waverly is classified as sandy fat clay (CH), with a clay content of 12%, a plasticity index (PI) of 30.6, and a field moisture content of 19.3%. The (CPT) results for Waverly indicate an average initial tip resistance (q_c) of 500 psi at a depth of three feet below the injection point, along with an average sleeve friction (f_s) of 20 psi at the same depth, as shown Figure 4.17. After 30 minutes of injection, the tip resistance (q_c) decreased to 250 psi, reflecting a reduction of

50%. After 60 minutes of injection, it further decreased to 180 psi, indicating a total reduction of 64%. Concurrently, sleeve friction (f_s) dropped from 20 psi to 10 psi after both 30 and 60 minutes of injection, which also represents a 50% reduction. The depth affected by water injection is approximately one foot below the injection point.

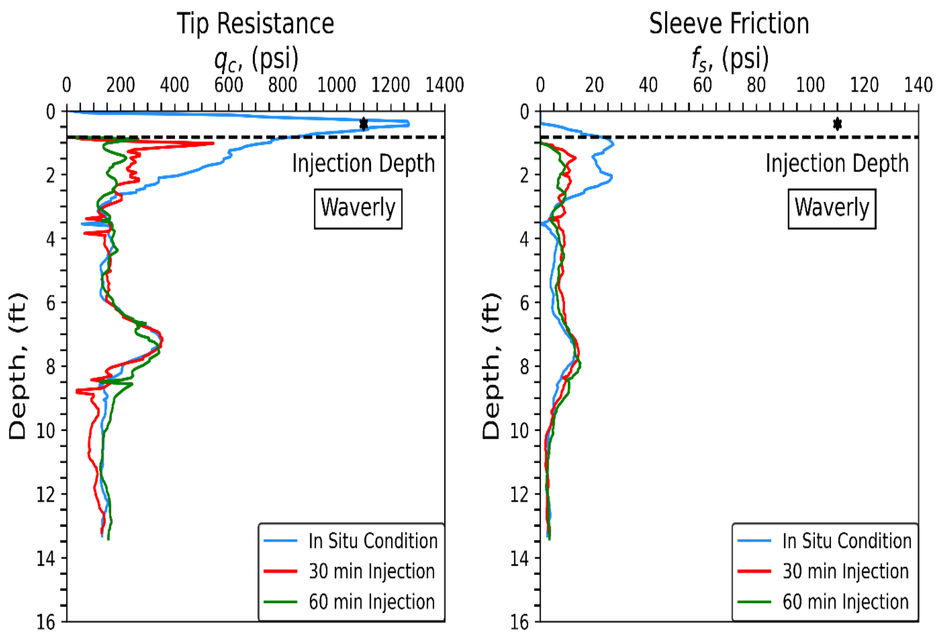

Figure 4.17 Waverly Revised CPT Results

The high plasticity of Waverly soils (30.6) explains the immediate reduction in tip resistance and sleeve friction, as these soils are more susceptible to swell and absorb more water, which leads to softening. However, the limited depth impacted can be explained by the low hydraulic conductivity of Waverly soils, which has a high clay content of 12%.

The results of the direct shear test show a significant reduction in shear strength as the soil absorbs water. Specifically, there are peak stress reductions of 83% at 3 psi, 64% at 7.3 psi, and 50% at 14.5 psi as shown in Figure 4.18. The soil is classified as sandy fat clay (CL), with a plasticity index (PI) of 30.6 and a clay fraction of 12%. This type of soil is particularly sensitive to moisture due to its relatively high plasticity and moderate clay content. The observed reduction in shear strength can be attributed to the breakdown of cohesion between particles. The large percentage of reduction at shallow depths may be explained by the low effective confining conditions in which the sample is more susceptible to swell and absorb water.

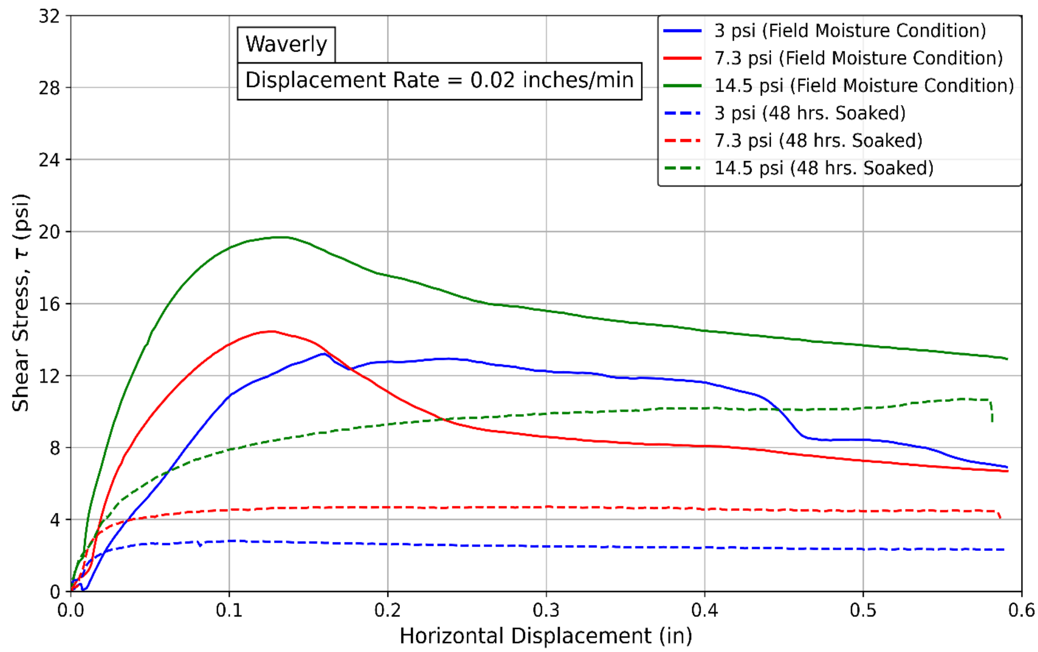

Figure 4.18 Waverly Direct Shear Test Results

4.3.9 Greenwood

Greenwood is classified as lean clay with sand (CL), with a clay content of 25%, a plasticity index (PI) of 23.8, and a field moisture content of 20.3%. The cone penetration test (CPT) results for Greenwood indicate a negligible reduction in both tip resistance (q_c) and sleeve friction (f_s) as shown in Figure 4.19. The lean clay with sand (CL) composition, coupled with a clay fraction of 25% and a PI of 23.8, suggests high plasticity and low hydraulic conductivity, as the field observations revealed that the injected water emerged from the borehole, indicating low hydraulic conductivity and limited dissipation.

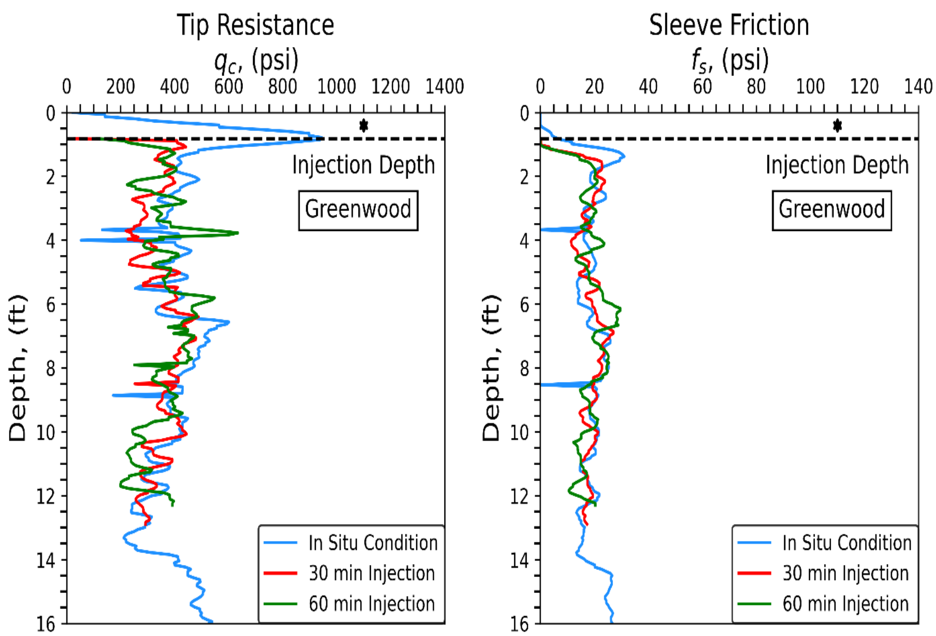

Figure 4.19 Greenwood Revised CPT Results

The results of the direct shear test indicate a notable reduction in shear strength as the soil becomes wet. Specifically, peak stress reductions of 83% occur at a normal stress of 3 psi, 40% at 7.3 psi, and 50% at 14.5 psi, as illustrated in Figure 4.20. The peak shear stress reached 17 psi at a normal stress of 3 psi, which is higher than the peak shear stress of 15 psi at a normal stress of 7.3 psi. This variation is attributed to the natural properties of the soil; not all tested samples will behave uniformly. The considerable decline in shear strength from peak to residual conditions can be linked to weak inter-particle bonds within the soil matrix (Skempton, 1970). These bonds act as a cohesive agent, holding soil particles together. The strength and effectiveness of these inter-particle bonds are influenced by the soil's clay mineral composition and its stress history (Li et al., 2016).

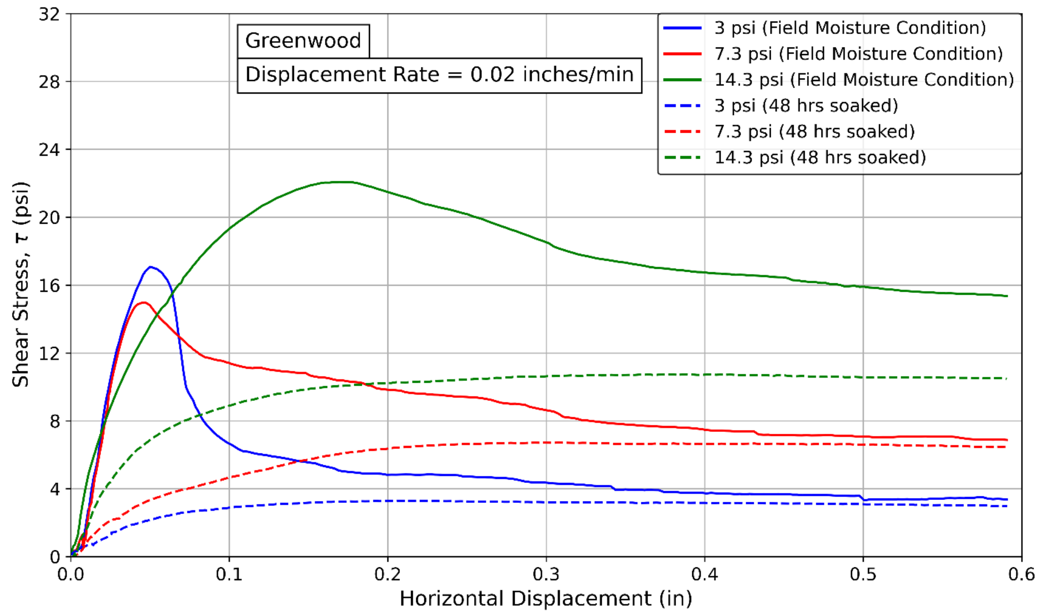

Figure 4.20 Greenwood Direct Shear Test Results

4.3.10 Norfolk, Hwy -275

The Norfolk location is classified as having sandy lean clay (CL), consisting of 11% clay content, a plasticity index (PI) of 17.7, and a field moisture content of 12.3%. The results from the cone penetration test (CPT) indicate that the tip resistance (q_c) gradually decreased from approximately 600 psi to around 450 psi after a 30-minute injection, reflecting a 25% reduction, as shown in Figure 4.21. After a 60-minute injection, the tip resistance (q_c) further declined to about 200 psi, resulting in a total reduction of 67%. The impacted depth reached 0.5 feet after the 30-minute injection and extended to approximately two feet below the injection point after 60 minutes. This shows the moisture sensitivity of the moderately plastic soil (PI of 17.7) and the drainage characteristics of sand. The sleeve friction was reduced from about 30 psi to 10 psi at 6 ft after a 60-min injection. The reduction happened 2 ft below the injection depth, indicating a 67% reduction.

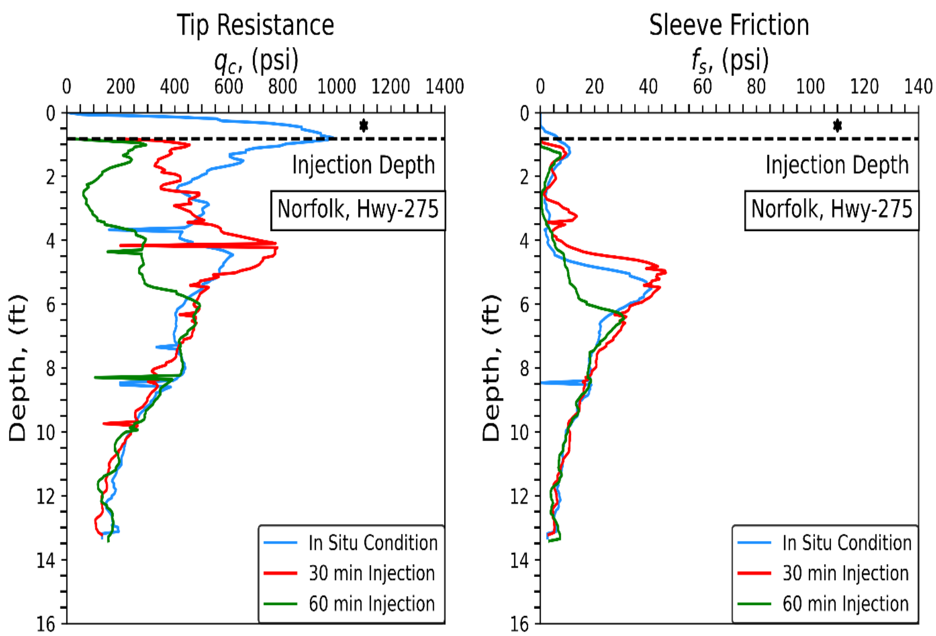

Figure 4.21 Norfolk Revised CPT Results

The results of the direct shear test indicate a high peak stress of 28 psi at a normal stress of 14.5 psi under field moisture conditions, primarily due to the low moisture content of 12.3%. The residual stress observed under these conditions was 14 psi, as illustrated in Figure 4.22. For the submerged sample tested at the same normal stress of 14.5 psi, the residual stress decreased to 11 psi, reflecting a 57% reduction. Furthermore, the results demonstrate a significant decrease in shear strength as the soil becomes wet, with a stress reduction of 25% to 7.3 psi. At low normal stress (3 psi), the shear strength of the soil remained extremely low, both under field moisture conditions and after 48 hours of submersion (shear stress reduced from 3 psi to 1 psi, 67% reduction). This diminished shear strength at lower normal stress is due to the sample's propensity to swell and absorb water, which reduces its strength.

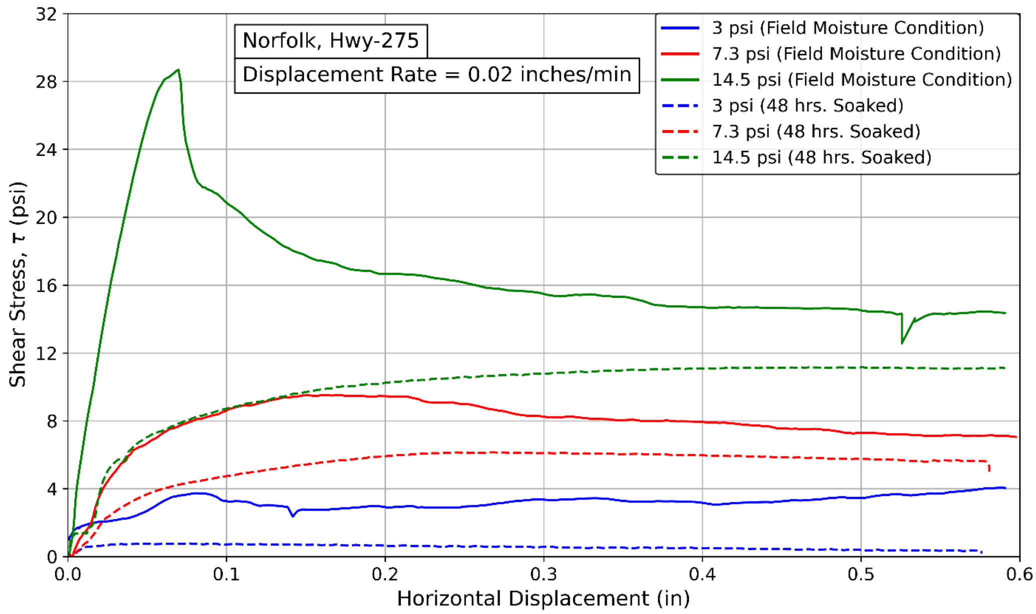

Figure 4.22 Norfolk Direct Sheat Test Results

4.3.11 Santee Spur

The Cone Penetration test (CPT) in Santee Spur, showed a substantial reduction in tip resistance and sleeve friction following a 60-min injection at a depth of 3.28 ft from the ground surface. The reduction occurred at 6 ft, which is approximately 2.7 ft below the injection depth, as shown in Figure 4.23. The tip resistance was reduced from 600 psi to 400 psi, a 33% reduction. While the sleeve friction decreased at the same depth from 20 psi to 15 psi, indicating a 25% reduction.

