STRENGTH AND STIFFNESS CHARACTERISTICS OF LIME STABILIZED SOIL IN A FLEXIBLE PAVEMENT DESIGN

by

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Abstract

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Lime treatment has proven to be a useful tool for stabilizing expansive soils. Expansive soils cause major damage to flexible and rigid pavements every year, since seasonal changes in soil moisture creates cyclical changes in the forces acting on a pavement. The use of lime to stabilize soil can reduce the soil's swell potential and add strength and stiffness to a pavement's subgrade. Due to the improved physical properties, lime stabilized soil (LSS) can be considered as part of the structural pavement layers. Utilizing LSS as a subbase in the pavement structure can reduce the thickness of the more expensive pavement layers.

In the North Tarrant Expressway (NTE) Segment 1 project, the results of the geotechnical investigation and quality assurance (QA) programs were used to verify the increase in strength and stiffness of the LSS with time. Unconfined compression strength (UCS) and resilient modulus tests were used, during the design phase, to determine if the reactions between the soil and lime would increase the strength and stiffness of the soil. Eight groups of soils from the NTE Segment 1 project were treated with lime and tested in the lab to confirm a 25,000 psi design assumption for the LSS layer resilient modulus.

During construction, as part of the QA program, UCS and Falling Weight Deflectometer (FWD) tests were conducted to further confirm the design assumption. The design, quality control, and QA processes along with testing results are reviewed and summarized in this thesis to demonstrate that LSS can be considered a pavement subbase, if the proper precautions are taken.

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Chapter 1

Introduction

1.1 Overview

Expansive clay materials are prevalent in Texas. These materials make pavement design and construction a very daunting task to the design engineer. Dealing with the expansivity of clay can cause a project's budget to swell, which can lead to funding issues for any private or public funded project. Ignoring the detrimental effects of expansive clays can cause the accelerated deterioration of a pavement structure and can lead to costly, future rehabilitation. Efficient and cost effective solutions must be developed when designing pavement in areas with expansive soil.

In the past, lime stabilization has proven to be an effective method to modify expansive clay soils in Texas. Lime can immediately reduce clay's plasticity, mitigate swell and shrinking properties, and decrease permeability (Bell 1996). In addition to modification, lime treatment stabilizes the soil, which can increase the long-term strength and stiffness of the soil (Consoli 2011). This increase in strength and stiffness is not typically considered in the design of pavements in Texas (Mallela et al. 2004). However, in the North Tarrant Expressway (NTE) project, located north of Fort Worth, the design team took full advantage of the lime treated soil and incorporated a subbase layer of lime stabilized soil (LSS) into the pavement design.

The reaction between lime and soil can vary depending on the type of soil and a soil's mineralogy (Pedarla et al. 2011). For large highway projects, it is common for the type of soil to vary along the alignment. Due to this variation, a sophisticated quality assurance (QA) program must be developed if LSS is to be used as a pavement subbase. An unprecedented QA program was developed for the NTE project, based on published documents, engineering judgment, and past experience in Texas.

The initial phases of the NTE project consisted of Segment 1 and Segment 2. The two segments made up a 13 mile-long section of I-820 and SH121/183, from I-35W to the SH 121 split north of Fort Worth. Construction on the two segments started in 2010. The project rebuilt existing roads and increased the number of traffic lanes, in order to accommodate the cumulative traffic over the 20 years following construction. The total estimated cost for the project was $2.02 billion. In order to protect this significant investment, the pavement was designed to have an efficient life-cycle, scant of significant rehabilitation or maintenance. The design process and QA results for the NTE Segment 1 were the main focus of this thesis.

It must be mentioned that the current design for the NTE Segment 1 fulfills any contractual obligations between the designer and the client. The design summarized in this study adheres to standards and codes available at the time of its completion. Any future changes to standards, codes, or protocols, which may deem this design unsatisfactory, cannot be used to evaluate this design.

1.2 North Tarrant Express Segment 1

The NTE Segment 1 is located between the I-35W interchange in the West to the I-820 Interchange in the East. The segment was approximately 7 miles long. The existing roads before construction consisted of general purpose lanes (GPL) and frontage roads (FR). The new roads were to consist of additional lanes for general purpose, managed lanes (ML) (toll roads), and frontage roads. The location of the project can be seen in Figure 1.1.

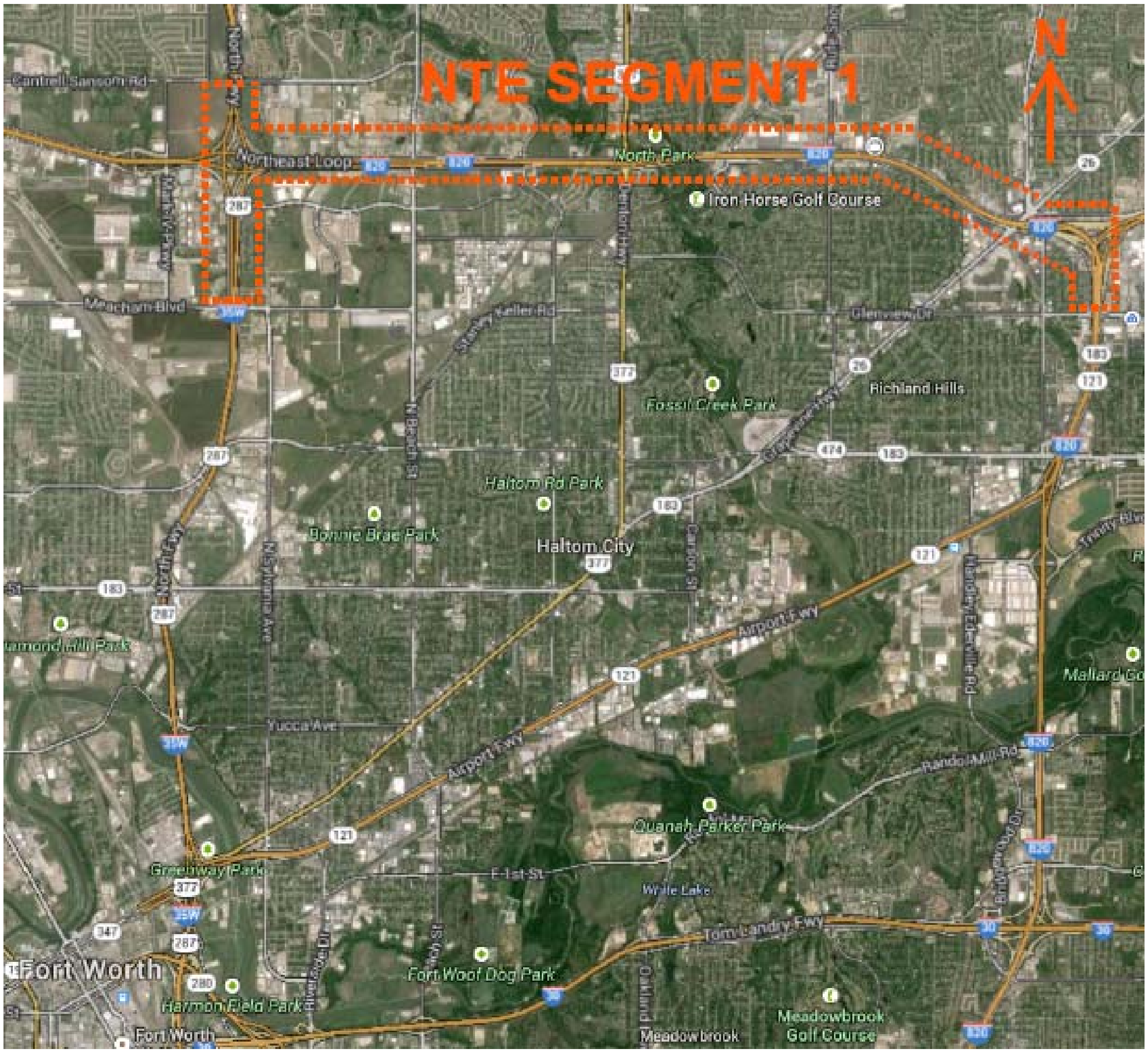


Figure 1.1 NTE Segment 1 Project Location (Google Maps)

The geotechnical design elements of the project consisted of pavement, ground improvement, drilled shaft wall, mechanically earth stabilized wall, and drilled shaft foundation design. This thesis research focuses only on pavement design and ground improvement.

It was decided that flexible pavement would satisfy the local traffic requirements and economical demands of the NTE project. Even though flexible pavement was to be designed, a small area of existing concrete pavement was reused in the final design.

The flexible pavement was designed using the 1993 AASHTO methodology. Structure coefficients were assigned to each pavement base and subbase. The predicted number of 20 year expected Equivalent Single Axle Load (ESAL) aided in determining layer thicknesses. Pavement structural layers consisted of Stone Matrix Asphalt (SMA), Hot Mix Asphalt (HMA), Flexbase (FB), LSS, and a natural soil subgrade. In most locations, additional thicknesses of the LSS beyond the required structural thickness were needed to counter the expansive properties of the natural clay soil.

During the geotechnical investigation (GI) of the NTE Segment 1 project, samples were gathered and mixed with lime. Following proper mixing protocols, these samples were subjected to testing, which consisted of PI, grading curves, and ph series, unconfined compression strength (UCS), and resilient modulus tests. This testing estimated the optimum lime content and measured the increase in strength and stiffness of the treated soil attributed to the lime/clay reaction. Based on the results from these tests, the appropriate amount of lime was added to the soil throughout the project during construction.