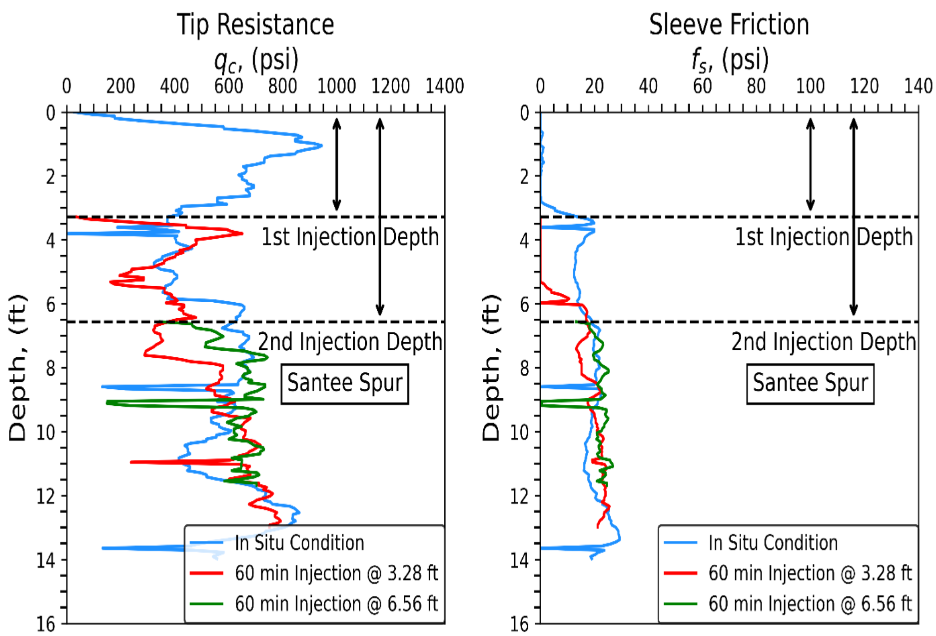

Figure 4.23 Santee Spur Revised CPT Results

The soil sample collected from the Santee Spur site was primarily composed of sand, with an inadequate amount for conducting direct shear tests or a grain size distribution analysis. Additionally, the lack of fine-grained material prevented an assessment of plasticity characteristics. However, the noticeable decrease in tip resistance and sleeve friction at a depth of 6 feet indicates the presence of a clay fraction that is sensitive to moisture fluctuations and demonstrates softening behavior when wet.

4.4 Test Results Summary

Table 4.3 presents a comparison of the reduction in cone penetration test (CPT) parameters, including tip resistance (q_c), sleeve friction (f_s), and direct shear test results across test sites. The reductions are recorded at 30 minutes, 60 minutes, and 90 minutes after moisture injection, where applicable. Reductions in direct shear test by submerging samples for 48 hours are shown for three normal stress conditions: 3 psi, 7.3 psi, and 14.5 psi.

Table 4.3 Summary of CPT and Direct Shear Test Reduction Percentages Across Test Sites

Location			CPT Reduction %						Direct Shear Test Reduction %		
			30 min		60 min		90 min				
Site	**PI**	**%CF***	**q_c**	**f_s**	**q_c**	**f_s**	**q_c**	**f_s**	**3 psi**	**7.3 psi**	**14.5 psi**
UNL East Campus	29.4	20	65	65	65	65	65	65	80	45	30
Lincoln I-180	23.1	10.5	40	60	40	60	NA	NA	78	45	48
Lincoln Hwy-2	32.2	21	NA	NA	0	0	NA	NA	78	70	43
Verdigre South	42.7	19.3	NA	NA	89	100	NA	NA	75	65	25
Verdigre East	39.6	22	83	100	NA	NA	NA	NA	88	61	57
Bristow	40.7	62	57	67	NA	NA	57	67	79	75	68
US-75	16	9	0	0	0	0	NA	NA	38	0	35
Waverly	30.6	12	50	50	64	50	NA	NA	83	64	50
Greenwood	23.8	25	0	0	0	0	NA	NA	83	40	50
Norfolk	17.7	11	25	0	67	67	NA	NA	67	25	57
Santee Spur*	NA	NA	NA	NA	33	25	NA	NA	NA	NA	NA

*%CF is the clay fraction of the sample as a percentage.
*Note: Santee Spur data was not available due to the inability of taking samples, the soil was sandy, and sampling was not easy using Shelby tubes.

The results of the revised (CPT) and direct shear tests provide valuable insights into the influence of moisture injection on soil strength parameters across multiple test sites. The key findings based on the observed reductions and soil behavior during CPT and direct shear tests are listed below:

- Sites with a higher plasticity index (PI) and clay fraction (%CF), such as Verdigre South, Verdigre East, and Bristow, exhibit significant reductions in q_c, and f_s indicating a pronounced softening effect due to moisture variation from high water retention and swelling potential.
- The US-75 and Greenwood sites showed no reduction, suggesting limited moisture sensitivity as US-75 has a low %CF of 9% and moderate plasticity (PI = 16%), while

Greenwood, with a %CF of 23.8%, showed a very low hydraulic conductivity and injected water coming out of the borehole, hence the impacted depth was limited.

- Norfolk and Waverly demonstrated a progressive reduction over time, as both locations have a relatively low %CF of 11%, and 12%, respectively. The low %CF minimized the moisture sensitivity even when the plasticity index is moderate to high (17.7%, and 30.6%, respectively). Other sites, such as Bristow and UNL East Campus, maintained similar reductions across multiple time intervals, indicating different softening mechanisms. The uniform reduction in these locations was due to the swelling characteristics of fat clay which was present at both locations.
- The direct shear results reveal that the highest shear stress reduction between samples tested under field moisture conditions and samples tested after 48 hours of soaking occurred at low confining stress (3 psi), as the sample may easily swell and absorb water under low confining stress.

Chapter 5 Correlation Between CPT and Direct Shear Test Results

This chapter presents the shear strength interpreted from the revised CPT and compares it with the shear strength obtained from the direct shear test in the laboratory to validate the accuracy of the revised CPT in evaluating the fully softened shear strength in the field.

5.1 Empirical correlation

The soil behavior type index (I_c), the soil undisturbed shear strength (S_u), and the disturbed shear strength ($S_{u(d)}$) were correlated based on the unified approach for interpretation of cone penetration tests suggested by Robertson's (2009). The tip resistance is typically related to the intact or undisturbed strength of soils, while the sleeve friction is related to the disturbed strength (Robertson, 2009; Song et al., 2021).

The friction ratio (R_f) is defined as the percentage of sleeve friction (f_s) to cone resistance (q_c). The corrected cone resistance (q_t) and R_f are calculated at the same depth using equations 5.1 and 5.2.

$q_t = q_c + u_2(1\text{-}a)$, where a = net area ratio, if u_2 is not available, $q_c = q_t$ (5.1)

$R_f = (f_s/q_t) \cdot 100\%.$ (5.2)

Normalized cone resistance (Q_t), normalized friction ratio (F_r), and I_c are calculated as:

$F_r = [(f_s/(q_t - \sigma_{v0})] \cdot 100\%$ (5.3)

$R_f = (f_s/q_t) \cdot 100\%.$ (5.4)

$I_c = ((3.47 - \log Q_t)^2 + (\log F_r + 1.22)^2)^{0.5}$ (5.5)

The peak shear strength, (S_u), the disturbed shear strength ($S_{u(d)}$), and soil sensitivity can be estimated using

$s_u = (q_t - \sigma_v)/ N_{kt}$ (5.6)

$S_{u(d)} = f_s$ (5.7)

$S_t = S_u / S_{u(d)}$ (5.8)

Where N_{kt} is a cone factor implemented in this study that varies from about 10 to 20 with an average of 14. The shear strength obtained from the direct shear test is based on the failure envelope that follows the equation $\tau = c + \sigma \tan(\emptyset)$, as presented in Figure 5.3. The shear strength parameters plot for all tested locations can be found in appendix B.

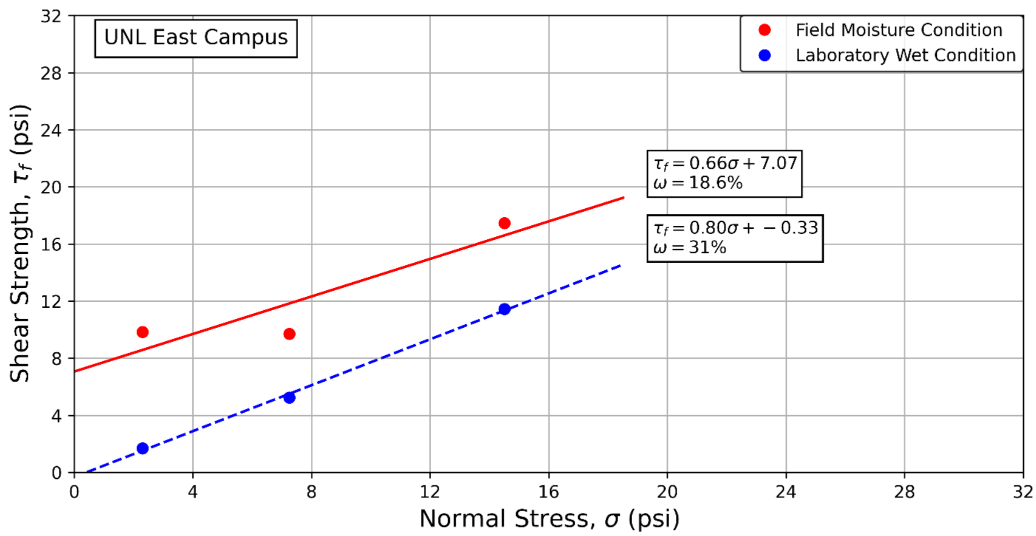

Figure 5.1 UNL East Campus Shear Strength Parameters

5.1.1 UNL East Campus

Figure 5.2 illustrates how the soil behavior type index (I_c) at the UNL East Campus testing site transitions from coarser-grained to finer-grained classifications as water is injected into the soil. This shift implies that the soil strength, stiffness and compressibility were impacted, and the fine material is now the dominant component that controls the soil behavior, since I_c depends on the above mentioned characteristics to classify the soils (Lunne et al., 2002b; Robertson, 2009).

Peak and disturbed shear strength correlated from the revised CPT for UNL East campus are shown in Figure 5.3. The in-situ peak shear strength S_u from the tip resistance was 40 psi on average from 1 to 2.5 ft, while the sleeve friction, which is related to the disturbed shear strength $S_{u(d)}$, at the same depth range was 20 psi on average. Both S_u and $S_{u(d)}$ reduced after water injection to approximately the same value (about 15 psi) as shown in Figure 5.3. The reduction in both S_u and $S_{u(d)}$ to the same value implies that the intact soil strength S_u was reduced to the fully softened shear due to the moisture variation, since it is close to the $S_{u(d)}$.

The high CPT correlated S_u compared to S_u measured from the direct shear test in the lab is related to the sample quality, as the Shelby tube samples may have been disturbed. The disturbance in the sample resulted in lower shear strength compared to the shear strength based on the CPT conducted in the field (Lunne et al., 2006).

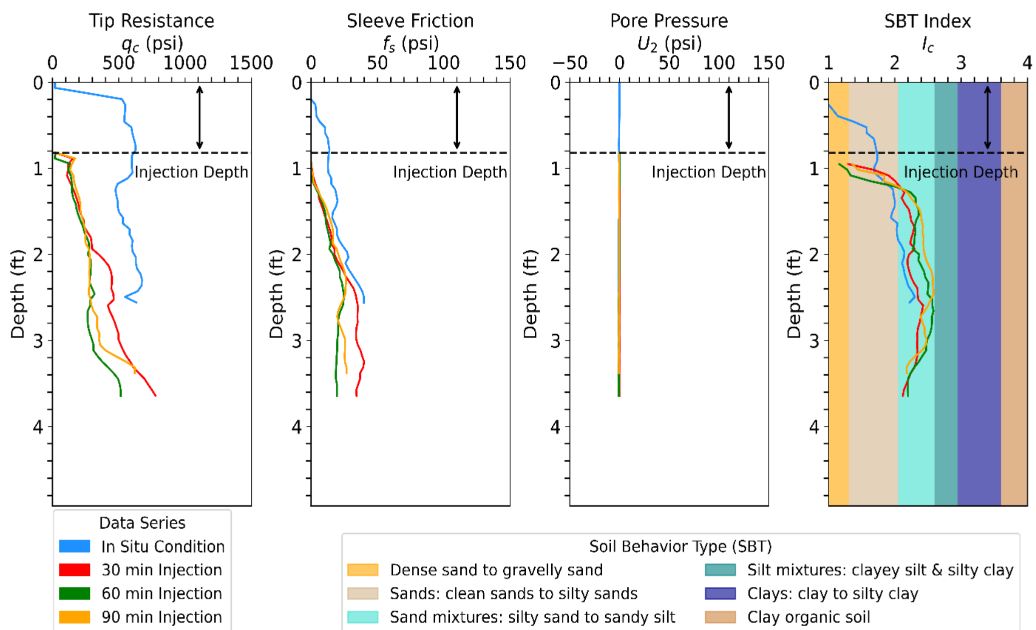

Figure 5.2 UNL East Campus SBT

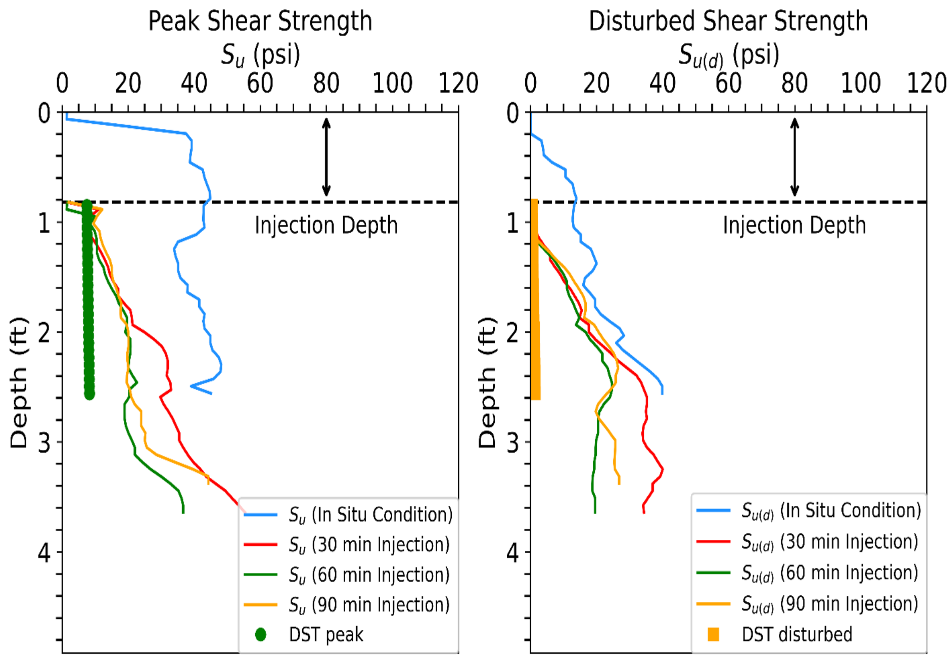

Figure 5.3 UNL East Campus Peak and Disturbed Shear Strength form CPT and DST

5.1.2 Lincoln I-180

The soil behavior type index (I_c) in Figure 5.4 for Lincoln I-80 illustrates slight transitions in the soil behavior type index (I_c) from sand mixtures to silt mixtures as water is injected into the soil. The limited shift is due to the low clay content (10.5%), as the soil behavior dominant component is the coarse-grained material.

Peak and disturbed shear strength correlated from the revised CPT for Lincoln I-180 are shown in Figure 5.5. The in-situ peak shear strength S_u from the tip resistance was 30 psi on average from 5 to 8 ft, while the sleeve friction, which is related to the disturbed shear strength $S_{u(d)}$, at the same depth range was 20 psi on average. Both S_u and $S_{u(d)}$ reduced after water injection to approximately 20 and 15 psi, respectively, as shown in Figure 5.5. The reduction in Both S_u and $S_{u(d)}$ implies that the intact soil strength S_u was reduced to the fully softened shear due to the moisture variation, since it is close to the $S_{u(d)}$.

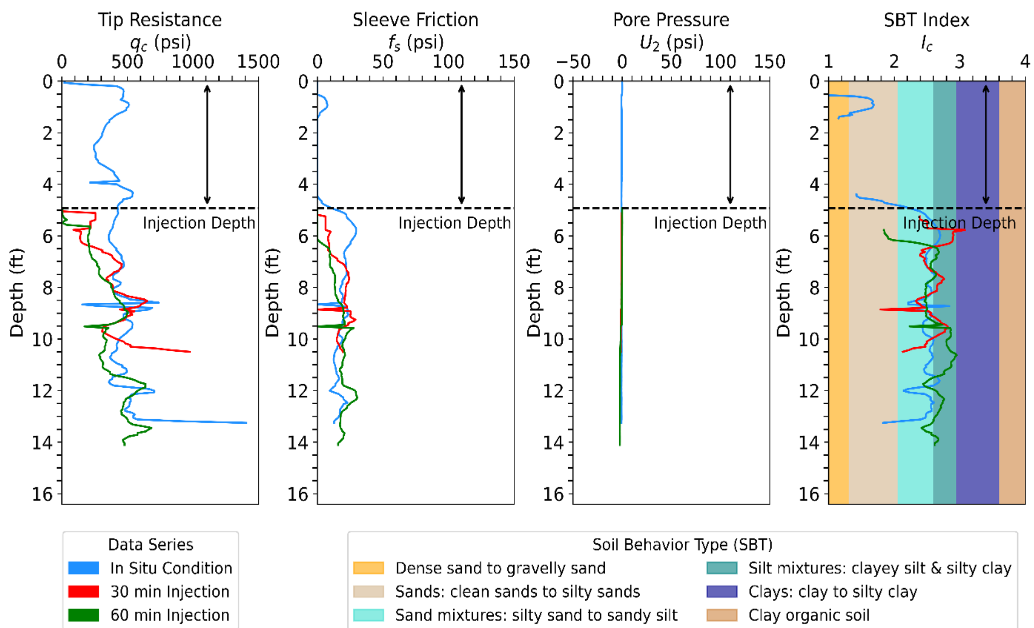

Figure 5.4 Lincoln I-180 SBT

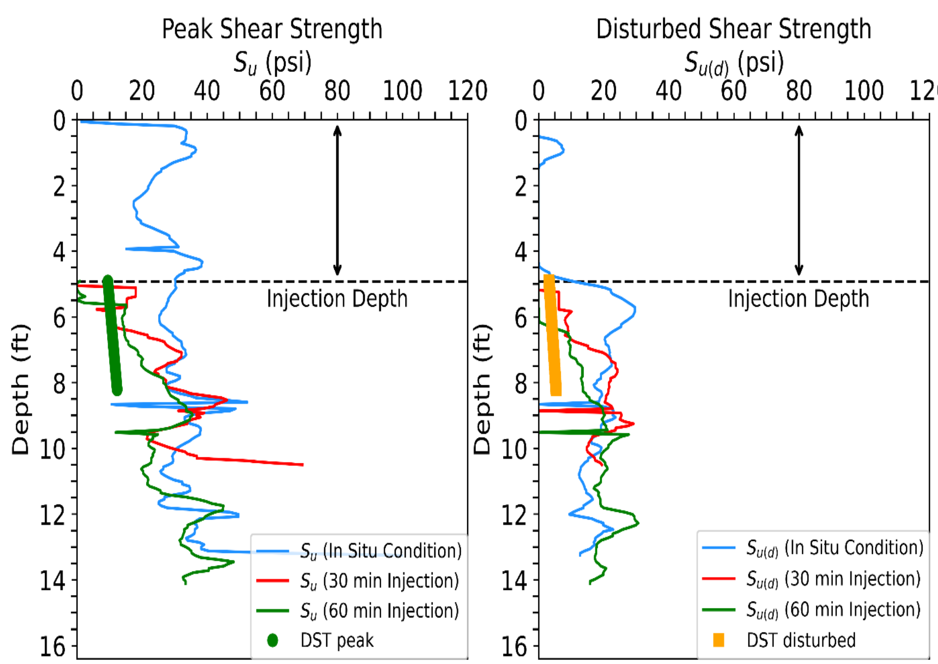

Figure 5.5 Lincoln I-180 Peak and Disturbed Shear Strength form CPT and DST

5.1.3 Lincoln Hwy-2

Figure 5.6 illustrates slight transitions in the soil behavior type index (I_c) at the Lincoln Hwy-2 location without changing the category as water is injected into the soil. The behavior of soil at Lincoln Hwy-2 was not anticipated and needs more research.

S_u and $S_{u(d)}$ results presented in Figure 5.7 for Lincoln Hwy-2 demonstrate a reduction in shear strength after the soil experienced moisture variation based on the laboratory direct shear test. In contrast, the revised CPT in that location resulted in increased shear strength after injecting water. More research is needed to figure out the reason for such behavior.

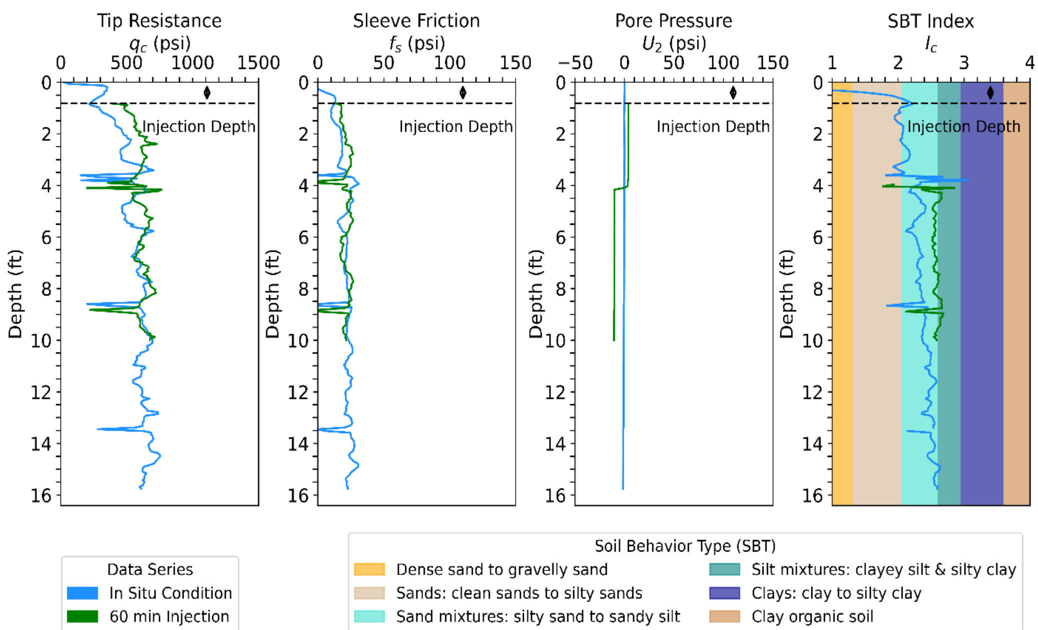

Figure 5.6 Lincoln Hwy-2 SBT

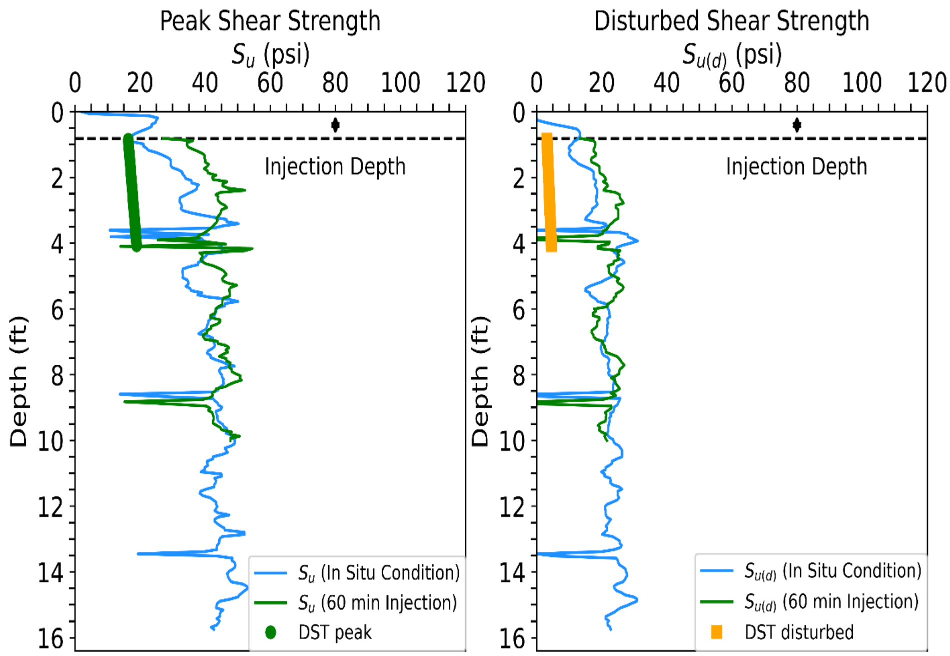

Figure 5.7 Lincoln Hwy-2 Peak and Disturbed Shear Strength form CPT and DST

5.1.4 Verdigre South

Figure 5.8 illustrates slight transitions in the soil behavior type index (I_c) at the Verdigre South slope from sand mixtures to sands as water is injected into the soil. The shift could be related to a sand layer or to inhomogeneity of soils, since the soil contains a clay content of 19.3% and is anticipated to shift towards the fine-grained material.

The peak and disturbed shear strength (S_u and $S_{u(d)}$) results presented in Figure 5.9 for Verdigre South demonstrate a substantial reduction in shear strength immediately below the injection depth after the soil experienced moisture variation based on the revised CPT. The peak and disturbed strength correlated from tip resistance and sleeve friction, respectively, align with the shear strength obtained from the direct shear test in the laboratory.

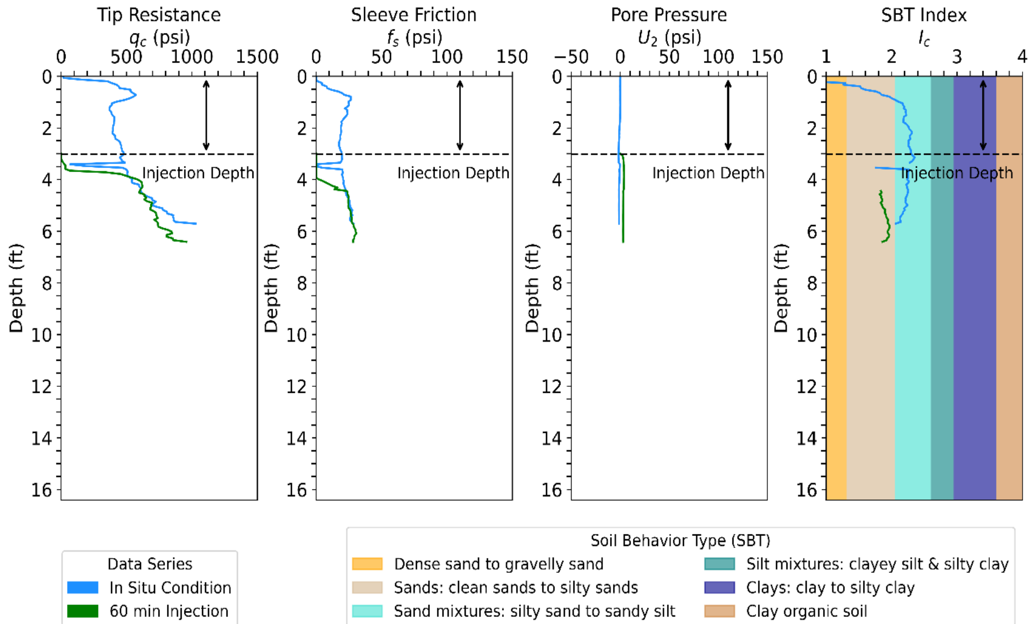

Figure 5.8 Verdigre South SBT

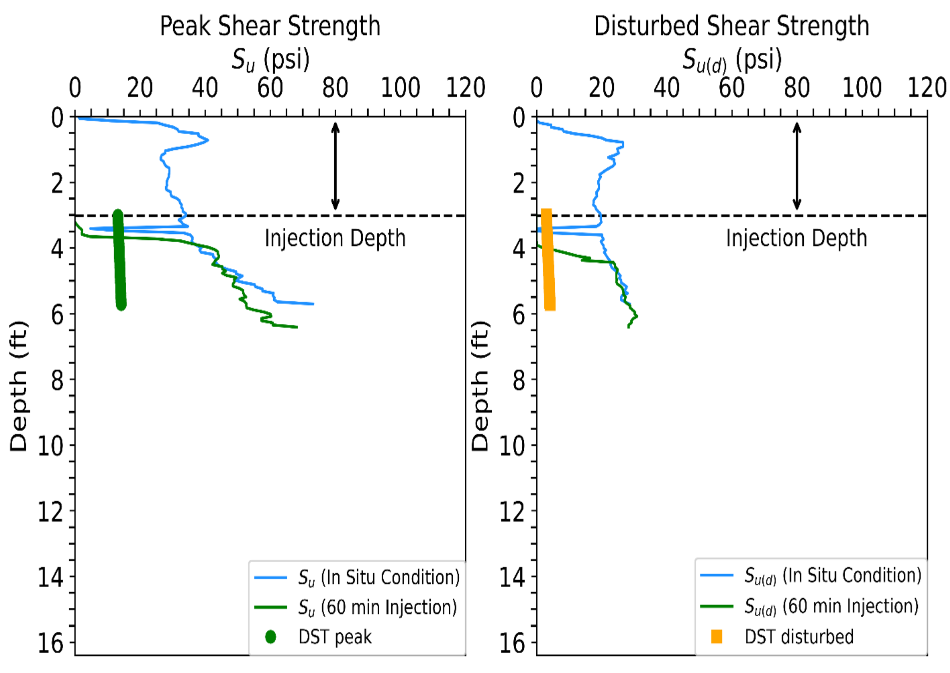

Figure 5.9 Verdigre South Peak and Disturbed Shear Strength form CPT and DST

5.1.5 Verdigre East

Figure 5.10 illustrates consistency in the soil behavior type index (I_c) at Verdigre East without changing the category as water is injected into the soil. The behavior of soil at Verdigre East may be due to the soil having been already wet and disturbed, as neither the tip resistance or the sleeve friction showed a significant reduction after water injection beyond 0.5 ft below the injection point.

The peak and disturbed shear strength results presented in Figure 5.11 for Verdigre East demonstrate a substantial reduction in shear strength in the soil immediately below the injection depth after experiencing moisture variation based on revised CPT. The peak and disturbed strength correlated from tip resistance and sleeve friction, respectively, align with the shear strength obtained from the direct shear test in the laboratory.

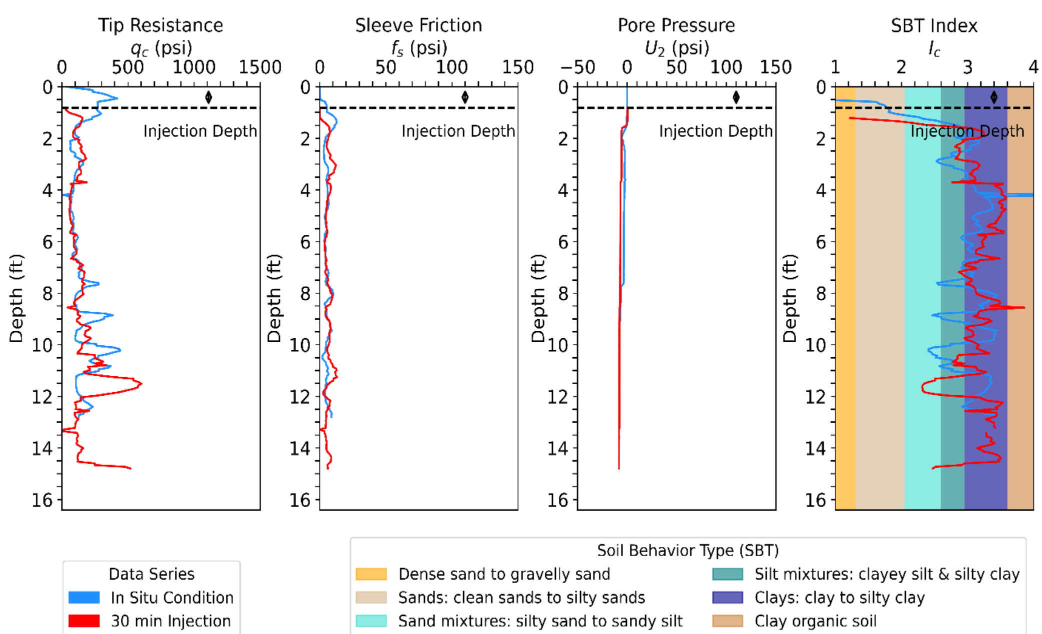

Figure 5.10 Verdigre East SBT

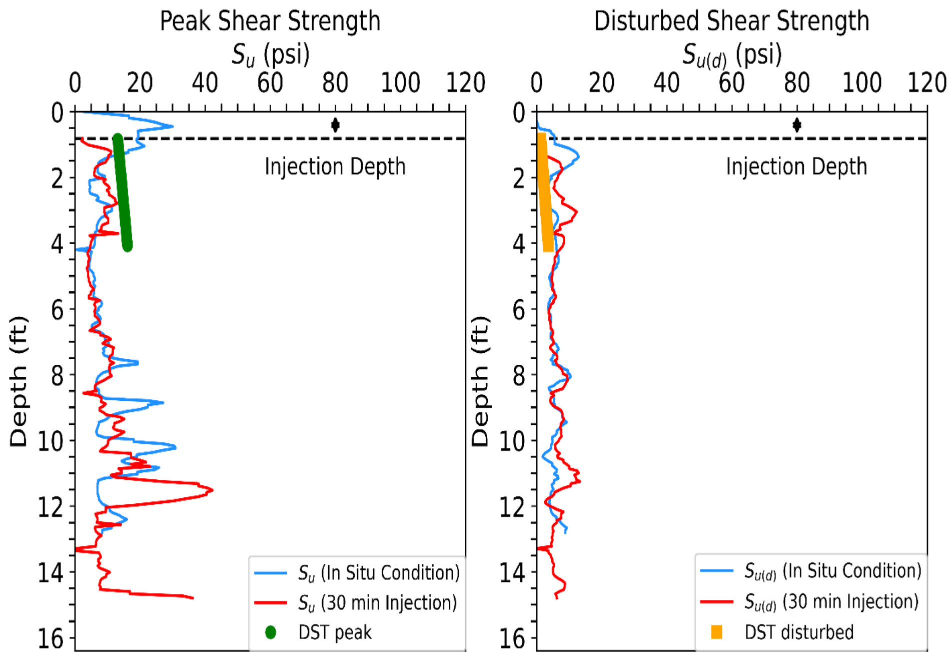

Figure 5.11 Verdigre East Peak and Disturbed Shear Strength form CPT and DST

5.1.6 Bristow

The soil behavior type index (I_c) illustrated in Figure 5.13 for the UNL East Campus site transitions from coarser-grained to finer-grained classifications as water is injected into the soil at depths from 1 to 3.5 ft.

Peak and disturbed shear strength correlated from the revised CPT for the Bristow location are shown in Figure 5.14. The in-situ peak shear strength S_u from the tip resistance was 22 psi on average from 1 to 3 ft, while the sleeve friction, which is related to the disturbed shear strength $S_{u(d)}$, at the same depth range was 15 psi on average. Both S_u and $S_{u(d)}$ were reduced after water injection to approximately the same value (about 10 psi) as shown in Figure 5.14. The reduction in Both S_u and $S_{u(d)}$ to the same value implies that the intact soil strength S_u was reduced to the fully softened shear due to the moisture variation, since it is close to the $S_{u(d)}$, and matches with the disturbed strength measured from the direct shear test after 48 hours submersion.