A quality control (QC) program was developed to supervise the construction of the LSS. The QA program provided a way to verify the reaction between the lime and clay during construction. Over 5000 UCS tests were conducted on lime treated material as part of the QA program. The author has compiled test results from various laboratories investigations into a database, which was used in this thesis to analysis the GI and QA programs.

In addition to the laboratory testing, Falling Weight Deflectometer (FWD) tests were conducted in areas of interest, to assure the stiffness properties of the lime treated soil were increasing as expected.

The QC and QA programs assured the designers that the lime was being mixed properly and the LSS was obtaining the required strength characteristics assumed during the pavement design. The design and QA testing programs will be discussed in Chapter 4 and Chapter 6 of this study.

1.3 Project Geology

The NTE Segment 1 spanned across multiple geologic formations, which can be seen in Figure 1.2. Changes in geologic formations along a project alignment could cause serious problems during the design and construction of any roadway. The formations in the Segment 1 project were the Quaternary Alluvium (Qal); Quaternary Terrace (Qt); Mainstreet and Grayson (undivided) (Kgm); Denton, Weno, Pawpaw (undivided) (Kpd); and the Fort Worth and Duck Creek (undivided) (Kfd) formations. When developing any geotechnical investigation or ground improvement program, a geotechnical design engineer must have a strong understanding of every geologic formation in the project area.

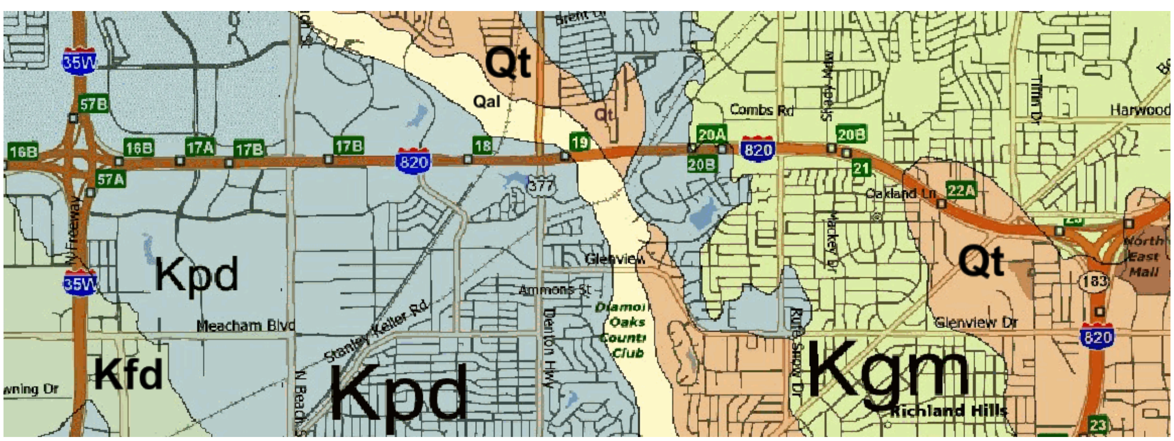


Figure 1.2 Geologic Formations along Segment 1 (Bureau of Economic Geology 1987)

The Denton, Weno, Pawpaw (undivided) was the largest formation in the project area. Kpd consists of clay, marl, and shale interbedded with limestone and sandstone ledges (Hudak 1998). This description was generally confirmed during the geotechnical investigation with sparse observations of marl or sandstone. It is likely that the marl was categorized as shale. As the formation moves east, the shale and limestone elevation decreases and the depth becomes shallower until the formation is split by the Qal and Qt formation. After the Qal and Qt split, the elevation of rock remains low. Two small bodies of water run through the split called Big Fossil and Singing Hills Creek. It would be pertinent to assume this water flow eroded the once high rock table, and, in the process, transported soils from different formations to the location. The clay soils in the Kpd formation are similar to those in the Grayson formation (Kgm) (Hudak 1998).

The Mainstreet and Grayson Formation consists of Grayson chalk/marl underlain by Mainstreet limestone which overlays compacted Pawpaw clay. This formation begins the transition into the Woodbine formation. Records indicated montmorillonite mineral dominates the Kgm formation (Hudak 1998).

The Quaternary Alluvium and Terrace are young formations in geological terms. The materials found in the alluvium formation are usually transferred to an area via a body of water. Typically, this formation consists of a variation of clay, sand, and gravel in flood plains. The terrace formation is similar to the alluvium formation but is a remnant of the Trinity River's previous path. When the Trinity River changed its path, these soils were left behind.

The Fort Worth and Duck Creek (undivided) formation has minimal effect on the NTE Segment 1 project. Clay overlaying limestone can be found in this area. The limestone elevation in this formation is close to the ground surface.

Historical data on some of the different types of soils found in these formations are shown in Table 1.1. This table was based on data collected by Tarrant County Soil

Survey (1981). Only the most abundant soils in the project area are summarized in the table.

Table 1.1 Soil Properties in NTE Segment 1 (Ressel 1981)

Geologic Formation	Type	Average						Shrink Swell Potential
		Percent Passing Sieve				LL	PI	
		4	10	40	200			
Kfd	Purves-Urban Complex	95	90	87.5	82.5	58	35	High
	Purves Clay	95	90	87.5	82.5	58	35	High
	Sanger Clay	97.5	97.5	95	87.5	50	35	High
Kpd	Sanger Clay	97.5	97.5	95	87.5	50	35	High
	Sanger-Urban Complex	97.5	97.5	95	87.5	50	35	High
	Slidell clay	97.5	97.5	97.5	87.5	59.5	35	High
Kgm	Lindal Clay Loam	95	92.5	85	60	30	14	Low
	Purves-Urban Complex	95	90	87.5	82.5	58	35	High
	Sanger-Urban Complex	97.5	97.5	95	87.5	50	35	High
Qal	Frio Silty Clay	90	90	85	77.5	43.5	27	Moderate
Qt	Navo-Urban Complex	97.5	97.5	95	70	36.5	18	Moderate
	Sunev-Urban Complex	95	90	90	52.5	32.5	13	Low
	Urban Land	97.5	97.5	82.5	45	<25	-	Low

1.4 Expansive Soils

One of the key concerns, during the pavement design for the NTE Segment 1, was the expansive properties of the native soil. Due to historical data, the native clay was expected to expand when inundated with water and contract when desiccated. Over the span of a year, soil naturally enters into a wet and dry state along with seasonal changes. Typically, summer is a drier period and winter brings wetter conditions. If nothing is done to reduce the soil expansivity, pavement placed in that location could experience accelerated deterioration due to changes in pressure.

Texas is well known for expansive clay induced pavement deformation; the situation along the NTE Segment 1 alignment was no exception. This was confirmed during the geotechnical investigation. Based on Atterberg limits, typical soil characteristics tests, Figure 1.3 (adapted from Marin-Nieto, 1997 and 2007), and Figure 1.4 (adapted from Mitchell and Gardner, 1975 and Gibbs, 1969) the soils in the project area were expected to range from low to highly expansive (Holtz et al. 2011).

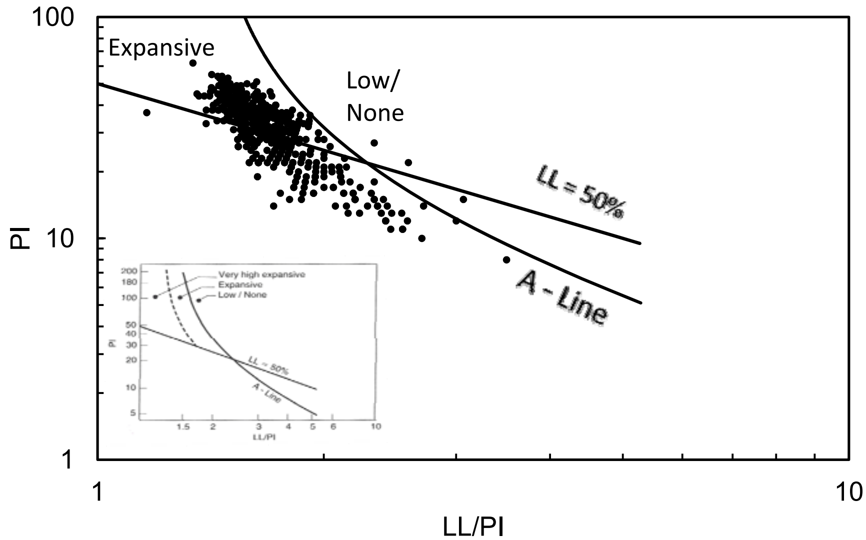


Figure 1.3 Log PI versus Log LL/PI (Holtz et al. 2011)

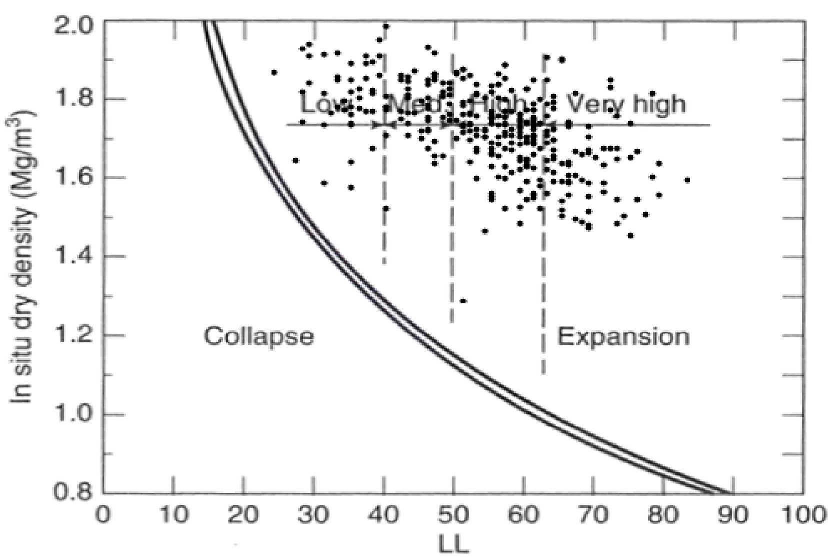


Figure 1.4 In Situ Dry Density and Liquid Limit (Holtz et al. 2011)

Typically, if expansive soil is found, it either must be removed or modified to create a more workable and reliable base and subgrade for pavement. Removing and replacing the expansive material with an inert material, typically, would not be cost effective. Lime stabilization had been used in Texas before, and many case studies and publications demonstrating its effectiveness had been recorded. Based on past and local experience, lime stabilization was chosen to be the optimum solution.