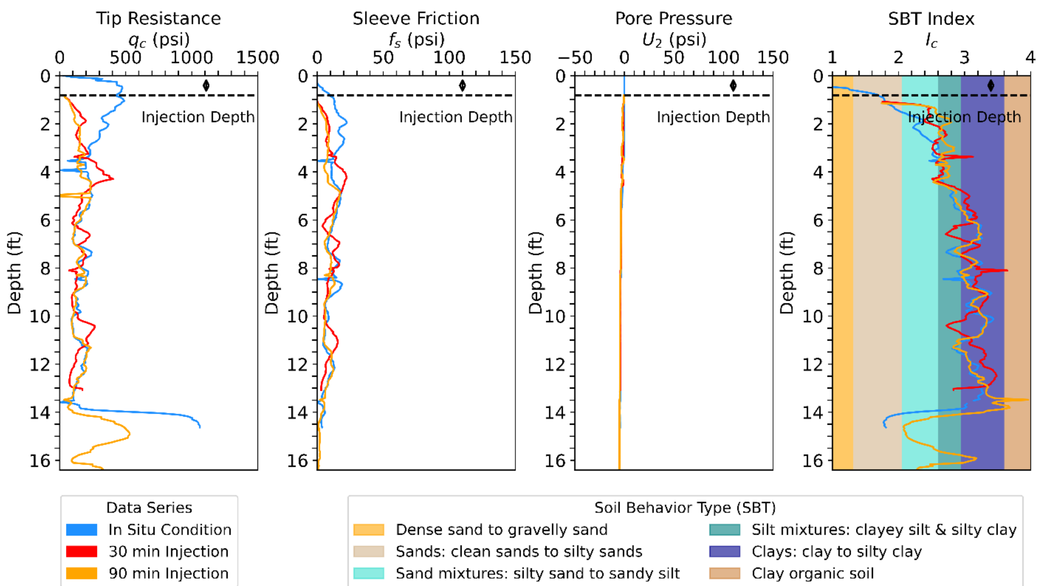

Figure 5.12 Bristow SBT

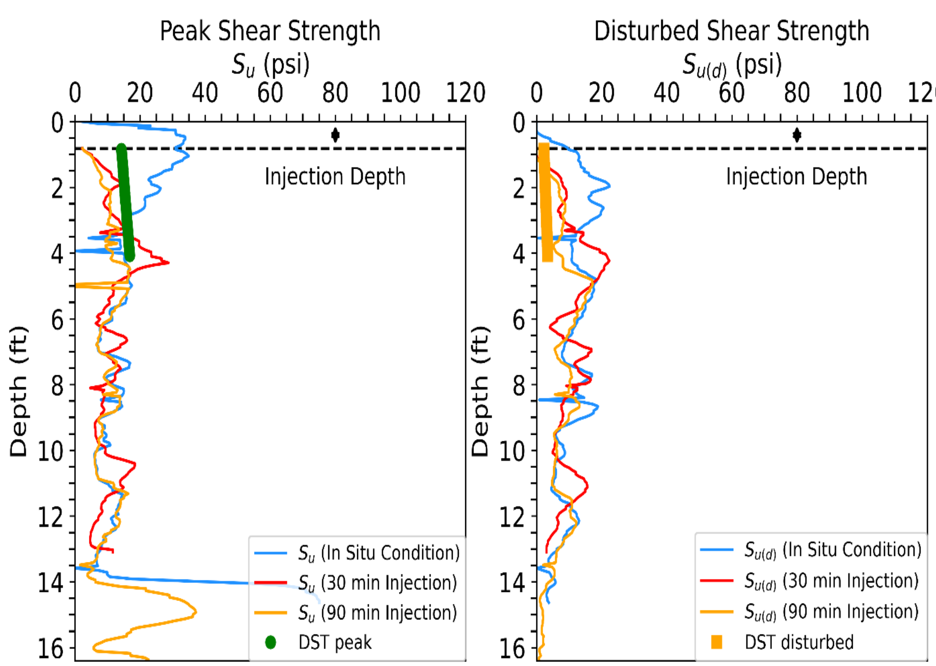

Figure 5.13 Bristow Peak and Disturbed Shear Strength form CPT and DST

5.1.7 US-75

Figure 5.14 illustrates consistency in the soil behavior type index (I_c) at the US-75 location without changing the category as water is injected into the soil. The behavior of soil at US-75 may be due to the high permeability of the top sand layer up to 2 ft. Moreover, the low clay content of the US-75 sample (9%) reduced the soil sensitivity to moisture variation.

The peak and disturbed shear strength results presented in Figure 5.15 for US-75 demonstrate no reduction in the shear strength of the soil after experiencing moisture variation based on revised CPT. The peak and disturbed strength correlated from tip resistance and sleeve friction, respectively, and the shear strength obtained from the direct shear test in the laboratory show no reduction.

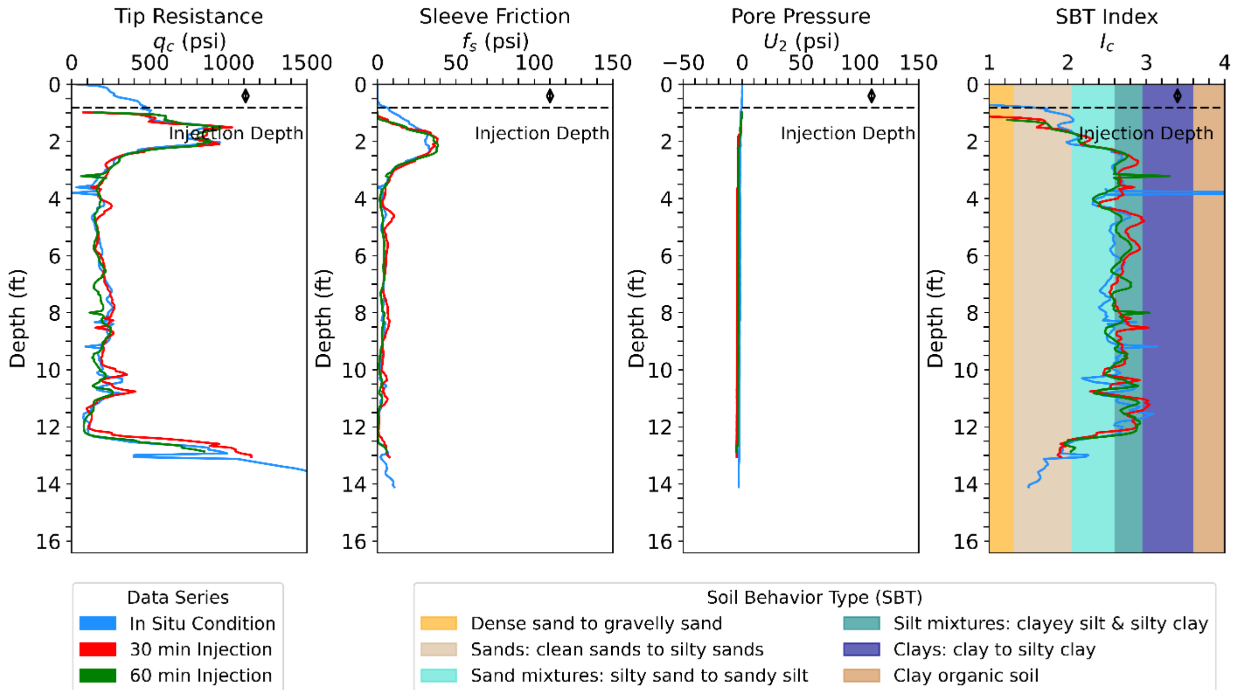

Figure 5.14 US-75 SBT

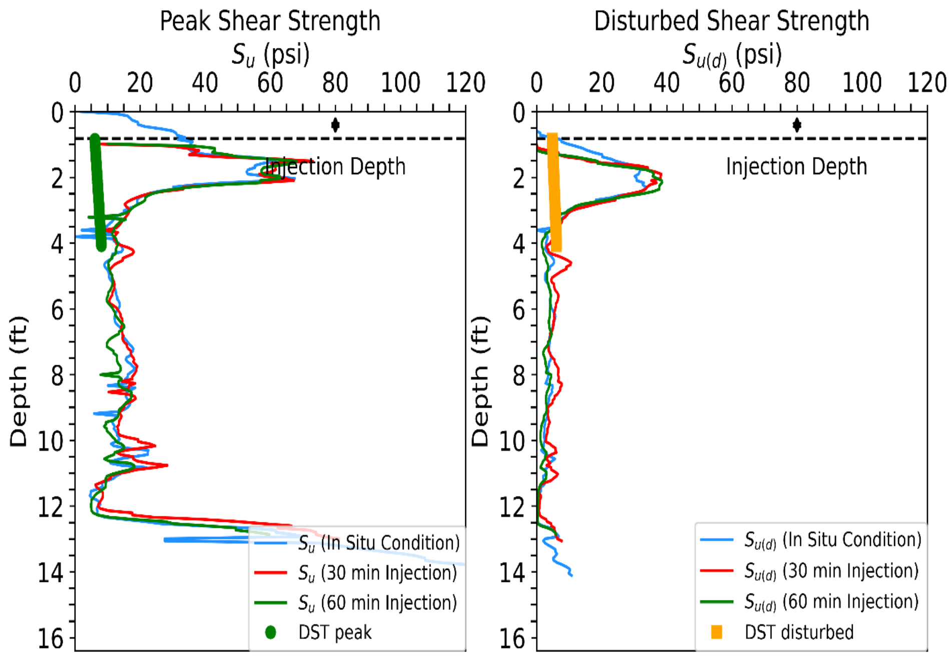

Figure 5.15 US-75 Peak and Disturbed Shear Strength form CPT and DST

The high CPT correlated S_u compared to S_u measured from the direct shear test in the lab is related to the sample quality. As the Shelby tube samples may have been disturbed. The disturbance in the sample results in lower shear strength compared to the shear strength based on the CPT conducted in the field (Lunne et al., 2006).

5.1.8 Waverly

The soil behavior type index (I_c) illustrated in Figure 5.16 for the Waverly slope transitions from coarser-grained to finer-grained classifications as water is injected into the soil. The top layer at Waverly shows a coarse-grained material based on the high tip resistance at 1-ft depth (1000 psi), below 1 ft from the surface the soil transitions to sand mixtures, which also shifts to the silt mixture category after injecting water and reflects the revised CPT ability to induce and measure softening through controlled water injection.

Peak and disturbed shear strength correlated from the revised CPT for Waverly are shown in Figure 5.17. The in-situ peak shear strength S_u from the tip resistance was 25 psi on average from 1 to 4 ft, while the sleeve friction, which is related to the disturbed shear strength $S_{u(d)}$, at the same depth range was 20 psi on average. Both S_u and $S_{u(d)}$ were reduced after water injection for 30 minutes to approximately 20 and 10 psi, respectively, as shown in Figure 5.17. The reduction in Both S_u and $S_{u(d)}$ to the same value (10 psi) after 60 minutes of injection implies that the intact soil strength (S_u) was reduced to the fully softened shear strength due to the moisture variation, since it is close to the $S_{u(d)}$.

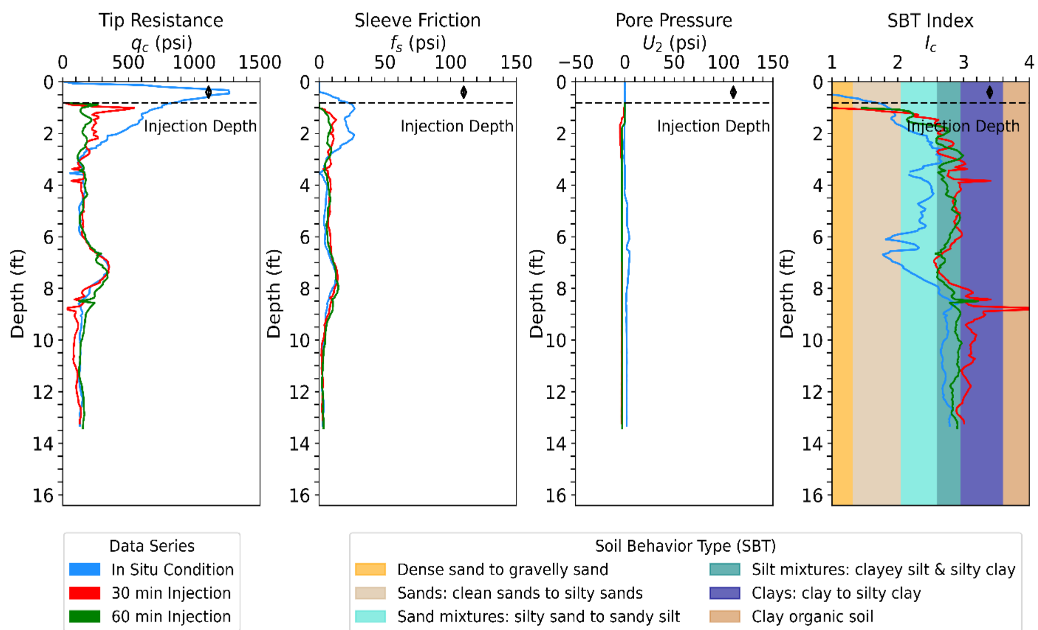

Figure 5.16 Waverly SBT

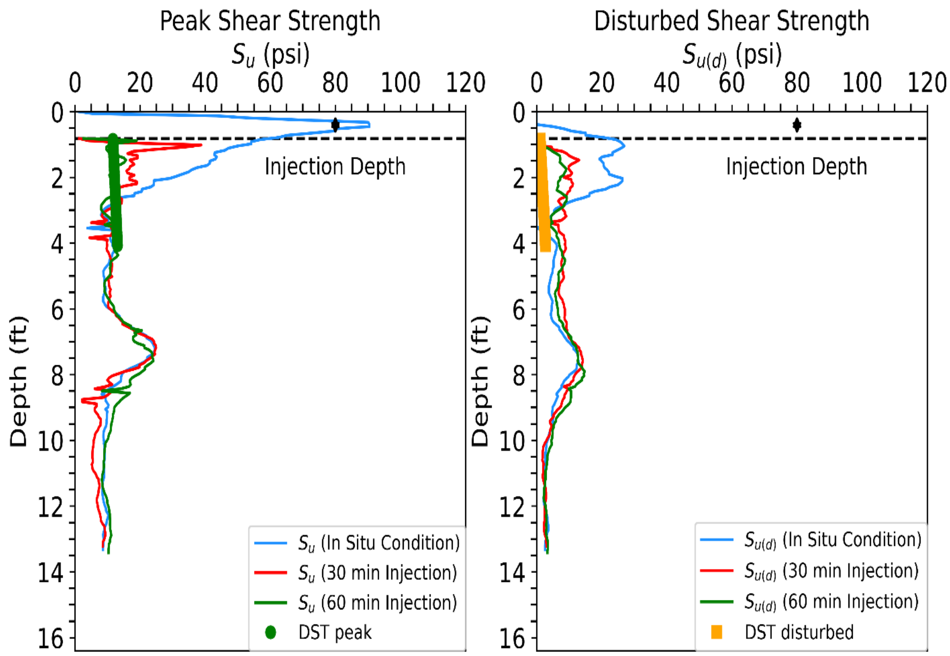

Figure 5.17 Waverly Peak and Disturbed Shear Strength form CPT and DST

5.1.9 Greenwood

The soil behavior type index (I_c) illustrated in Figure 5.18 for Greenwood does not change categories as water is injected into the soil. The behavior of soil at Greenwood may be due to the low permeability as the clay content is high (25%). Additionally, the clay mineralogy could be the reason behind the Greenwood soil's low sensitivity to moisture variation.

The peak and disturbed shear strength results presented in Figure 5.19 for Greenwood demonstrate no reduction in shear strength in the soil after experiencing moisture variation based on revised CPT. The peak and disturbed strength correlated from tip resistance and sleeve friction, respectively. However, shear strength obtained from the direct shear test in the laboratory showed reduction, which can be related to slow swelling in the field compared to the laboratory (Chandler, 1984a)

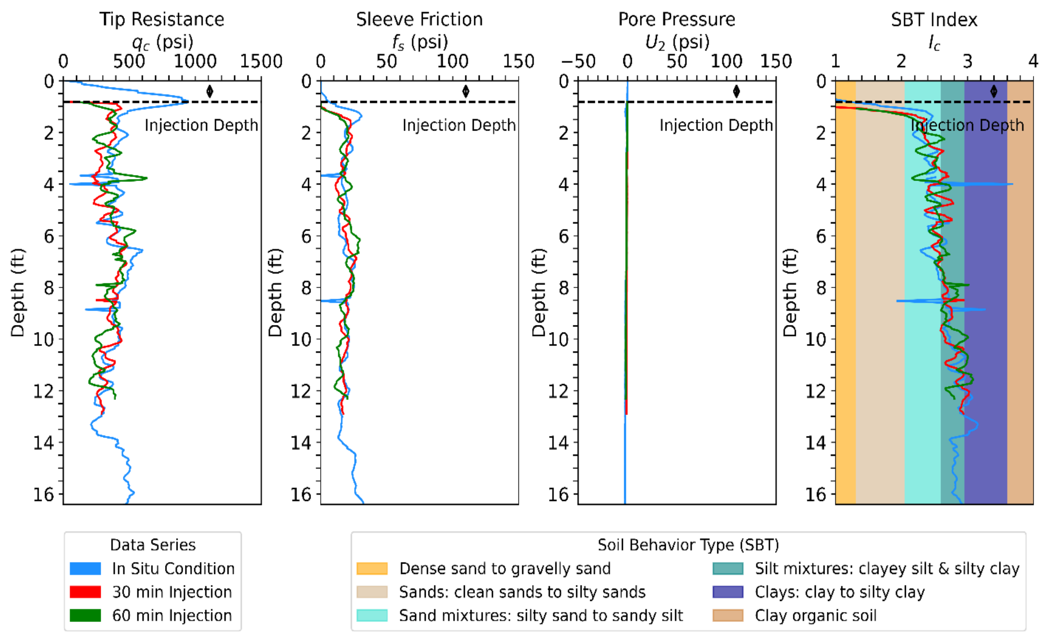

Figure 5.18 Greenwood SBT

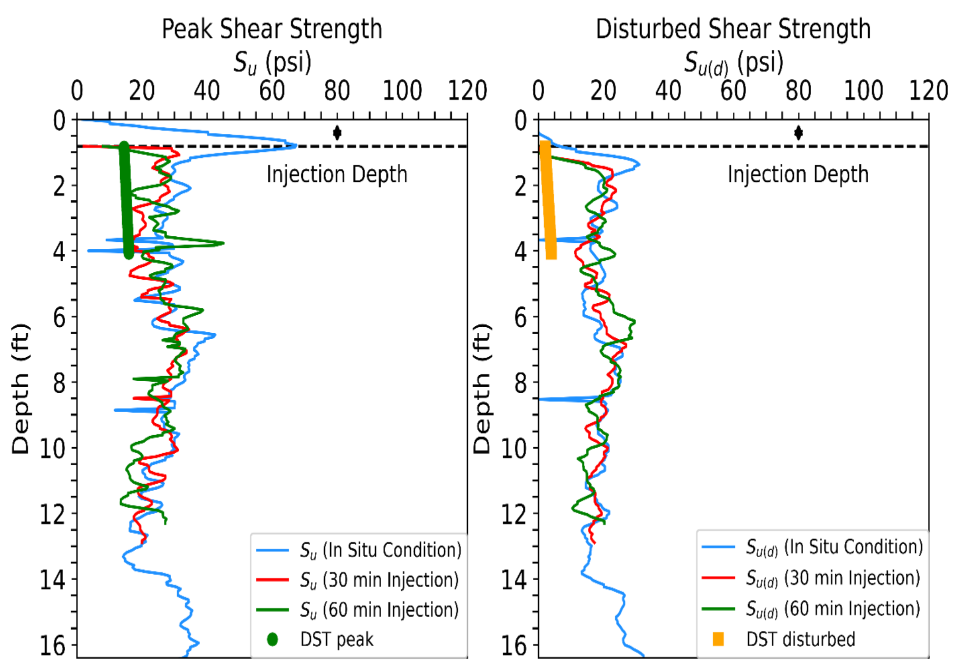

Figure 5.19 Greenwood Peak and Disturbed Shear Strength form CPT and DST

5.1.10 Norfolk, Hwy-275

The soil behavior type index (I_c) Figure 5.20 for the Norfolk location is illustrated in Figure 5.20. In the figure, I_c transitions from coarser-grained to finer-grained classifications as water is injected into the soil. The top layer of Waverly shows a coarse-grained material based on the high tip resistance at a 4-ft depth (550 psi). Below 4 ft from the surface the soil transitions to sand mixtures. The soils from 0 to 4 ft depth gradually shift toward the fine-grained material as the water injection duration increases, fully shifting to the silt mixture category after injecting water for 60 minutes.