1.5 Lime Stabilization

Lime is typically produced in three forms quicklime (CaO), hydrated lime ($Ca[OH]_2$) and lime slurry, which is a suspension of lime hydrate in water (Little 1995). Quicklime reacts with water and forms hydrated lime. Quicklime is usually placed and water is added to the system; whereas hydrated lime along with the soil's moisture content already contains the required amount of water.

TxDOT's *Guidelines for Modification and Stabilization of Soils and Base for Use in Pavement Structures* recommends lime treatment if sieve analysis shows 25% or greater passes the No. 200 sieve and the PI of the material is greater than 15 (TxDOT 2005). However, basing the effectiveness of lime stabilization on the plasticity of the soil alone is an unacceptable technique. Agencies have reported a lack of lime reaction and subgrade failures, in spite of the soil having the recommended parameters for lime treatment (Pedarla et al. 2011). In addition to the sieve analysis and plasticity value, an investigation into a soil's mineralogy can aid in the evaluation of the soil's reaction potential.

The proper percentage of lime by weight to add to the clay must be determined during the geotechnical investigation. This percentage is found through extensive testing and evaluation of the LSS's physical properties, such as pH, PI, compression strength, and stiffness. The quantity of lime must be large enough to satisfy the clay's fixation point, which is the minimum amount of lime required to bring initial, immediate reactions to fruition, and create pozzolanic reactions after the initial reactions have finished (Bell 1996). An offhand estimate for lime content is 1% by weight of lime for every 10% of clay in the soil (Ingles 1987).

The initial reactions between lime and soil cause permanent alterations in the soil's chemical makeup. These reactions are cation exchanges between the very positive calcium ion from lime and present, weak cations surrounding the negatively charged clay particles (Little 1995). When lime is mixed with the soil, the bonds within lime molecules break and the very positively charged calcium cations begin to replace the weaker cations surrounding the clay minerals (Little 1995).

This cation exchange occurs immediately and the soil begins to flocculate. The soil becomes more workable and appears to be more granular than clay like. After this initial process completes, the clay lime mixture gains a small amount of strength. This process also reduces the plasticity index of the soil by increasing its plastic limit, which occurs due to stronger bonds forming between particles (Little 1995). After flocculation and cation exchange, the soil demonstrates a reduction in water absorption potential, an increase in friction angle and shear strength, and a greater workability. These are the short-term effects of the lime reaction (Little 1995).

Pozzolanic reactions begin when the siliceous or aluminous material from the clay are exposed to water and calcium hydroxide from the lime mixture. When enough lime is added to the clay, the pH of the system will increase significantly, causing the clay silica and alumina to become soluble and reactive (Little 1995). The reaction between the calcium, aluminum, silica forms a cemented material. These bonds are where LSS gains significant strength. The pozzolanic reactions will continue as long as the pH remains high and residual calcium remains in the system (Little 1995). This can lead to large strength increases over time.

The aforementioned immediate reactions will occur in all clays. Unfortunately, the presence of pozzolanic bonding, time required to form these bonds, and strength of those bonds varies with the mineralogy of the clay soil. The initial reactions are referred to as soil modification, and the long-term strength development is called soil stabilization.

1.6 Lime Stabilized Soil (Structural)

Lime modification reduces the expansive qualities of the soil to a benign level. Concurrently along with the reduction in expansivity, improvement in soil's stiffness and strength from lime stabilization can be observed. This improvement in the soil's engineering properties can lead to reductions in the thickness of other more expensive pavement layers. Due to these improved subbase strength and stiffness qualities, the required quantities of hot mix asphalt and flexbase were minimized in the Segment 1 NTE pavement design, which was cost effective and optimized the design.

In order to consider the LSS a subbase for pavement, the LSS must obtain certain strength and stiffness values, which leads to a designation of a structural number. These strength and stiffness values are represented by the soils compression strength and resilient modulus. These values can be estimated through testing, correlations, or experience.

1.7 Thesis Objective

The objectives of this study are the following:

1. Compare tests conducted on the lime stabilized soil during the geotechnical investigation to similar tests conducted in the field.

2. Evaluate correlations between compression strength and resilient modulus for project soil treated with lime.
3. Confirm design assumptions used in the flexible pavement design.
4. Evaluate the quality assurance program conducted during lime stabilized soil construction.
5. Determine recommendations for future projects in regards to the use of lime stabilized soil as a pavement subbase.

These objectives will be achieved through:

1. Evaluation of the laboratory test results, conducted during the geotechnical investigation and QC/QA phase, and established correlations.
2. Evaluation of the QA program, where multiple UCS tests were conducted at every location lime was mixed with soil and limited FWD tests were conducted periodically on the constructed LSS.

1.8 Thesis Organization and Summary

Chapter 2 presents a literature review of clay mineralogy and expansivity, Potential Vertical Rise, flexible pavement design, and an evaluation of the LSS and correlations used to express resilient modulus values based on UCS results. The effects of soluble sulfates and accelerated lime soil curing methods are also discussed.

Chapter 3 presents a summary of the testing procedures followed during the design and construction phases.

Chapter 4 reviews the geotechnical investigation conducted during the design phase of the project and summarizes the results of the geotechnical investigation. This provides an overall summary of the soil characteristics in the project site.

Chapter 5 reviews design of the lime stabilized layers and flexible pavement based on the results from the geotechnical investigation.

Chapter 6 summarizes the QA and QC programs implemented during construction of the LSS. These programs served to guarantee the material met required specifications. Results of UCS and FWD tests are presented and discussed.

Chapter 7 summarizes the conclusions from design procedure and quality program. Recommendations for future projects are also discussed.

A list of references used in this paper follows the conclusion section.

Chapter 2

Literature Review

2.1 Introduction

In order to fully understand the GI testing, design, and the QC/QA program, a basic understanding of clay mineralogy, potential vertical rise, flexible pavement design, physical properties of lime treated soil, the effects of sulfates, and accelerated curing methods is required. This review was conducted through the summarization of published material by AASHTO, the National Lime Association, academic organizations, and practicing engineers and researchers.

2.2 Clay Mineralogy and Expansivity

The damage from expansive soils in the United States costs more on an annual average basis than all the annual natural disasters combined (Kehew 1995). The cost of structure and pavement damage from expanding soils amounts to billions of dollars each year (Kehew 1995). The distribution of expansive soils in the United States is sporadic but the number of states affected by expansive soils is significant (AASHTO 1993).

The expansive quality of clay can be explained by breaking down its molecular composition. Clay soil contains clay minerals. The most common clay minerals are made up of sheets of silica and oxygen atoms, a silica tetrahedron, and/or aluminum and oxygen molecules, an alumina octahedron (Little 1995). The silica tetrahedron is formed by a silicon atom (Si) joined by 4 oxygen atoms (O) (Holtz et al. 2011). The aluminum octahedron is formed by an aluminum atom (Al) and 8 oxygen atoms or 8 hydroxyl molecules (OH) (Holtz et al. 2011). Eighty-three percent of the Earth's crust contains these two molecules (Little 1995). The oxygen atoms in the system are shared by adjoining silica or aluminum atoms which form sheets of molecules. These sheets form in different ratios and manners to create different clay minerals (Little 1995). The way these clay minerals are formed and bonded together contributes to the mineral's plasticity. Clay minerals ordinarily have a negative charge and a high amount of surface area, which both contribute to their attraction of water (Pedarla et al. 2011). The negative charge attracts the bipolar water molecules and pushes apart the clay sheets, causing expansion (Little 1995).

The most common clay minerals found in soils are montmorillonite, kaolinite, and illite (Pedarla et al. 2011). Each of these three groups is characterized by the stacking order of the alumina and silica sheets (Mitchell & Soga 2005). The arrangement, ratio and number of the silica tetrahedral and aluminum octahedron further define the type of mineral (Holtz et al. 2011).

Montmorillonite is part of the 2:1 mineral group and smectites subgroup. The 2:1 clay minerals are composed of one alumina octahedral between two silica tetrahedral; oxygen atoms or hydroxyl molecules are shared between the tetrahedral and octahedral (Holtz et al. 2011). This 2:1 bonding forms a single layer which is bonded to other layers by Van der Waals' forces, which are weak fluctuating dipole bonds. Due to the weak Van der Walls' bonding the layers of silica and alumina are very susceptible to water infiltration (Mitchell & Soga 2005). This infiltration layers causes the layers to move apart and the overall system to swell. Skempton (1953) identifies a high PI ratio to clay fraction for montmorillonite. This ratio, which is the activity value, can help define the type of clay in a project area. Bell (1996) shows montmorillonite to have high liquid limit and plasticity values.

The 1:1 mineral group contains the kaolinite mineral which is part of the kaolinite-serpentine subgroup (Holtz et al. 2011). The layers in this group are made up of sheets of one alumina tetrahedral and one alumina octahedral, which share the oxygen atoms (Mitchell & Soga 2005). This 1:1 layer is bonded with other layers by a hydrogen bond. This hydrogen bond is much stronger than the Van der Waals' forces bonding of the montmorillonite layers (Holtz et al. 2011). This strong hydrogen bond makes kaolinite less susceptible to water infiltration.

Illite is the most common clay mineral found in engineering projects (Mitchell & Soga 2005). Illite has the same 2:1 ratio that can be found in montmorillonite, but the layers are bonded with a potassium ion, which is much stronger than the Van der Waal's bond. Along with the potassium ion, isomorphous substitution of aluminum for silica in the silica sheet strengthens the bond (Holtz et al. 2011). These strong bonds permit less water infiltration than the bonds in the montmorillonite and kaolinite minerals. Ergo, the expansivity of illite clay would be lower than those clays with Montmorillonite or Kaolinite minerals.

Two of the three most common clay minerals are susceptible to swelling. Identification of the type of mineral can assist in the identification of swell and reactivity potential. Montmorillonite exhibits a high affinity for the initial cation exchange reaction from lime, yet Kaolinite expresses a relatively low cation exchange capacity (Bell 1996).

The pozzolanic reaction capacities of clays vary along with the amount of the specific clay mineral present within the soil (Bell 1996). UCS tests for clays containing high amounts of montmorillonite have higher strength results at lower lime contents than at higher lime content; kaolinite dominate clays show relatively high UCS results with reasonable increases in lime content, 2 to 10% (Bell 1996).

2.3 Potential Vertical Rise

Potential Vertical Rise (PVR) refers to the native soil's potential to swell when water saturates the system. There are three relatively simple methods used to estimate the PVR in a location: the Tex-124-E, AASHTO, and Schneider and Poor methods (Endley et al. 1992). The Tex-124-E predicts a higher value of PVR than the AASHTO and Schneider methods of PVR calculation (Endley et al. 1992). Table 2.1 summarizes and compares the PVR values from each method which were calculated by Endley et al. (1992). When designing pavement placed on zones with highly expansive soil, conservatism is typically the correct philosophy to follow.

Table 2.1 Comparison of PVR Values (Endley et al. 1992)

Active Depth (ft)	Plasticity Index	Moisture Condition	Potential Vertical Rise (in)		
			Tex-124-E	AASHTO	Schneider and Poor
5	50	Dry	2.80	1.20	1.45
10	50	Dry	4.60	2.45	2.36
5	50	Wet	1.50	0.50	0.53
10	50	Wet	2.20	0.85	0.95
5	40	Dry	2.00	0.83	1.08
10	40	Dry	3.30	1.80	1.86
5	40	Wet	1.15	0.20	0.52
10	40	Wet	1.55	0.30	0.97
5	60	Wet	1.80	0.70	0.57
10	60	Wet	2.90	1.30	1.05
5	60	Average	2.60	1.10	0.94
10	60	Average	4.20	2.20	1.59

TxDOT recommends Tex-124-E for determining potential vertical rise. This procedure quantifies the PVR in inches for a given soil active zone. The method uses the soils plasticity index, liquid limit, wet density, moisture content, % finer than # 40 sieve, Figure 2.1, and Figure 2.2 to determine the PVR for a soil (TxDOT 1999).

Typically, the greater the liquid limit and plasticity index of a soil, the greater the swelling potential. Liquid limits greater than 40 and plasticity indexes greater than 25 suggest a high potential for swelling (Endley et al. 1992). The plasticity index, saturation moisture, and cation exchange capacity are some of the most important parameters used to estimate swell potential (Gill & Reaves 1957).

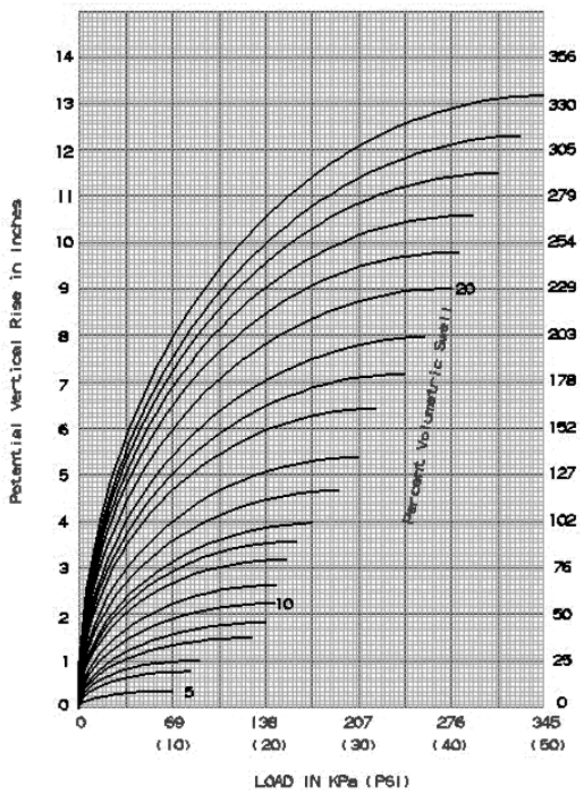


Figure 2.1 Relations of Load to PVR (Tex-124-E)

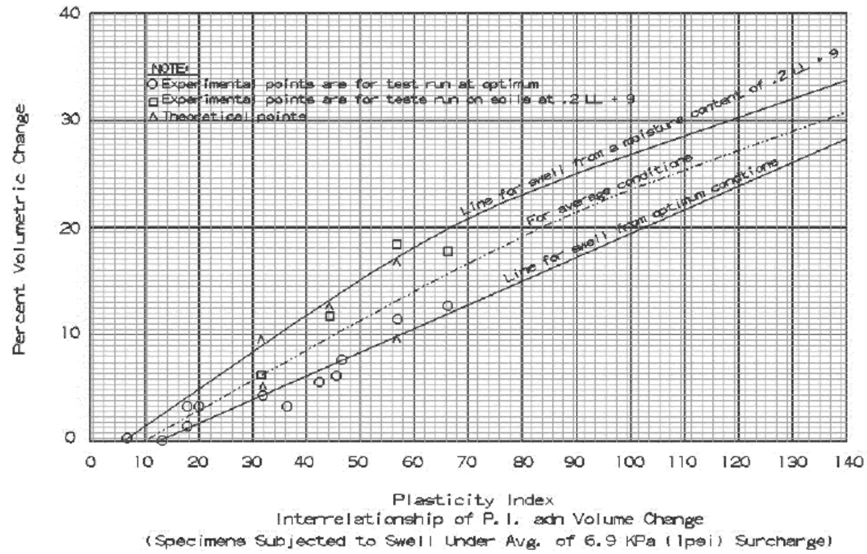


Figure 2.2 Relationship of PI to Volume Change (Tex-124-E)

The Texas method requires an active depth to be determined in the area. The active depth refers to the depth soil that either contributes to or has the potential to heave (Nelson et al. 2001). Active zone thickness can vary from area to area. The active zone depth can be measure though long-term observations of moisture content fluctuation over a period of multiple years or by standard practice and local experience.

The Tex-124-E recommends the assumed active zone be divided into multiple segments of equal size, and soil characteristics are assigned to each layer reflecting the results of laboratory testing. Typically these layers are divided such that each layer experiences an overburden stress increase of 1 psi as the depth increases layer by layer.

Percent volumetric swell must be determined for each location using Figure 2.2 and through an evaluation of the soils in-situ saturation (TxDOT 1999). For simplification purposes, the engineer can assume dry conditions (a moisture content of 20% of LL +

9%) which would be conservative. This assumption would lead to a greater overall volumetric swell of the system.

Figure 2.2 provides the percent volume swell of a loaded specimen. In order to use Figure 2.1, the percent swell change with no load value must be used. This can be found by taking the percent volumetric change obtained from Figure 2.2 and multiplying it by 1.07 and adding 2.6% to the product. This value is used in Figure 2.1, along with the applied load at the top and the bottom of each specified layer, to calculate the PVR at the top and the bottom of each layer. The difference between the top and the bottom PVR calculations is the overall assigned PVR for that layer. The net sum of these values is the total active zone PVR. Correction factors are considered when the percent passing sieve No. 40 is less than 100%. (TxDOT 1999)

Using the PVR value, the design engineer determines the depth of treatment required to reduce the swell potential to a safe value. TxDOT Pavement Design Guide requires any main road to have a PVR less than 1.5" and any frontage road to have a PVR less than 2.0" (TxDOT 2011).

Treatment of the upper active zone layers with inert material leads to a significant decrease in PVR, as the upper layers contribute more to the overall PVR than the bottom layers of the active zone. Approximately 80% of the total swelling occurs in the upper 50% of the active zone (Rao et al. 1988). Replacement of the upper 5 feet of the active zone leads to significant reductions in PVR. The pavement structure is included in the treatment thickness.

2.4 Flexible Pavement Design

The American Association of State Highway and Transportation Officials (AASHTO) pavement design methodology is a well-recognized method for designing pavement in the United States. The AASHTO, previously AASHO, methodology is based off the AASHO Road Test conducted in the late 1950s in Ottawa, Illinois. A loop of different pavement structures was construction and traffic loads were applied to the structure over a period of 2 years (AASHTO 1993). The information gathered on the deterioration of the pavement with traffic loading forms the base of the design methodology (AASHTO 1993). The methodology has been updated and improved over the past half century as more information has been obtain on pavement performance over time. However, the AASHTO method is limited, since the design cannot be used to predict different modes of pavement distress and only satisfies the serviceability requirement (Mamlouk et al. 2000).