Peak and disturbed shear strength correlated from the revised CPT for Norfolk are shown in Figure 5.21. The in-situ peak shear strength (S_u) from the tip resistance was 40 psi on average from 1 to 3.5 ft, while the sleeve friction, which is related to the disturbed shear strength $S_{u(d)}$, at the same depth range was 5 psi on average. S_u reduced after water injection for 30 and 60 minutes to approximately 30 and 15 psi, respectively, while $S_{u(d)}$ remained the same at the same depth range, as shown in Figure 5.21. The reduction in S_u to 10 psi after 60 minutes of injection implies that the intact soil strength S_u was reduced to the fully softened shear strength due to the moisture variation, since it is close to the $S_{u(d)}$.

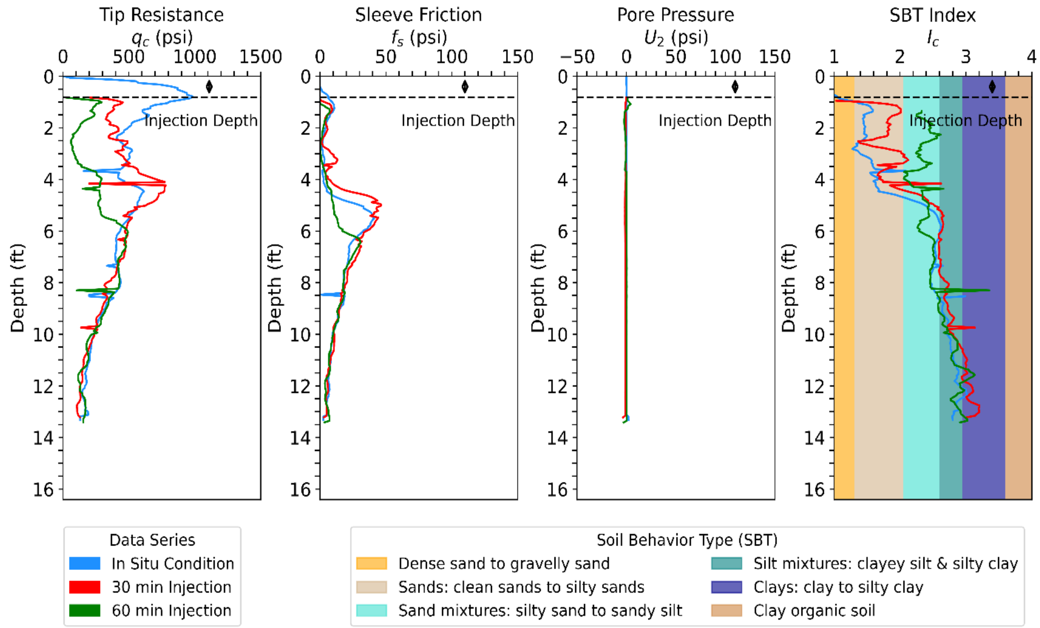

Figure 5.20 Norfolk Hwy-275 SBT

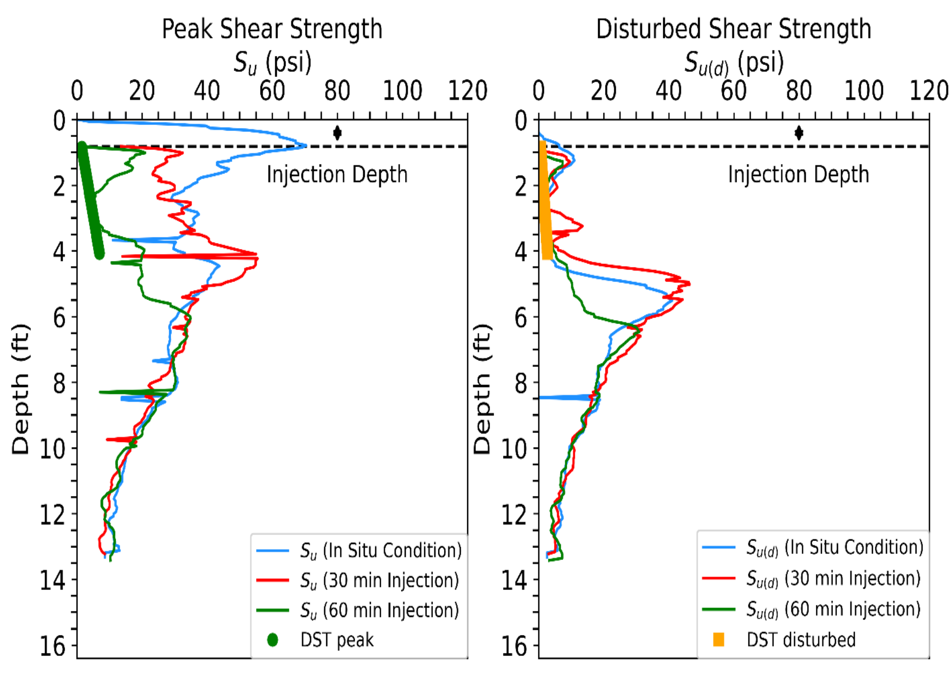

Figure 5.21 Norfolk Hwy-275 Peak and Disturbed Shear Strength form CPT and DST

5.1.11 Santee Spur

Analysis of the soil behavior type index (I_c) for the Santee Spur site indicates that the upper 3 ft of the profile is predominantly composed of a coarser-grained sandy material, as evidenced by the high tip resistance and low sleeve friction presented in Figure 5.22. Below this depth, the soil profile remains relatively consistent. At approximately 6 ft in depth, a significant reduction in tip resistance is observed when water is injected from 3.28 ft (red line), whereas water injection from 6.56 ft (green line) results in a less pronounced reduction. This variation in response to moisture variation at the same depth may be attributed to soil anisotropy. Moreover, the revised CPT effectively captured these differences in softening, with the I_c index reflecting a higher transition toward finer-grained behavior following a 60-minute injection period from 3.28 ft (red line).

Peak and disturbed shear strength correlated from the revised CPT for Santee Spur are shown in Figure 5.23. The in-situ peak shear strength S_u from the tip resistance was 45 psi on average from 6 to 9 ft, while the sleeve friction which is related to the disturbed shear strength $S_{u(d)}$ at the same depth range was 20 psi on average. S_u reduced after water injection for 60 minutes from 3.28 ft to approximately 30 psi, while $S_{u(d)}$ reduced to 15 for the same water injection duration, as shown in Figure 5.23. The reduction in S_u to the value (30 psi) after 60 minutes of injection implies that the intact soil strength S_u was reduced to the fully softened shear strength due to the moisture variation, since it is close to the $S_{u(d)}$.

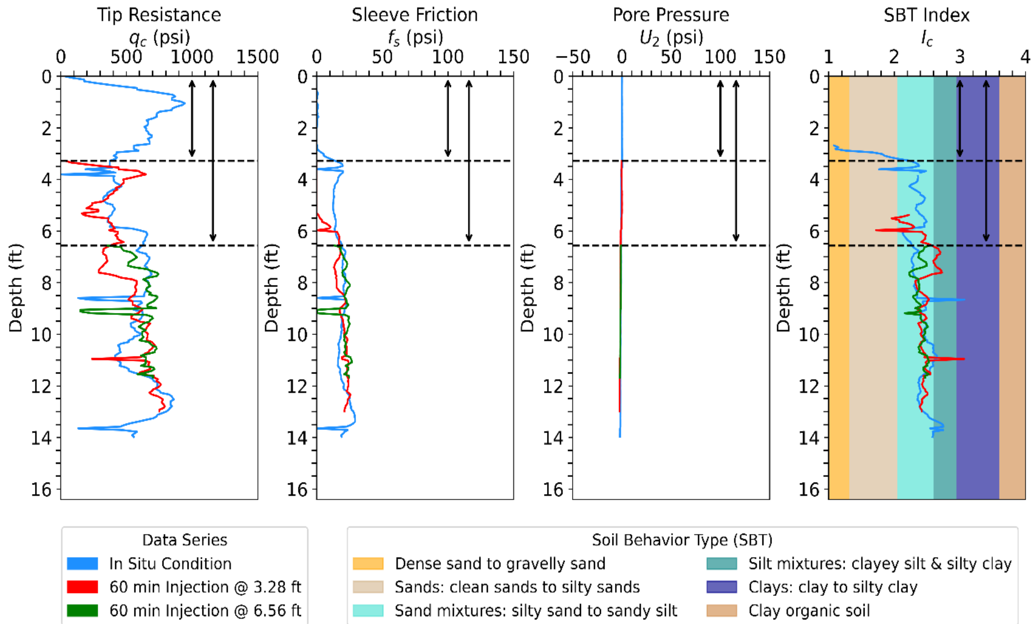

Figure 5.22 Santee Spur SBT

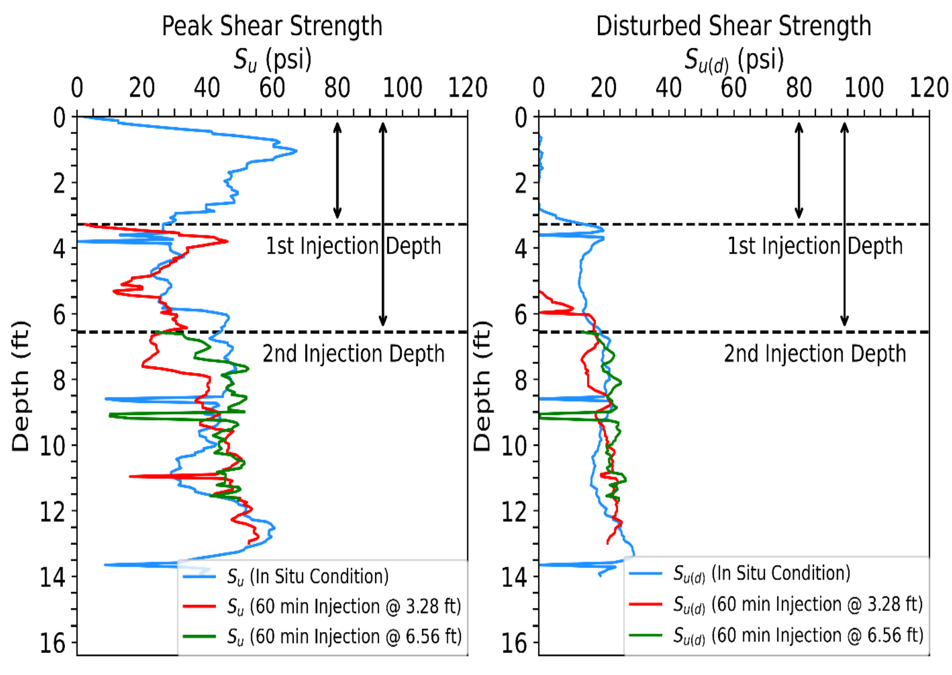

Figure 5.23 Santee Spur Peak and Disturbed Shear Strength form CPT

Table 5.1 presents a comparative analysis of the peak shear strength (S_u) and disturbed shear strength ($S_{u(d)}$) values derived from the cone penetration test (CPT) correlations, following the methodology proposed by Robertsons (2009). Specifically, the table details the shear strength measurements under two distinct conditions: the in-situ condition, representing the field condition prior to water injection, and the post-injection condition hereafter referred to as "soft", corresponding to the maximum observed reduction in shear strength following water injection.

Table 5.1 Peak and Disturbed Shear Strength Reduction and Soil Sensitivity

Location	S_u (in-situ) (psi)	$S_{u(d)}$ (in-situ) (psi)	S_u (soft) (psi)	$S_{u(d)}$ (soft) (psi)	S_t (in-situ)	S_t (soft)
East Campus	41	20	15	15	2.05	1
Verdigre South	43	20	5	3	2.15	1.67
Verdigre East	9	9	6.5	6.5	1	1
Bristow	22	15	10	10	1.5	1
Lincoln I-180	30	20	20	15	1.5	1.3
Lincoln Hwy-2	32.49	NR*	NR*	20.32	NA*	NA*
Greenwood	30	20	30	20	1.5	1.5
US-75	30	30	30	30	1	1
Waverly	28	20	10	7.5	1.4	1.3
Norfolk Hwy-275	40	5	15	5	8	3
Santee Spur	45	20	30	15	2.25	2

*NR: No reduction
*NA: Not applicable

The data presented in Table 5.1 highlight the ability of the revised CPT to capture the reduction in shear strength. For instance, locations such as East Campus and Verdigre South exhibit pronounced decreases in both S_u and $S_{u(d)}$ following water injection. At East Campus, S_u was reduced from 41 psi in-situ to 15 psi, while the sensitivity index reduced from 2.05 to 1.00. Similarly, Verdigre South shows a reduction in S_u from 43 psi to 5 psi and in $S_{u(d)}$ from 20 psi to 3 psi, accompanied by a decrease in S_t from 2.15 to 1.67. Moreover, Norfolk shows a reduction in S_u from 40 psi to 5 psi and in S_t from 8 psi to 3 psi.

Based on Skempton's (1970) definition of fully softened shear strength as the drained normally consolidated peak of clay, and the results presented in Figure 2.1, it can be concluded that soil will show lower sensitivity when in fully softened condition, since the soil is deformed and experienced shear strength loss. The lower sensitivity in soil after water injection indicates that the soil reached a fully softened condition, and the revised CPT effectively captured the shear strength reduction.

Chapter 6 Conclusion

The main objective of this study was to evaluate the fully softened shear strength of soils involved in previously reported slope failures in Nebraska. A new and innovative Cone Penetration Test (CPT) was developed to inject water into the soil at a specific depth using the Geoprobe DI HPT flow module. This modified test is referred to as the revised CPT in this report.

The purpose of the revised CPT is to compare soil behavior in its "in-situ" condition with behavior after water injection for varying durations (30, 60, and 90 minutes). This moisture variation acts as a softening mechanism, and the high strain from the sleeve indicates that the soil around it is fully disturbed.

The revised CPT logs, which include both tip resistance (q_c) and sleeve friction (f_s), were analyzed under both "in-situ" and "wet" conditions. Additionally, soil samples were collected from each site to examine properties that might influence soil behavior, such as particle size distribution, clay size fraction, and plasticity index.

Furthermore, the shear strength of the collected samples was evaluated in the laboratory using a direct shear test, which was conducted under "field moisture conditions" and "laboratory wet conditions". In the latter case, the samples were submerged in water for 48 hours before testing to observe the effect of moisture variation on shear strength.

Finally, the soil behavior type (I_c) and the peak and disturbed shear strength of the soil, before and after water injection, were interpreted using the CPT data based on Robertson's (2009) approach. A comparison was made between the shear strength correlated with the revised CPT and that measured from the direct shear test. The findings of this research can be concluded in the following points:

1. Revised Cone Penetration Test (CPT)

- The revised Cone Penetration Test (CPT) was developed to artificially create a "wet" condition in the field, to evaluate the fully softened shear strength.
- The revised CPT procedure offered a reliable and efficient method to evaluate the soil behavior in the field within six hours.

2. Soil Index and Field Behavior

- Locations such as Verdigre South, Verdigre East, and Bristow, which exhibited high plasticity indices (PI) and high clay fractions (%CF), showed pronounced decreases in cone tip resistance (q_c) and sleeve friction (f_s). These observations imply susceptibility to moisture-induced softening, primarily attributed to the soil's ability to retain water and swell.
- Minimal or no softening was observed at sites like US-75 and Greenwood. At US-75, the low clay fraction of 9% combined with a moderate plasticity index (PI = 16%) restricts excess water retention, whereas the low hydraulic conductivity at Greenwood (23.8% clay fraction) prevented deep water infiltration by allowing injected water to escape at the borehole surface.
- At Norfolk and Waverly, with clay fractions of 11% and 12% respectively, a gradual reduction in (q_c) and (f_s) was observed over the injection period, indicating time-dependent softening despite moderate-to-high plasticity (PI ≈ 17.7% and 30.6%). In contrast, Bristow and UNL East Campus experienced relatively uniform reductions at multiple injection intervals, suggesting a different softening mechanism, likely linked to the expansive properties of fat clay prevalent at both sites.