AASHTO flexible pavement design is based around a single equation, Equation 2.1 (AASHTO 1993). This equation was developed empirically from the previously mentioned road test. The equation has many variables which change with each different pavement design. Typically, the equation is used to find a pavement's overall structural number. This overall structural number is used in Equation 2.2, which applies individual layer structural numbers, thicknesses, and drainage coefficients (AASHTO 1993). Through an iterative process the different layer thicknesses are determined.

$$\log_{10}(W_{18}) = Z_R \times S_O + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta\, PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 \times \log_{10} M_R - 8.07$$

(2.1)

where

SN = structural number

W_{18} = number of 18 kip ESALs

Z_R = standard normal deviate

S_O = combined standard error of the traffic and performance prediction

Δ PSI = difference between the initial design serviceability index and final serviceability index

M_R = resilient modulus (psi)

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_i D_i m_i \tag{2.2}$$

where

a_i = i^{th} layer coefficient

D_i = i^{th} layer thickness (in)

m_i = i^{th} layer drainage coefficient

The AASTHO design equation has many variables that must be determined by the design engineer. One of the more important variables is the predicted number of 18 kip ESALs expected over the pavement's lifecycle. The number of ESAL loads is based on the amount of traffic using the roadway every day, which is obtained by a traffic volume analysis conducted on a given stretch of roadway (AASHTO 1993). However, future values must be predicted based on a growth analysis for the life span of the roadway. The pavement must be designed to accommodate the expected, accumulated ESAL loads.

The standard normal deviate, Z_R, is a function of the design reliability level. AASHTO design manual defines the reliability concept as “the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period” (AASHTO 1993). Many variables are considered when determining a standard normal deviate and standard deviation. These variables include, but are not limited to, the reliability of the pavement to perform as expected, the reliability of traffic loads to increase as predicted, and the reliability of the environment to behave as predicted over the pavement lifecycle (AASHTO 1993). A reliability level is assigned based on these factors and each level has a corresponding normal deviate. This relationship can be seen in Table 2.2, which demonstrates that the greater the required percent reliability, the higher the absolute value of the standard normal deviate. An increase in the reliability value causes the number of allowable ESALs for the pavement to decrease when other variables remain constant.

Table 2.2 Standard Normal Deviate Values for Reliability Levels (AASHTO 1993)

Reliability, R (%)	Standard Normal Deviate, ZR
50	0.000
60	-0.253
70	-0.524
75	-0.674
80	-0.841
85	-1.037
90	-1.282
91	-1.340
92	-1.405
93	-1.476
94	-1.555
95	-1.645
96	-1.751
97	-1.881
98	-2.054
99	-2.327
99.9	-3.090
99.99	-3.750

The overall standard deviation, S_O, is based on performance prediction models. The range of this value spans from 0.40 to 0.50 for flexible pavements (AASHTO 1993). As this value decreases, so then will the expected number of allowable ESALs.

The change in Present Serviceability Index (PSI) is an indication of the allowable deterioration of the pavement structure (AASHTO 1993). A large change in PSI would suggest more deterioration of the pavement would be allowed than a change in PSI of a lower value. Many agencies have different ways of determining the PSI of a pavement structure (AASHTO 1993). However, these evaluations are typically based on the quantity and extents of physical distresses and pavement roughness. PSI values range from 5, which is a new perfectly constructed road, to 0, which is a completely dilapidated road (AASHTO 1993). Change in PSI is calculated by subtracted the expected final PSI value from the initial starting PSI value. However, a pavement may fail before the final PSI value is achieved, which is the shortcoming in the ASSHTO design that was mentioned previously (Mamlouk et al. 2000).

The final component of the design equation is the contribution of the roadbed soil's resilient modulus, M_R. The resilient modulus is a measure of the subgrade elastic modulus at a given stress level, which is defined by applied deviatoric stress to recoverable strain (AASHTO 1986). Many times organizations do not have the required funding or time to conduct a study to determine the local subgrade resilient modulus. Suitable values have been established to estimate the resilient modulus of the subgrade based on California Bearing Ratio (CBR), R-Value, and soil index test results (AASHTO 1993). A variation in M_R has the most pronounced affect on the overall structural number (Baus & Fogg 1989).

Applying the described variables to the design equation allows the design engineer to determine the overall structural number of the pavement system. This overall structural number is used along with economical and drainage information to design the individual layer thicknesses (AASHTO 1993).

After the overall structural number is determined, the pavement thicknesses must be calculated using Equation 2.2 (AASHTO 1993). Typical flexible pavement structures consist of the following layers: surface course, base course, subbase course, and subgrade or roadbed. The structure can also include a drainage layer and/or filter material. These layers are assigned structural coefficients based on their elastic modulus which are dependent on the type of material used in each layer (AASHTO 1993). The structural coefficient is a regression constant allowing (to be used with the smallest error (AASHTO 1993). Determining a realistic structural coefficient can be a difficult task for the design engineer as the original road test was built of asphalt, crushed limestone base, gravel subbase these are the only structural coefficients directly measured. All other pavement materials must have a structural coefficient assigned to them based on indirect measurements (AASHTO 1993).

Normally, Flexible pavement surface course consists of an asphalt concrete material. These materials typically have relatively high elastic modulus. The layer coefficient for this material can be estimated using Figure 2.3 (AASHTO 1993). The structural coefficient for a granular base course can be estimated from Figure 2.4, and the structure coefficient for subbase layers can be calculated using Figure 2.5 (AASHTO 1993).

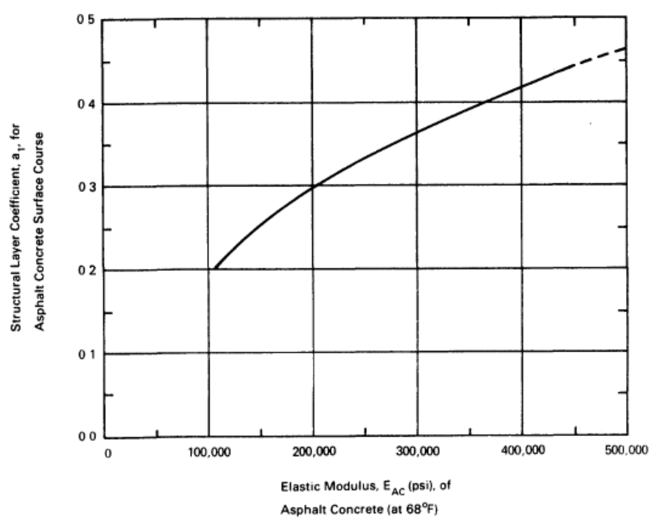


Figure 2.3 Structural Coefficient of Dense Graded Asphalt Concrete (AASHTO 1993)

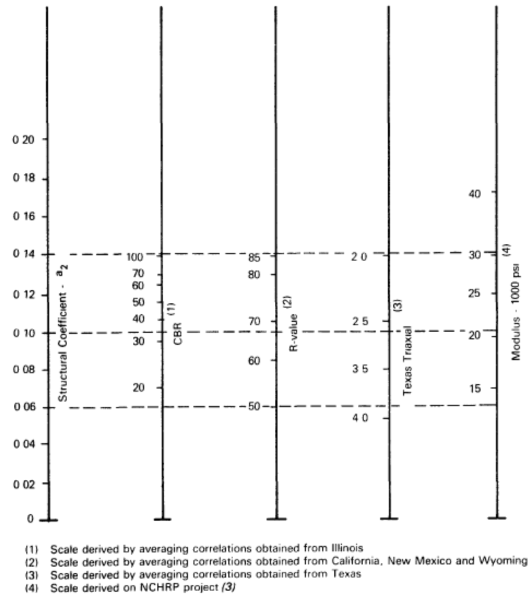


Figure 2.4 Granular Base Coefficient based on Various Strength Parameters (AASHTO 1993)

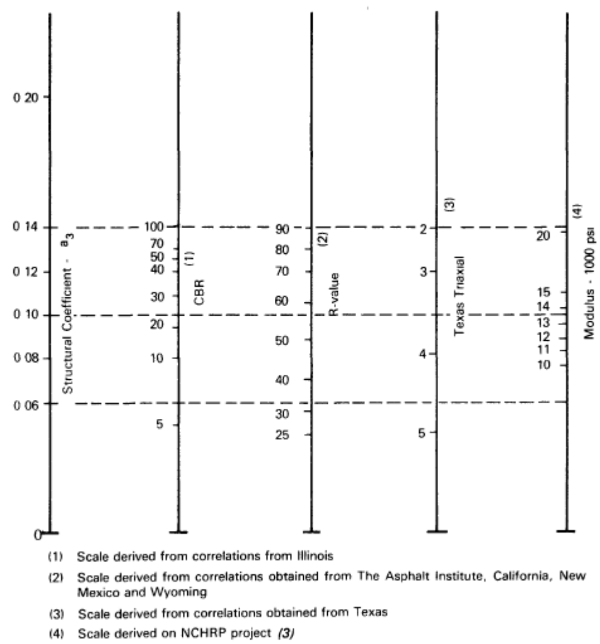


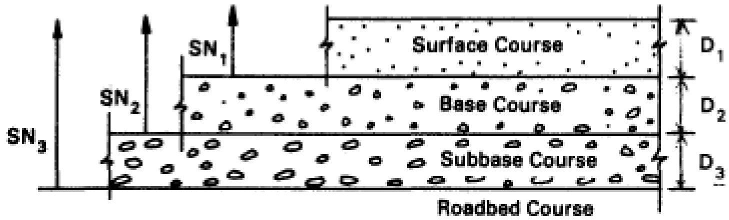
Figure 2.5 Granular Subbase Coefficient based on Various Strength Parameters (AASHTO 1993)

The type of material used for each layer will be specified by the engineer. These materials should be easy to obtain and pass the specific local specifications. Typically, a cost analysis is considered when determining which materials should be used in a pavement structure.

After determining the type of materials and assigning the correct layer coefficient to each material, individual layer thicknesses must be evaluated. However, since flexible pavement is a layered system, each layer must be evaluated using a top down evaluation method (AASHTO 1993).

The structural number must be evaluated for a system assuming the top course is the only structural layer; the resilient modulus of the second course is used in (to obtain a SN (AASHTO 1993). This individual SN is then used with a condensed version of Equation 2.2 to determine the minimum thickness of the surface course.

This process is repeated using a difference in individual layer structural numbers to determine minimum thicknesses of the subsequent layers. Figure 2.6 is provided in the AASHTO design manual to assist in understanding this process (AASHTO 1993). These minimum thicknesses are compared to the thicknesses calculated for the overall pavement structure and final design thicknesses are determined.



$$D^*_1 \geq \frac{SN_1}{a_1}$$

$$SN^*_1 = a_1 D^*_1 \geq SN_1$$

$$D^*_2 \geq \frac{SN_2 - SN^*_1}{a_2 m_2}$$

$$SN^*_1 + SN^*_2 \geq SN_2$$

$$D^*_3 \geq \frac{SN_3 - (SN^*_1 + SN^*_2)}{a_3 m_3}$$

1) a, D, m and SN are as defined in the text and are minimum required values

2) An asterisk with D or SN indicates that it represents the value actually used, which must be equal to or greater than the required value

Figure 2.6 Layered Analysis Process for Determining Minimal Layer Thicknesses (AASHTO 1993)

This section is only a simplified explanation of the AASTHO 1993 design procedure for flexible pavements. There are many additional evaluations that should be considered when designing flexible pavement. However, this thesis research focuses on applying a structural layer coefficient to a lime stabilized subbase material. This general background on the AASHTO design procedure proves beneficial to the overall understanding of the topic.

2.5 Structural Evaluation of LSS

2.5.1 Introduction

The improvement of soil's workability, strength, swell characteristics and bearing capacity through lime stabilization are well documented (Ingles & Metcalf 1972; Little et al. 1994; Little 1995, 1999, 2000; Puppala et al. 1996; Consoli et al. 2008; Bearce et al. 2013). The strength and stiffness of clay can increase significantly when treated with lime, if the clay is reactive and the proper design and mixing protocols are followed (Solanki et al. 2009). These increases in the soil's physical properties allow the LSS to be included as a subbase for the pavement design. The strength of lime treated clay can increase in excess of 200 psi or around 223%; the resilient modulus can increase by 459% to 1,000% from its untreated value (Little 1999 & Solanki et al. 2009).

These strength characteristics of the treated soil develop immediately upon mixing and can continue to develop for a long period of time. The compaction strength of clay can even increase directly after lime mixing without any curing period (Puppala et al. 1996). The strength increase in some systems can continue in excess of 10 years as long as the pH remains high and calcium remains in the system (Little 1995). The short-term strength gain is attributed to the immediate calcium cation exchange that takes place between the lime and the negatively charged clay particles, while the long-term gain is contributed to pozzolanic reactions (Little 1995). The strength gain depends on the reactivity of the clay, which is unique based on the clay's mineralogy (Pedarla et al. 2011).

2.5.2 Evaluation of LSS and Procedures Overview

Unconfined compression strength tests are used to determine the compression strength of the LSS. The test is simple, fast, relatively cheap, and reliable. However, when LSS is being placed over a low bearing material, the LSS often fails in tension not compression (Consoli et al. 2008). Nevertheless, the UCS test is still appropriate; it has been established that LSS tensile strength ranges from 9% to 14% of the UCS strength (Ingles & Metcalf 1972 and Consoli et al. 2001).

The procedures for UCS testing on lime stabilized soils vary from state to state and agency to agency. Similar procedures with similar curing times, mix designs, samples sizes, compaction methods, and testing apparatuses should be followed. If procedures remain constant, results from different studies can be compared, which advances the understanding of lime treatment.

UCS and resilient modulus tests can be used to determine the strength and stiffness of a lime treated material. The ASTM D 5102 procedure for UCS testing on LSS is recommended by the National Lime Association (2006). This procedure recommends methods of preparing, curing and testing for laboratory compacted samples of LSS (NLA 2006). Tex-121-E is an alternative procedure for testing lime treated material. The Texas procedure includes additional lime testing recommendations beyond the UCS procedures. In a series of tests, the compaction method, sample size, optimum moisture content, and curing specifications should be clearly documented (TxDOT 2002). The AASHTO T-307 or an equivalent test can be used to estimate the resilient modulus (Mallela et al. 2004).

Standard or modified proctor can be used for compaction of the samples and determination of optimum moisture content and maximum density of the LSS. Some literature suggests standard proctor should be used for laboratory mixed samples (Solanki et al. 2009; Little 1995; Mooney & Toohey 2010). However, modified proctor provides higher UCS testing results and better reflects actual field compaction (Little

2000). This being said, the compaction procedure can be determined by each individual agency. If the testing goal is to form a resilient modulus and UCS correlation, it is essential the same compaction procedure is conducted on all samples.

Sample size or height to diameter ratio should be established and similar samples should be prepared for both UCS and resilient modulus testing. ASTM D5102-09 recommends two different height to diameter ratios: procedure A recommends between a 2.00 and 2.50 ratio and procedure B recommends a 1.15 ratio (ASTM 2009). Tex-121-E recommends between a 1.33 and 1.5 height to diameter ratio (TxDOT 2004). As long as the correct procedures are followed, the size of the sample should not affect the UCS results (Toohey et al. 2013). A height to diameter ratio should be determined at the beginning of a study and should be maintained for UCS and resilient modulus testing throughout the completion of the study.

Due to restrictions in construction, the period of curing for LSS samples must be accelerated. Typically, construction scheduling does not allow for samples to be cured for 28 days at room temperature. For this reason, samples are cured at higher than room temperatures to expedite the pozzolanic reactions.

The quantity of lime added to a soil may play a part in the gain of the engineering properties depending on the soil mineralogy. At a certain percentage of lime there can be a significant increase in UCS strength and resilient modulus as long as the mixture is compacted and allowed to cure (Bell 1996). As the percent of lime increases, the strength and stiffness at 28 days does not show significant increases (Solanki et al. 2009 & Bell 1996). However it stands to reason, at higher percentages of lime, the pozzolanic reactions may continue well past the 28 day period. That is to say, the more lime in the system, the longer the reactions will take place and the greater the gain in the engineering properties. Field data indicates that with some mixtures strength continues to increase with time up to in excess of ten years (Little 1995). However, an increase in lime content may have a prejudicial effect on the strength as shown in Bell (1996).

Determining the correct strength and stiffness parameters for the LSS is critical for the evaluation of the strength contribution to the pavement structure. However, these two parameters are not the only variables for evaluating the soundness of the LSS. Fracture and fatigue and the durability of the layer must also be considered (Mallela et al. 2004).

Fracture and fatigue properties are directly related to compression strength and can be accurately predicted using compression testing (Little 1999). The stress ratio approach can be used to evaluate the LSS's capability to resist fatigue cracking (Little 1999). This approach states that the critical flexural tensile stress induced under load should not be greater than a specific percentage of the material's flexural tensile strength (Little 1999). The durability of the LSS has been well established by evaluating the effects of water exposure, freeze-thaw, strength recovery, dielectric value measurements and long-term strength retention (Mallela et al. 2004).

Water saturation has a deleterious effect on the strength gain of the LSS (Puppala et al. 1996). It was found that UCS values of soaked samples were between 70 and 85% of the unsoaked UCS values (Thompson 1970). Little (1998) confirmed the effects of water on the strength of the LSS but determined only a 10% decrease in UCS strength from an unsoaked sample to soaked sample, as long as a large amount of the pozzolanic reaction has taken place. Puppala et al. (1996) showed that a 9.6% increase in saturation can cause the UCS of the treated material to decrease 77% to 62% after normal temperature curing periods of 0 and 3 days, respectively. This negative effect of water on the overall strength of the LSS expresses the importance of keeping water away from any LSS layer. The laboratory samples should be soaked for 24 hours following the curing period to mimic field conditions and the effects of moisture (Little 2000). This soaking provides a worst case scenario evaluation.

Directly evaluating the LSS resilient modulus in the field can be done through Falling Weight Deflectometer (FWD) testing. The FWD provides the elastic modulus of pavement layers and resilient modulus of subgrade materials through a back calculation process.

Establishing the proper testing procedures is vital if structural coefficients are to be estimated based on test results and correlations. The laboratory procedures must mimic field conditions as closely as possible in order to give accurate representation of expected in-situ parameters.

2.5.3 Resilient Modulus Estimations

Three methods can be used to estimate the resilient modulus of the LSS layer during the design phase. These methods are discussed in Consideration of Lime-Stabilized Layers in Mechanistic-Empirical Pavement Design June 2004 (Mallela et al. 2004). In the lab, proper lime/soil sample preparation should be followed in accordance with local standards and past publications to obtain estimated soil properties at 28 days field curing.

The first method to estimate resilient modulus is the AASHTO T-307 test or an equivalent laboratory test (Mallela et al. 2004). The testing apparatus is similar to a triaxial test. The test sample should contain the optimum amount of lime and be molded at optimum water content to achieve maximum density. The testing apparatus applies different levels of deviatoric stress at different confining pressures and deformations are measured. The sample is confined using a pressure chamber.

The most representative deviatoric stress is 6 psi for design purposes (Little 2000). The representative confining pressure should be based on the overburden and applied pressures from a trial design (Mallela et al. 2004). Confining pressures of 2 and 4 psi are recommended for pavement design (Bearce et al. 2013). There are several constitutive models available to calculate and predict the resilient modulus of a soil (Solanki et al. 2009). These models use regression constants to determine the resilient modulus.