3. <u>Shear Strength Based on Revised CPT and Direct Shear Test</u>

- Direct shear test results confirmed that shear strength was reduced after soil experienced increased moisture content, which confirms the observed reductions in CPT logs after water was injected into the soil and reflects the ability of the revised CPT to capture real-time reduction.
- The soil behavior type index (I_c) from CPT data interpretation based on Robertson's (2009), indicates a shift in soil behavior towards fine-grained materials after water injection, which reflects the revised CPT ability to capture the influence of water on strength, stiffness and compressibility of soil.
- Field observations at East Campus, Verdigre South, and Norfolk confirm the revised Cone Penetration Test (CPT) effectively captures moisture-induced softening. Notable reductions in peak shear strength (e.g., from 41 psi to 15 psi at East Campus, from 43 psi to 5 psi at Verdigre South, and from 40 psi to 5 psi at Norfolk) illustrate the extent to which water injection compromised soil resistance. Simultaneous reductions in disturbed shear strength (e.g., from 20 psi to 3 psi at Verdigre South) corroborated the reliability of the revised CPT as an in-situ diagnostic tool for identifying weakened soil zones under high moisture conditions.
- The observed lower sensitivity aligns with Skempton's (1970) concept of fully softened shear strength, where extensively disturbed and wet soil exhibits a drained, normally consolidated peak strength. Under these circumstances, further deformation does not produce significant additional shear strength loss, reflecting a diminished sensitivity index. Consequently, the revised CPT's ability to detect these concurrent changes in peak, disturbed shear strength, and sensitivity validates its usefulness for evaluating fully softened shear strength.

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Appendix A Soil profiles

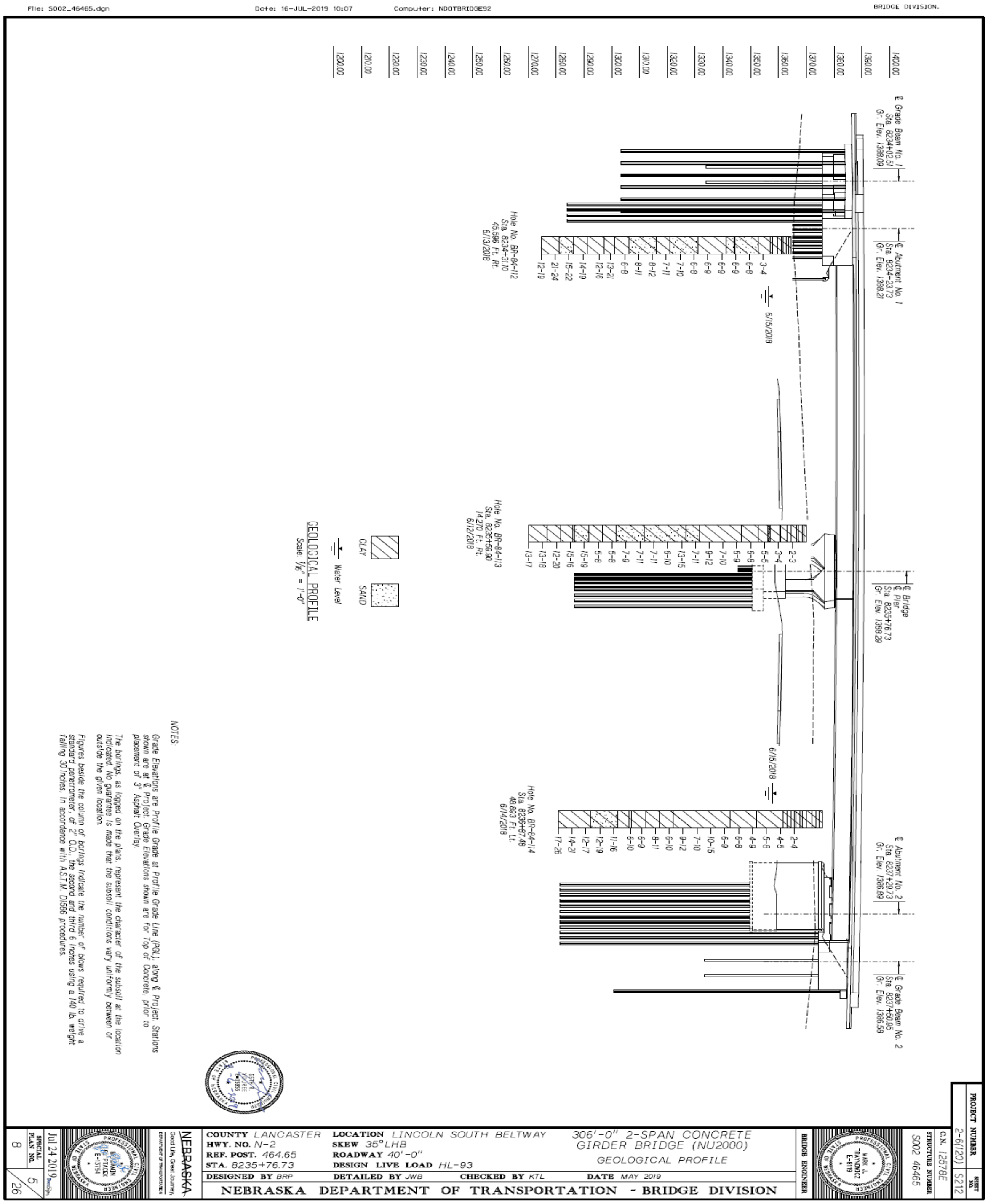


Figure A.1 Lincoln Hwy-2 Soil Profile

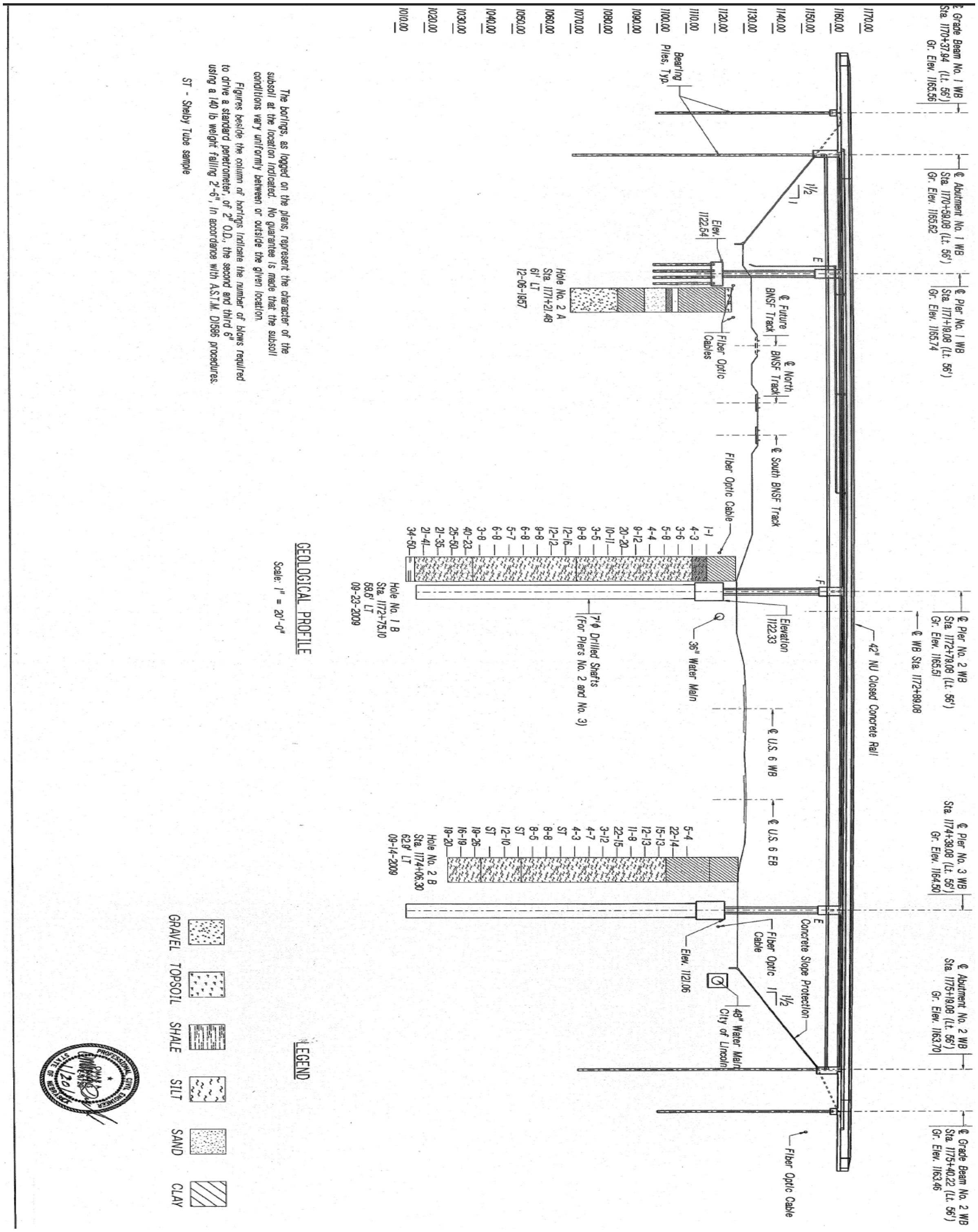


Figure A.2 Waverly Soil Profile

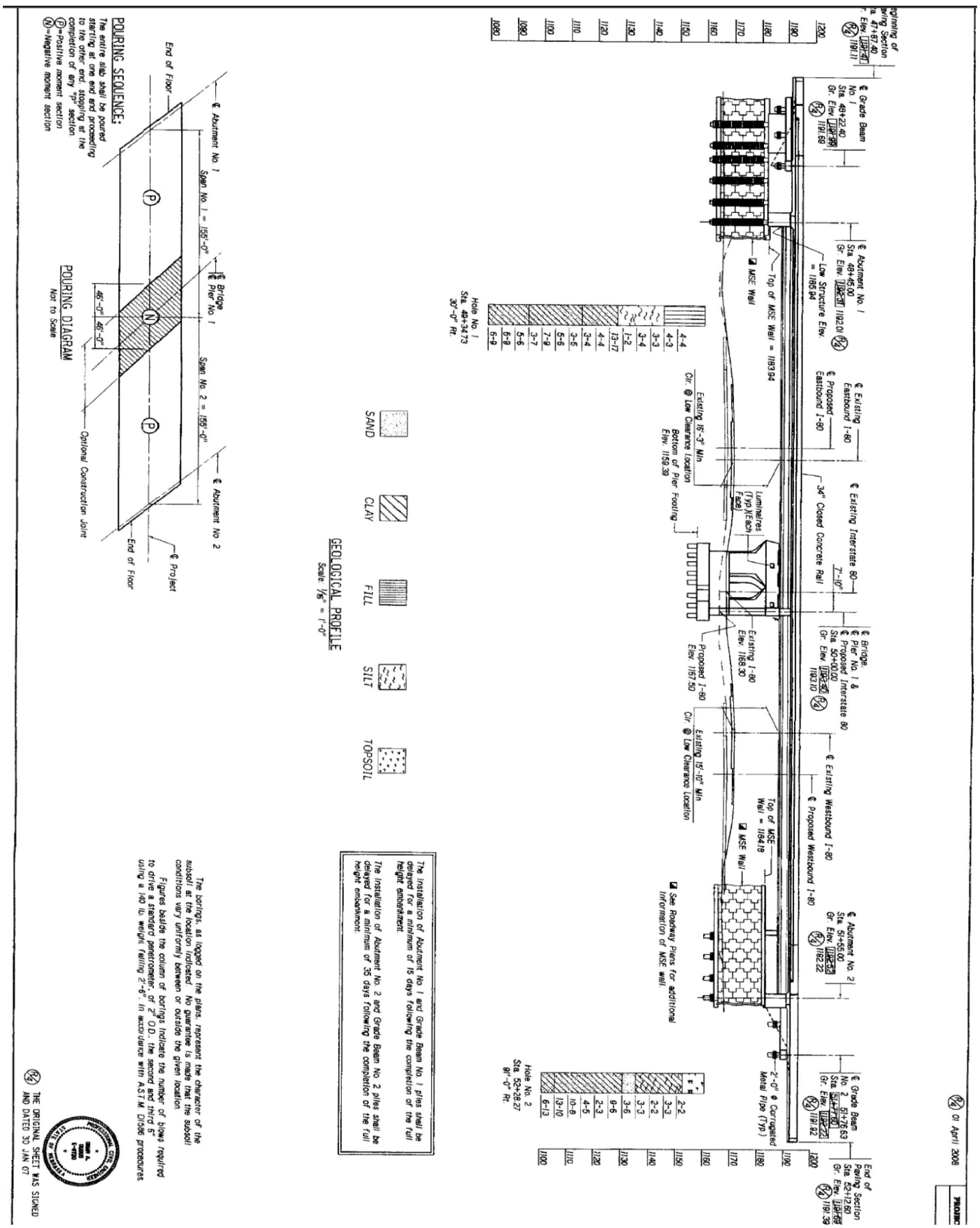


Figure A.3 Greenwood Soil Profile

NEBRASKA DEPARTMENT OF TRANSPORTATION	BOREHOLE REPORT NO. SM-180	Sheet 1 of 1

PROJECT NAME		STRUCTURE NUMBER / R. P.
N-57 - N-15 _ Norfolk - Wisner		S275 08860 / 88.60
PROJECT NUMBER	**CONTROL NUMBER**	**STATION**
275-6(1052)	32319A	179+90.81, 86.8' RT

■ Shelby Tube

APPROX. SURFACE ELEV. (ft):1500.9

ELEVATION (ft)	MATERIAL DESCRIPTION	DEPTH (ft)	SAMPLE TYPE NUMBER	CLASSIFICATION (USCS)	BLOWS/6" N-VALUE	UNC. STR. (psi)	MOISTURE (%)	DRY DENSITY (pcf)	LL/PI (%)	ADDITIONAL DATA/ REMARKS
1500	***TOPSOIL*** *Lean clay, dark gray - black, some organics, soft, moist (topsoil)* **4.0'**	0		CL						
1495	***PEORIA LOESS*** *Light brown - brown lean clay w/ some silty layers, soft, v. moist (peorian)*	5								
1490		10	U 1	CL/ML		5.35	26.7	88.8	41/20	TV=0.4 tsf
1485		15								
1480		20	U 2			5.39	29.5	91.8	43/21	TV=0.4 tsf
1475	**26.0'**	25								
1470	***PEORIA LOESS*** *Light brown to light gray silty clay, lean to med plastic, soft, v. moist to wet (peorian)*	30	U 3	CL/ML		5.42	34.2	86.8	43/19 55/29	TV=0.15 tsf
1465	**36.0'**	35								
	ALLUVIUM *Light gray - gray silty clay, tfs, soft, wet (alluv)* **40.0'**	40	U 4	CL/ML			28.4	95.2	38/15	TV=0.4 tsf

BASE OF BORING AT 40.0 FEET

WATER LEVEL OBSERVATIONS		Nebraska Department of Transportation Materials & Research Division Geotechnical Section	STARTED: 7/9/24	FINISHED:
WD	∇ Not Performed		DRILL CO.: NDOR	DRILL RIG: Geoprobe 3230DT
IAD	▼ 24.0 ft after 0 Hrs		DRILLER: Nikolas Glennie	LOGGED BY: Alex Silvey
AD	▼ Not Performed		METHOD: HOLLOW STEM AUGER	

Figure A.4 Norfolk Hwy-275 Soil Profile

Appendix B Direct Shear Results

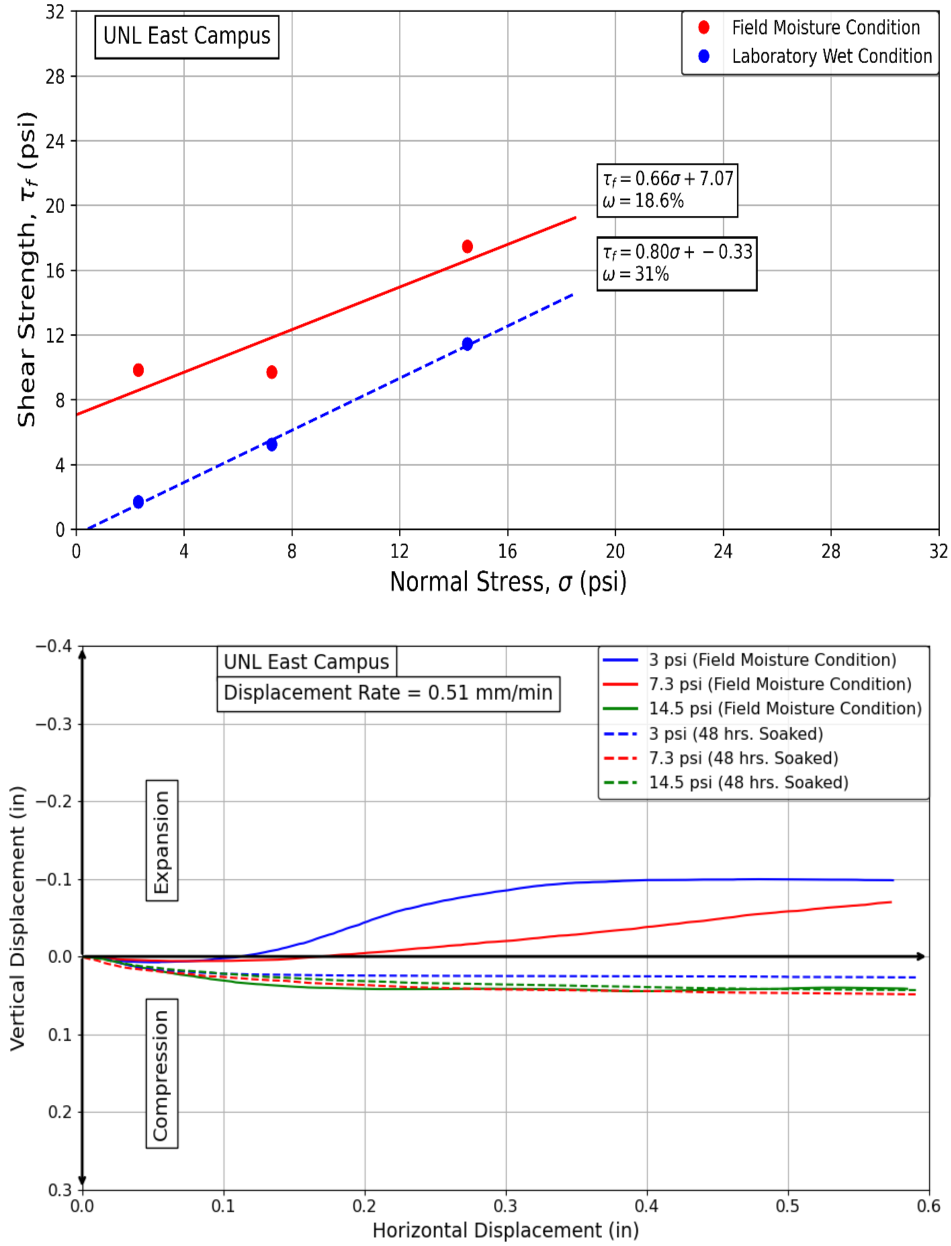


Figure B.1 UNL East Campus Shear Strength Parameters and Change in Height of Specimen against Shear Displacement

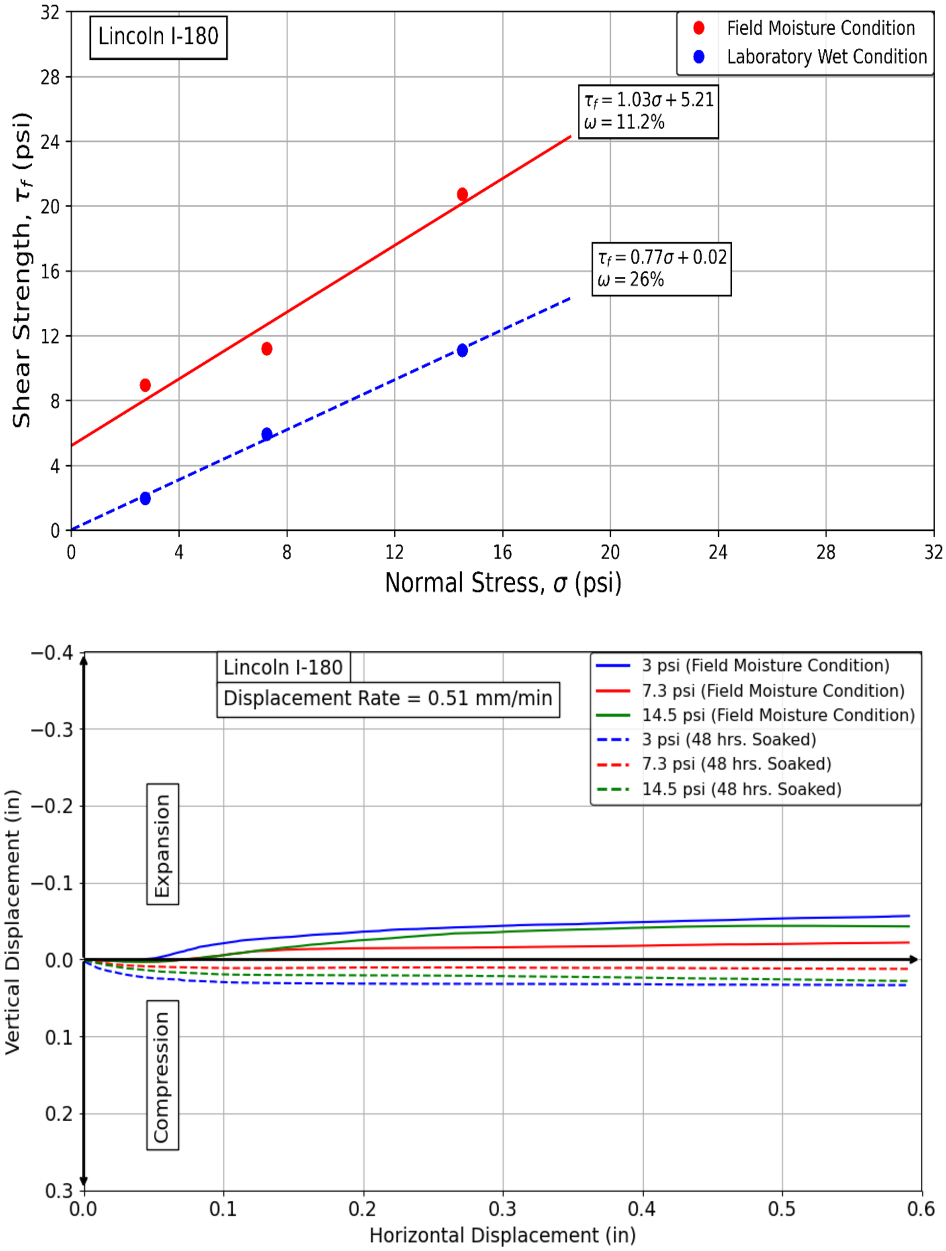


Figure B.2 Lincoln I-180 Shear Strength Parameters and Change in Height of Specimen against Shear Displacement

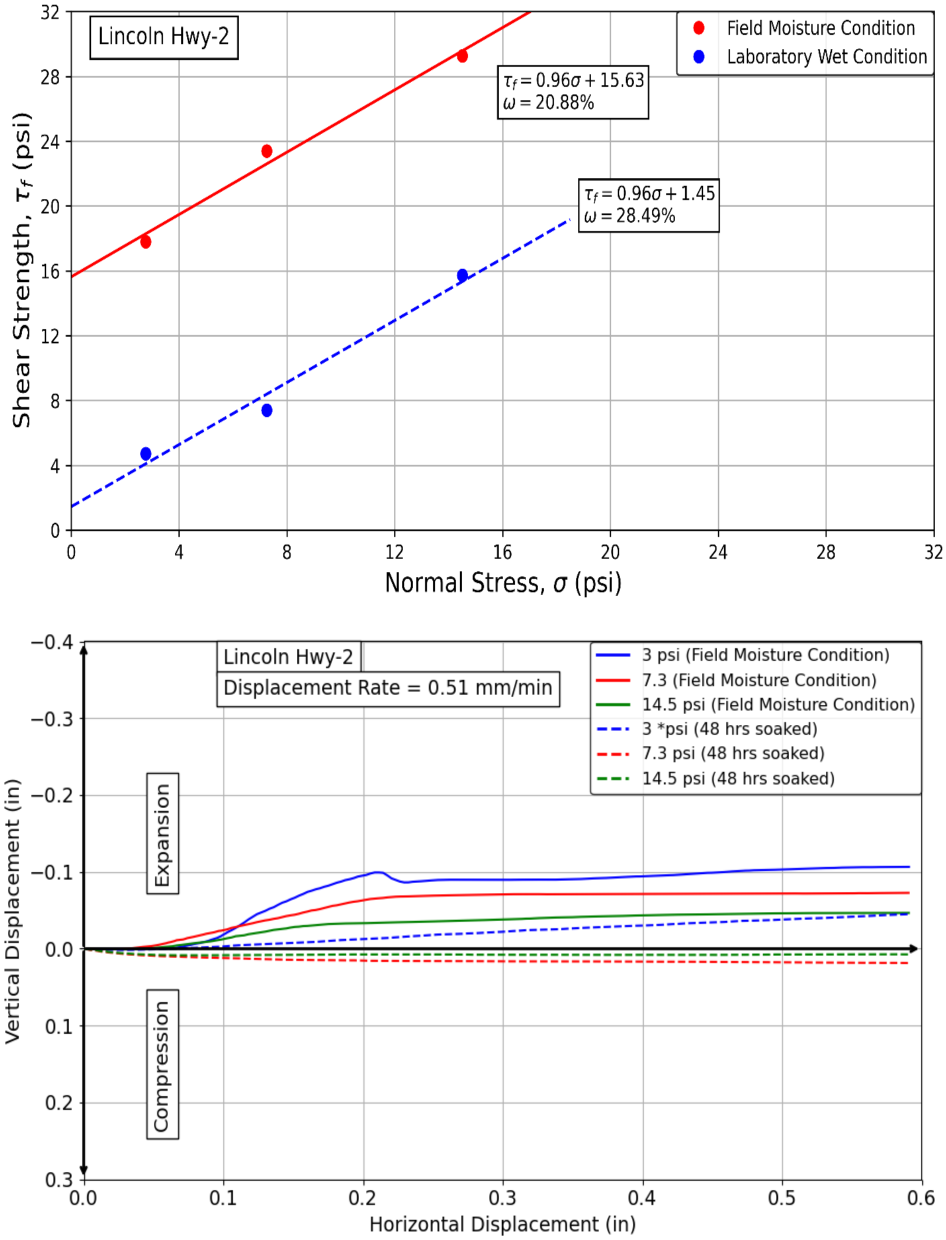
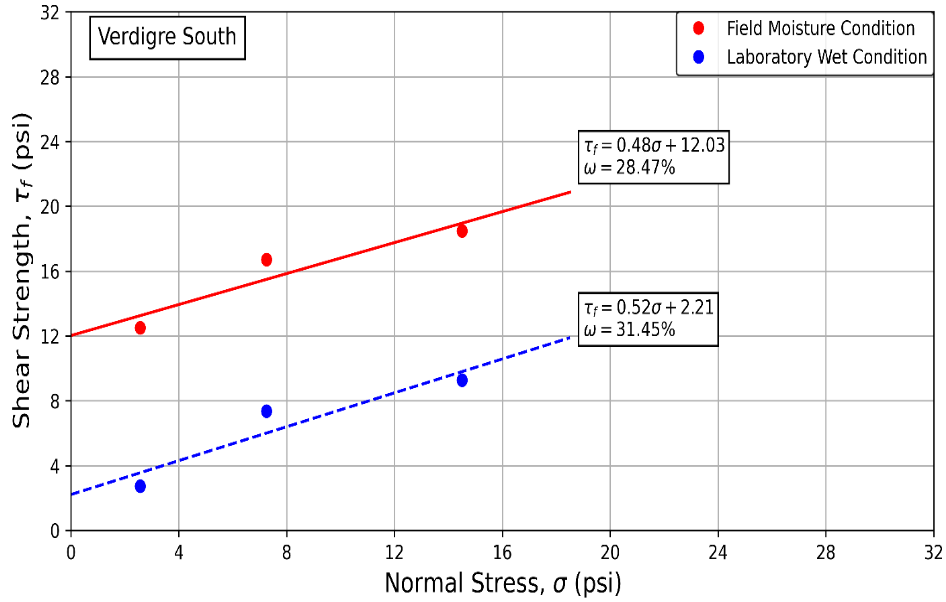


Figure B.3 Lincoln Hwy-2 Shear Strength Parameters and Change in Height of Specimen against Shear Displacement



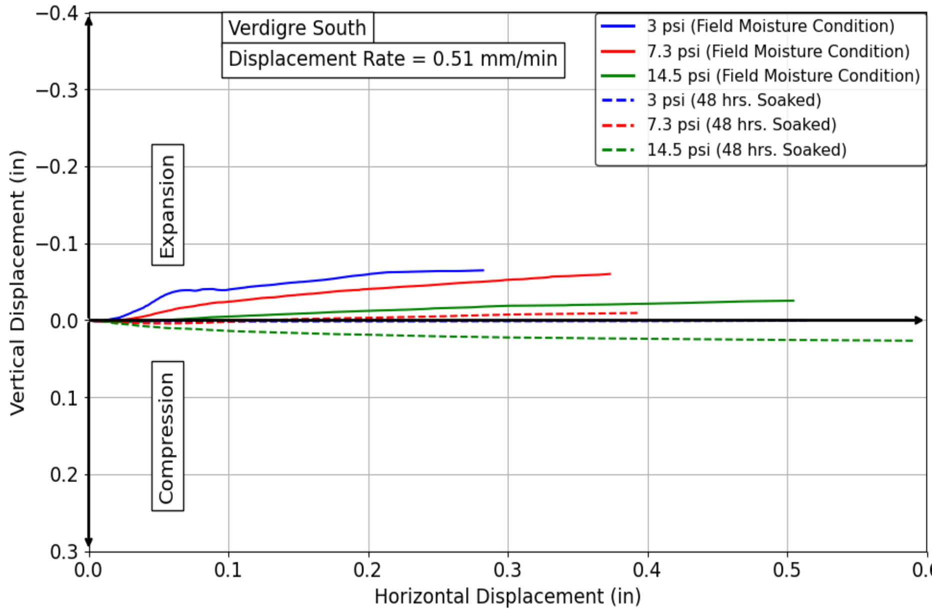
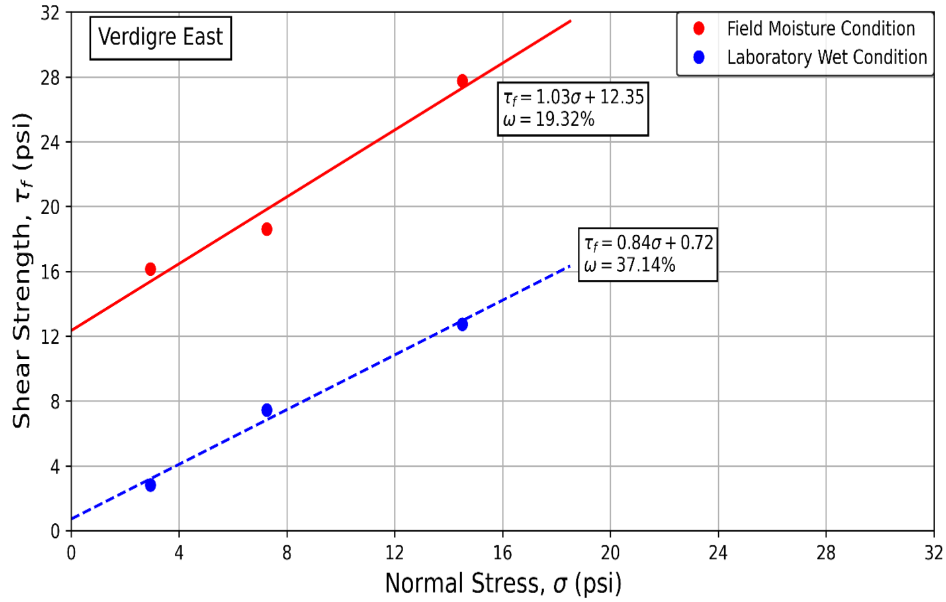


Figure B.4 Verdigre South Shear Strength Parameters and Change in Height of Specimen against Shear Displacement



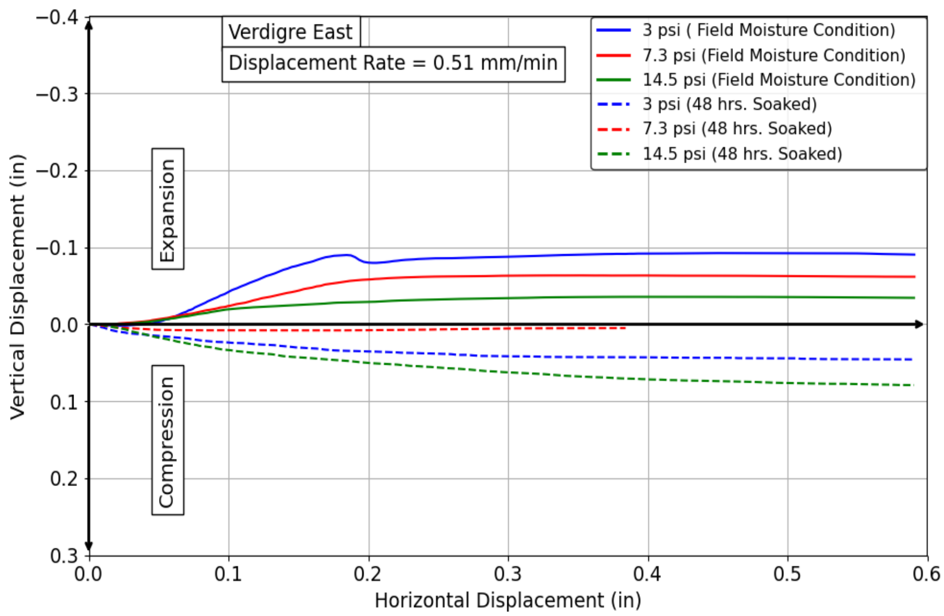


Figure B.5 Verdigre East Shear Strength Parameters and Change in Height of Specimen against Shear Displacement