During the resilient modulus laboratory test, as confining pressure and applied stress increase, the resilient modulus will increase, as seen in Figure 2.7. This figure shows the resilient modulus test from a NTE Segment 1 sample (Ahmed & Ranasinghege 2012). Laboratory tests are difficult and expensive tests to run. Many agencies do not have the resources or the time to do such a detailed investigation. Furthermore, significant variation in results from laboratory to laboratory may be seen. Consequently, it is important to have another reliable, quick and cost efficient method to determine the resilient modulus.

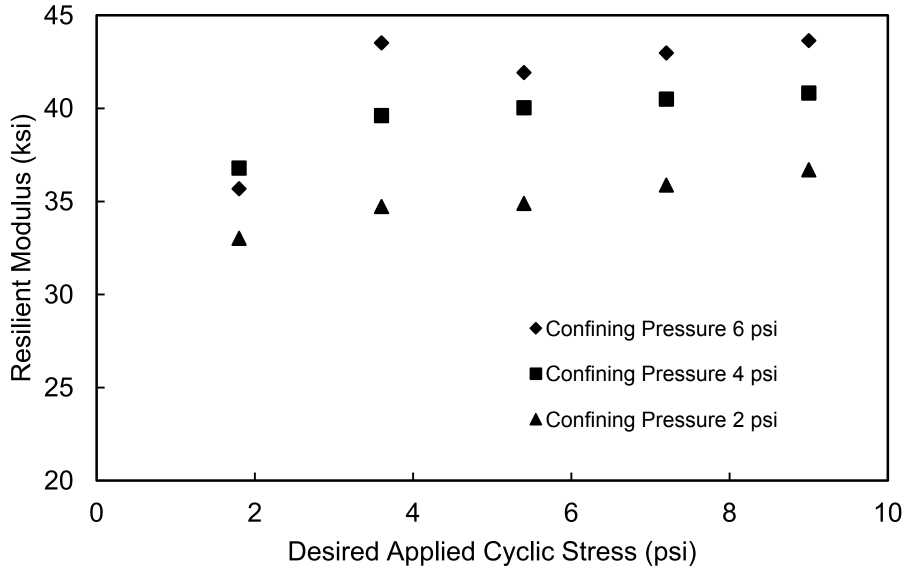


Figure 2.7 Variation in Resilient Modulus (Ahmed & Ranasinghege 2012)

The second method of estimating resilient modulus is to use correlations from well established publications. Correlations of resilient modulus have been established based on results from the California Bearing Ratio (CBR), R-value, and UCS. An improvement in stress-strain relationship along with the improvement in strength has been observed in lime treated material (Little 1999). Based on this relationship, resilient modulus can be estimated through the use of UCS testing and a correlation developed by Thompson's (1966), Equation 2.3. The 28-day unconfined compression strength is used to estimate the material's resilient modulus (Mallela et al. 2004). This provides agencies with a quick, simple, and reliable method to estimate resilient modulus.

$$M_r = 0.124(q_u) + 9.98 \tag{2.3}$$

where

M_r = resilient modulus (ksi)

q_u = unconfined compression strength (psi)

Puppala et al. (1996) provided another correlation to predict the resilient modulus based on multiple input values including the UCS strength, Equation 2.4. This equation uses UCS, dry density, degree of compaction, and moisture content to estimate the resilient modulus (Puppala et al. 1996). This relationship was formed through statistical regression analysis and only requires a few basic tests, proctor and UCS (Puppala et al. 1996). However, this equation is only valid for the soils used in that specific study (Puppala et al. 1996). In spite of this, a similar relationship could be established for soil in specific project areas, if a proper testing protocol was implemented.

$$\frac{M_r}{UCS} = 5{,}594 + 8.47 \cdot \sigma_3 + 12{,}633 \cdot DC + 69.4 \cdot w - 1{,}138.3 \cdot \gamma_d \qquad (2.4)$$

where

M_r = Resilient Modulus (kPa)

UCS = Unconfined compression strength (kPa)

σ_3 = Confining pressure (kPa)

DC = Degree of compaction

γ_d = dry density (kN/m3)

w = moisture content (%)

The Thompson correlation has been widely accepted over the past 50 years as a reliable method to estimate the resilient modulus for any clay reactive with lime. Equation 2.3 was formulated from shear strengths and secant modulus of elasticity found using unconsolidated undrained triaxial compression tests (Thompson 1966). Figure 2.8 shows a comparison of the Thompson correlation and observations from Little et al. (1994) based on three relationships: Equation 2.3, UCS strength and flexural modulus, and UCS verses FWD resilient modulus data (Little et al. 1994). The results from a study conducted to evaluate the UCS to resilient modulus relationship can also be seen in the figure (Toohey et al. 2013). In that study, UCS and resilient modulus tests were performed on samples with a height 200 mm and 100 mm diameter, cured at 28 days at room temperature, and subjected to UCS and resilient modulus testing. The results shown are the resilient modulus tests at a 6 psi deviator and 2 psi and 4 psi confining pressure.

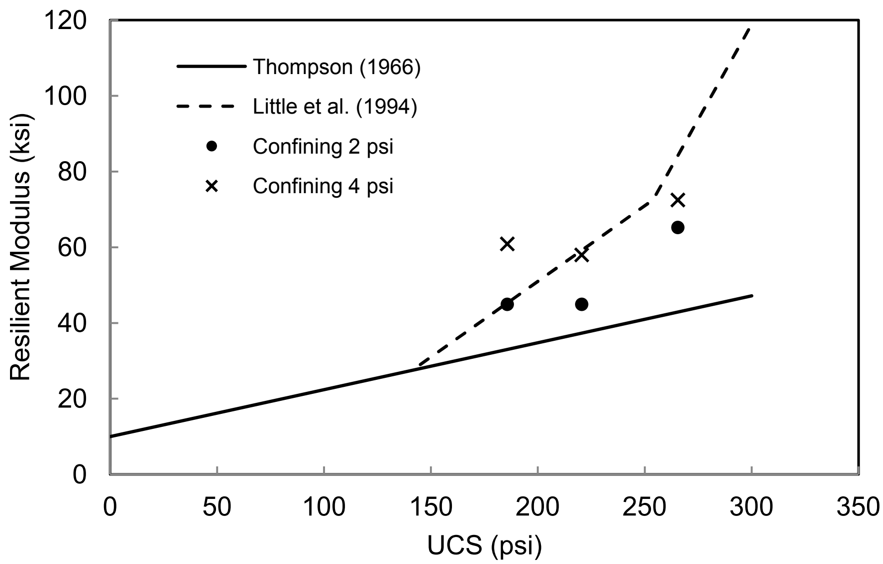


Figure 2.8 Thompson 1966 Correlation, Little 1994 Observation, and Test Results for Resilient Modulus (Toohey et al. 2013)

As can be seen in the figure, the Thompson correlation provides lower bound resilient modulus estimations. All measured points fell above the Thompson correlation yet straddled the Little observation (Toohey et al. 2013). The results also revealed significant scatter that may call in to question the general UCS to resilient modulus relationship (Toohey et al. 2013). If possible, a project should develop its own unique relationship between the project soil UCS strength and resilient modulus through testing.

The third option for estimating the resilient modulus is to use local experience or historical records. TxDOT design manual recommends a design modulus of 40 to 45 ksi for lime stabilized subgrade and 60 to 75 ksi for lime stabilized base (TxDOT 2011). Mallela et al. (2004) states a reactive soil could achieve a resilient modulus between 30 to 60 ksi. Resilient modulus results from FWD testing can range from 31 to 507 ksi (Little 1999).

Estimating resilient modulus values for a design is a complicated process. The actual resilient modulus of a material is dependent on many different variables. Moisture content, lime percentages, loading, confining stresses, compaction effort, and curing periods are just some of the variables that could affect the resilient modulus (Little 1995; Puppala et al. 1996; Solanki et al. 2009 & 2010; Mooney & Toohey 2010; Ahmed & Ranasinghege 2012; Khoury et al. 2013). Conservative values should always be considered if estimating a resilient modulus for design.

2.5.4 Summary

Lime treated soil must be constantly evaluated and tested throughout the duration of a project. The reactions between the clay and lime must mature and the pozzolanic bonds must develop. If the soil is not reactive and the strength gain does not occur, the treated soil should not be considered acceptable for pavement design. Testing procedures and correlations have been developed to estimate the engineering properties of LSS. These methods have been beneficial to the design engineer in the past and will continue to be beneficial in the future.

2.6 Soluble Sulfates

The presence of high amounts of soluble sulfates $(SO_4)^{2-}$ in a clay material can lead to significant heaving after lime treatment (Hunter 1988). Studies have shown that soils with varying degrees of sulfate content, 1,000 to 10,000 ppm, can react with a calcium based stabilizer and clay (Little et al. 2010). The reaction between the sulfates, lime and clay produce a calcium-sulfate-aluminate-hydrate. This product can range from a high amount, forming Ettringite, to a low amount, forming a monosulfoaluminate (Little 1995).

The formation of Ettringite, which are large formations of the calcium-sulfate-aluminate-hydrate, causes a heaving in the system. These formations develop in the soil voids of the LSS. Eventually, the formations grow and more reactions occur and the soil expands to accommodate the reaction. Small reactions can be accommodated in the soil voids. For this reason low amounts of sulfate in the soil are not detrimental (Little 1995).