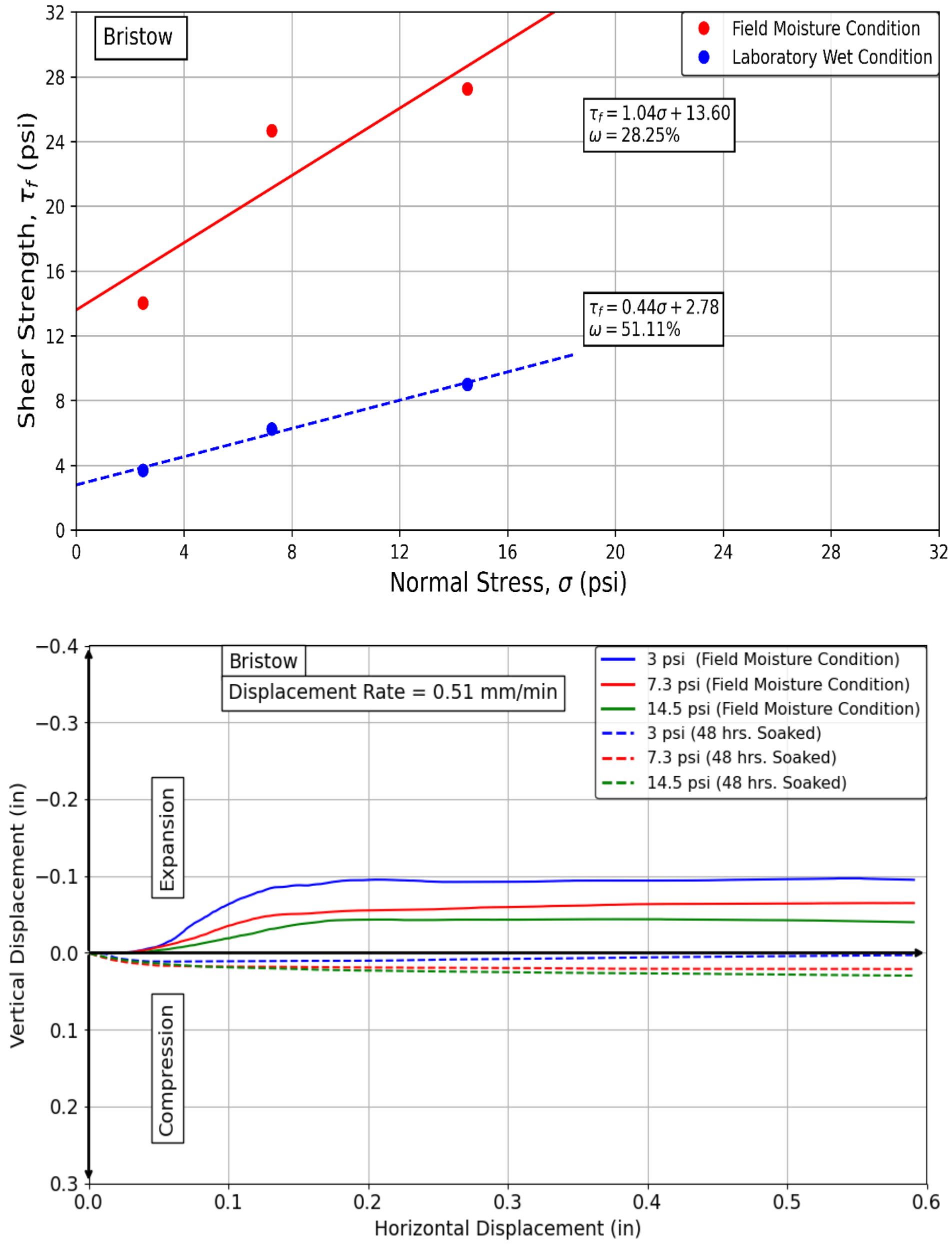
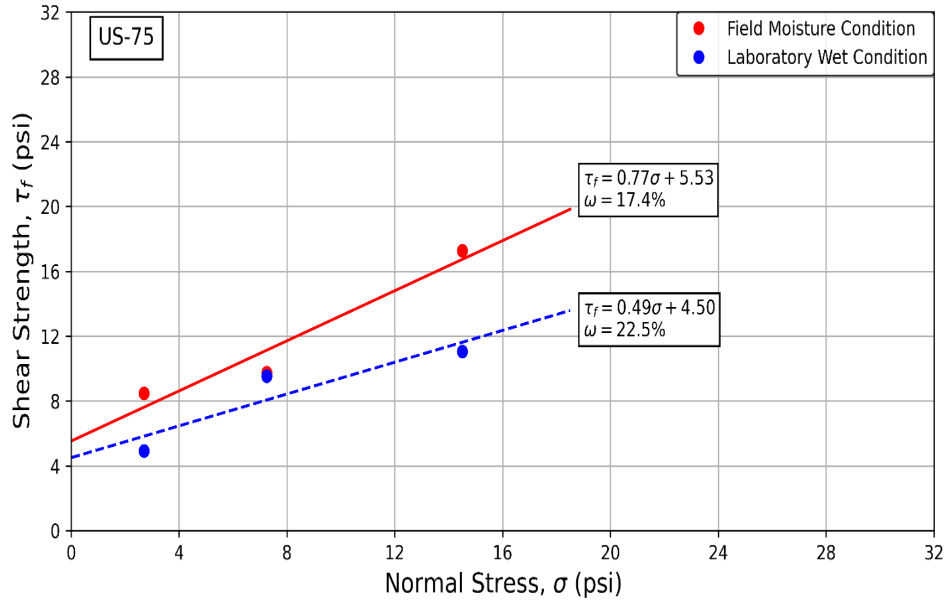


Figure B.6 Bristow Shear Strength Parameters and Change in Height of Specimen against Shear Displacement



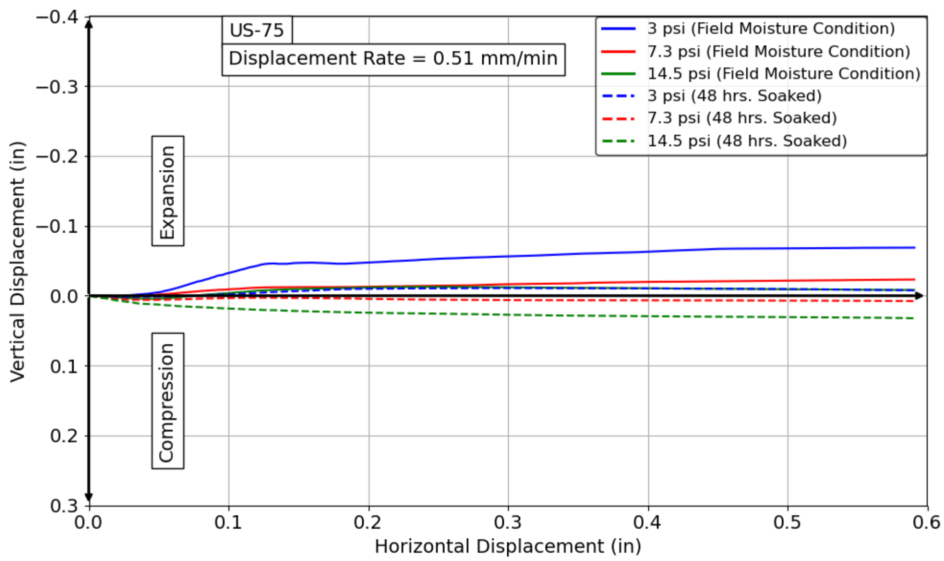


Figure B.7 US-75 Shear Strength Parameters and Change in Height of Specimen against Shear Displacement

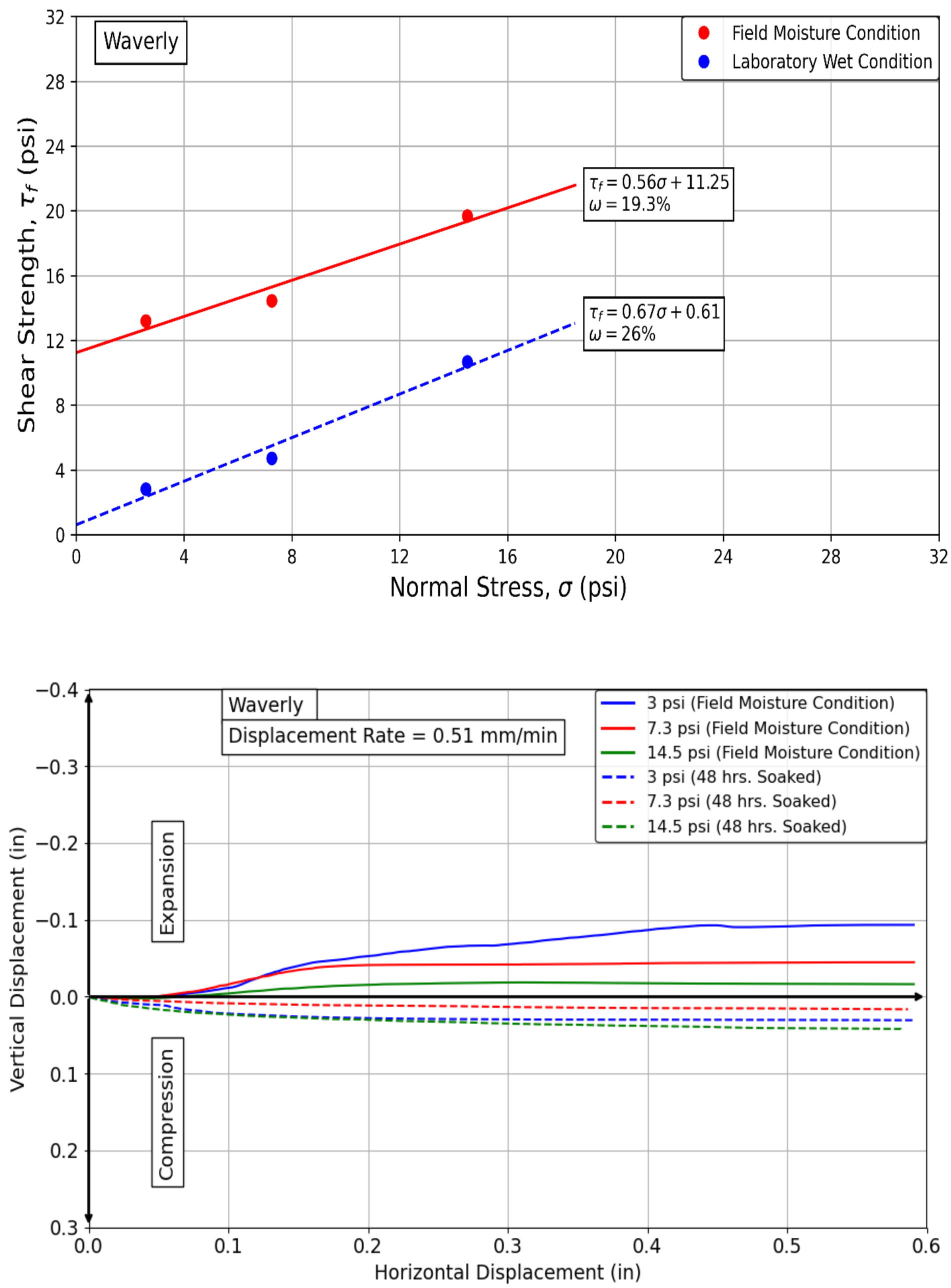
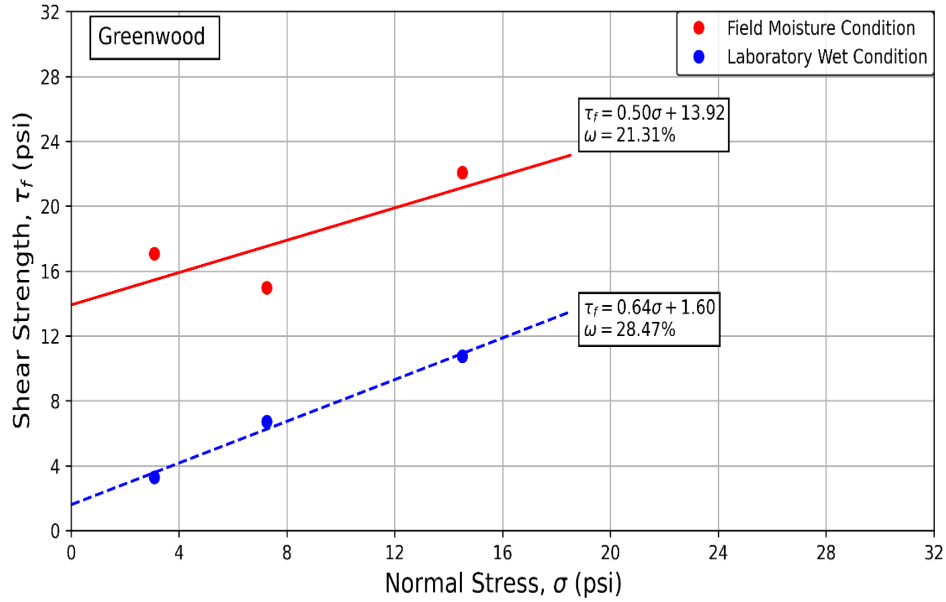


Figure B.8 Waverly Shear Strength Parameters and Change in Height of Specimen against Shear Displacement



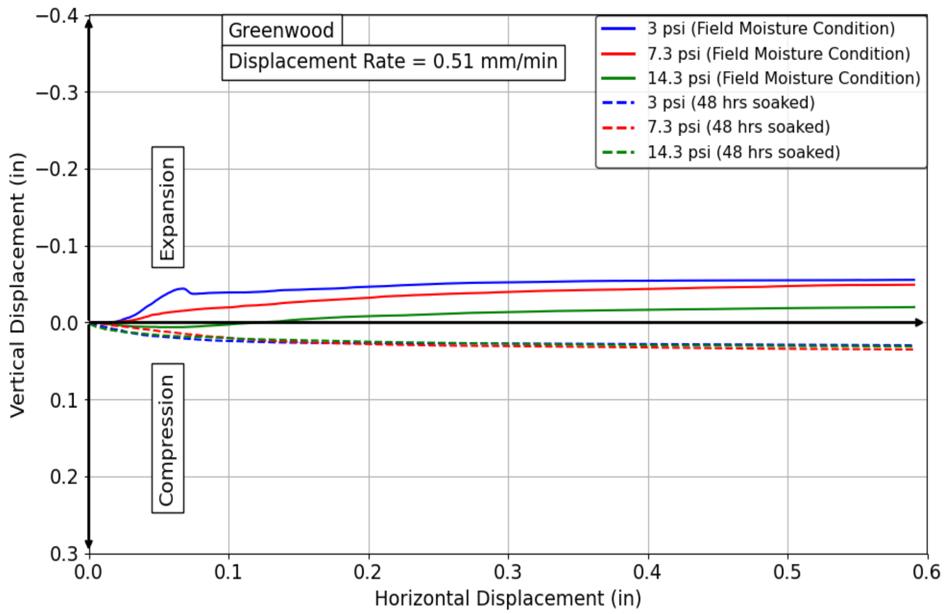
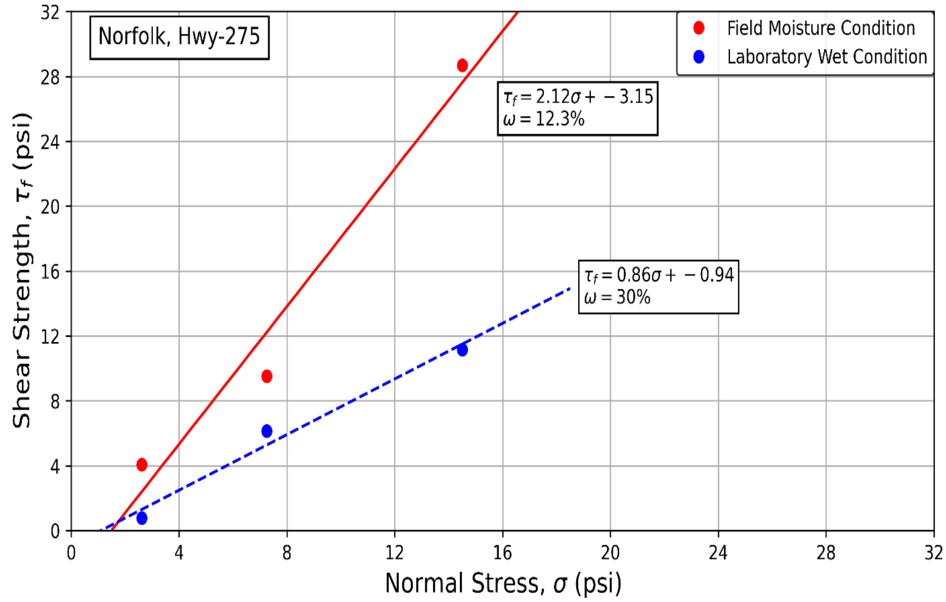


Figure B.9 Greenwood Shear Strength Parameters and Change in Height of Specimen against Shear Displacement



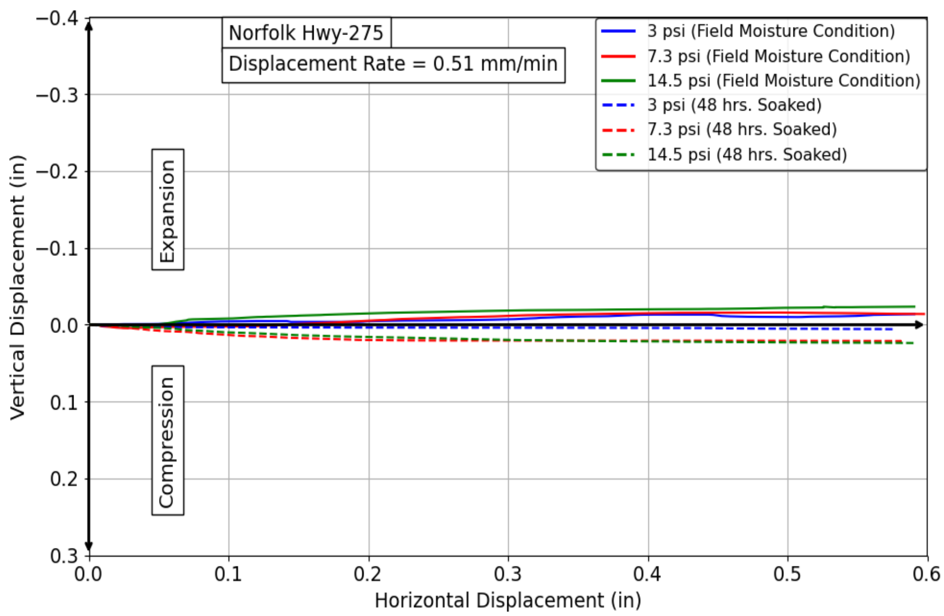


Figure B.10 Norfolk Hwy-275 Shear Strength Parameters and Change in Height of Specimen against Shear Displacement