The heaving in clay soils from Ettringite formation are more likely to be greater than the heaving in sand soils under similar chemistry and environmental conditions (Puppala et al. 2005). These sulfate-lime products from the chemical reaction can cause short-term and long-term heaving of the lime treated soil. Heaving of the soil from lime treatment can take place from days to years after completion of treatment (Puppala et al. 2005). The native soil must be tested for sulfates during the design and the construction of an LSS system.

Each agency will have its own maximum allowable level of sulfate when considering lime treatment. TxDOT recommends the soluble sulfate content should be below 3000 ppm (TxDOT 2005). In 2005, the District of Dallas only recommended lime treatment for soils with soluble sulfate content below 2000 ppm (Chen et al. 2005).

It is possible for the soluble sulfate content in a soil to change greatly within a project area. In the area of U.S. 82 and 69, sulfate contents ranged from 4,000 to 27,800 ppm (Chen et al. 2005). This is a significant change in sulfate content which led to significant heaving of the pavement. A significant amount of sulfate content testing must be done within a project area.

If a large amount of soluble sulfate is found in a project area, there are many different techniques to mitigate the heaving, which include extended mellowing periods and double lime treatment (Harris et al. 2004). Extended mellowing periods have successfully mitigated Ettringite induced heave in Texas. However, the double lime treatment method has produced inconclusive results (Puppala et al. 2005). Sulfate rich soils can also simply be removed and replaced.

2.7 Accelerated Lime Curing

Typically, the properties of a lime stabilized material are based on the 28 day strength. During the construction of a roadway, there are significant time restraints, which would not allow a 28 day curing period. Consequently, LSS samples are subjected to higher curing temperatures in the lab than would be encountered in the field. This process expedites the pozzolanic reactions between the lime and the soil (Little 1995). However, the strength gain of the LSS, assuming the same type of soil and sample preparation, is dependent on the curing temperature and time. For this reason, different methods and theories have been developed to determine the appropriate accelerated curing procedure.

It is recommended curing temperatures should not be greater than 120°F, since pozzolanic compounds could form that would not normally form in field conditions (Little 1995). Muzahim (2010) showed the amount of total pozzolanic reactions are more or less equal at the end of a 90 day curing period when compared to samples subjected to curing temperatures of 122°F and 68°F (Muzahim 2010).

There are three accelerated curing procedures, which recommend time and temperature specifications, typically followed. The National Lime Association recommends a curing period of 7 days at 104°F (NLA 2006), this curing method is based on Little (2000). The Metropolitan Government Pavement Engineers Council in Denver, CO recommend a 5 day curing period at 100°F (Mooney & Toohey 2010). The US Army recommends 2 day curing at 120°F (ARMY 1994). These procedures produce results that show strength gain with time. Table 2.3 summarizes research conducted to observe the strength gain with time under normal and 7 day accelerated conditions (Toohey et al. 2013).

Table 2.3 Equivalent 28 Day Strength and/or 7 Day Accelerated / 28 Day Normal Curing (Toohey et al. 2013)

Original Reference	Classification		LL	PI	Lime (%)	28 day strength at 106°F (days)	7 day Acc./ 28 day
	AASHTO	USCS					
Biswas and Bhupati (1972)	A-7-6	CL	44	23	4	2.4	N/A
	A-7-6	CL	49	20	5	2.9	N/A
	A-7-6	CH	64	41	5	2.2	N/A
	A-7-6	CH	65	42	5.5	2.8	N/A
	A-7-6	CH	72	40	6.5	4.0	N/A
Drake and Haliburton (1972)	A-7-6	CL	38	21	4	1.3	N/A
	A-7-7	CL	38	21	8	3.0	N/A
	A-7-6	CH	60	30	6	1.3	N/A
	A-7-7	CH	60	30	11	3.0	N/A
Toohey, Mooney, and Bearce (2013)	A-6	CL	39	13	6	5.4	1.25
	A-6	CL	39	24	6	4.6	1.27
	A-7-6	CL	41	26	6	5.9	1.13
	A-7-6	CH	55	37	6	1.8	2.56
Alexander (1978)	A-6	CL	34	14	7	N/A	1.13
	A-6	CL	36	11	7	N/A	2.18
	A-7-5	CH	56	30	7	N/A	1.71
	A-2-4	SM	16	NP	7	N/A	1.23
	A-7-6	CH	52	30	7	N/A	1.92
	A-7-5	MH	51	15	7	N/A	0.93
	A-4	ML	24	7	7	N/A	1.35
	A-6	CL	33	14	7	N/A	1.23
	A-4	ML	31	7	7	N/A	1.90
	A-7-5	CL	41	22	7	N/A	1.87
	A-4	CL	25	10	7	N/A	1.60
	A-7-5	MH	50	22	7	N/A	1.51

Based on Table 2.3 and the average of results comparing the 28 day strength to the 7 day strength, the strength of a LSS sample cured for a 3.1 day period at 100°F to 106°F correlate closely to a 28 day normal curing strength. Therefore, the results suggest a curing period of 7 days at 104°F tends to overestimate the 28 day UCS strength. This strength to time relationship varies from soil to soil, as seen in the table.

Based on these results, a curing period of 5 days at a temperature of 100°F to 106°F is recommended to estimate the 28 day normal curing strength.

2.8 Summary

The literature summarized in this chapter should be part of any process to formulate and evaluate proper design, construction, QC, and QA procedures. This information presented in this chapter is summarized as follows:

- Mineralogy of clay affects the expansivity of a soil and its reactivity with lime.
- Tex-124-E is an effective yet conservative method to estimate the PVR in an area and can be used to establish a treatment depth.
- Flexible pavement design is an iterative process with many different variables.
- Lime treatment improves the strength and stiffness of a clay material.
- The Thompson correlation provides a conservative estimation of LSS resilient modulus from UCS values.
- The presence of sulfates in a soil should be reviewed and considered before lime treatment commences.
- Accelerated curing of lime/soil samples can be used to expedite the evaluation of 28 day field curing strength.

Chapter 3

Material Testing and Procedures

3.1 Introduction

Throughout the design and QA programs for the NTE Segment 1, many laboratory soil tests were conducted to classify the project soil and determine its properties. In addition to soil classification tests, strength tests were performed on natural soil and lime treated soil. In some locations, FWD tests were carried out in the field after construction of the LSS layers. The different testing procedures, as pertains to the soil before and after lime treatment, are summarized in this section.

3.2 Natural Soil Testing

3.2.1 Atterberg Limit Tests

As per standard practice, the Atterberg Limit tests served as the main classification method for the fined grained material encountered in the project area. These tests were developed by Albert Atterberg and refined by Arthur Casagrande. The ASTM D 4318 served as the standard and provided the procedure to be followed.

Two values are determined from the results of these tests, plastic limit (PL) and liquid limit (LL). The PL of a soil is the percent moisture content, weight of water to the weight of solids ratio in a soil sample, needed by the soil to change from a semisolid to a plastic state. The LL is the percent moisture content required by the soil to change from a plastic state to a liquid state. The third very useful value is the plasticity index (PI), which is the difference between the PL and LL. The PI of the soil assists the engineer in determining the possible activity the soil could express, and allows the engineer to estimate the possible swell potential. The LL and PL both have specific tests and procedures that must be followed. The ASTM D 4318 provides very detailed instructions the laboratory technician must follow during the testing procedures.

The results from the Atterberg limit tests are fundamental to the Casagrande plasticity chart. The Atterberg limit values in conjunction with the chart are used to classify soils using the Unified Soil Classification System. The engineer determines the type of soil and the plasticity of the soil, and uses this information to determine a strategy for design.

3.2.2 Sieve Analysis

When determining the expansive properties of a soil, the percent passing the number 40 (0.425 mm opening) sieve is a valuable piece of information, as it assists in estimating the expected amount of swell potential. The percent passing the number 200 sieve (0.075 mm opening) helps predict whether the soil will react with lime. These values can be determined using procedures conducted according to the ASTM D 6913.

The procedure requires oven dried soil of known weight to be placed in a sieve apparatus, which consists of stacked sieves of decreasing internal opening sizes. This apparatus is shaken and the weight of material retained on each soil is measured. This process gives a particle size distribution curve, which assists the engineer in identifying the type of soil and possible soil properties.

3.2.3 Pressure Swell

Pressure swell tests measure the potential swell of a soil sample as the soil's moisture content increases. This measurement is expressed it terms of a swell index (C_s). ASTM D 4546 – 08 B can be used to measure the soil's swell index.

This test measures the one dimension wetting induced swell of a soil. The soil is loaded to the expected in-situ vertical pressure. Following the initial loading, the sample is inundated. The change in strain, caused by the wetting, is measured; this value is used to determine the swell index. The higher the swell index, the greater the potential swell for a soil.

3.2.4 Soluble Sulfates

As mentioned previously, measuring the amount of sulfates in the natural soil helps estimate the possibility of Ettringite formation, which inhibits the soil and lime mixture from gaining meaningful strength and may lead to pavement heaving in the future. The amount of sulfates in the soil can be measured using the Tex-145-E method. This method uses turbidimetric techniques to determine the amount of soluble sulfate. The turbidimetric technique observes and measures the cloudiness of a liquid and uses the value to determine concentration. Results are provided in parts per million (PPM).

3.2.5 Unconfined Compression Strength

UCS testing was conducted on the natural soil to gauge the existing soil strength before lime stabilization. UCS testing allows for a simple and quick method to determine compression strength, which relates well to the Unconsolidated Undrained (UU) triaxial test results. ASTM D 2166 method was used to run the UCS testing.

The undrained shear strength of a fine grained material is determined by taking one half of the compressive stress at failure. Tested samples should be collected with minimal disturbance from the field. This test proves important in determining the reactivity of the soil after treatment with lime. The lime treated soil should exhibit a significant increase in soil strength. This compression strength can be compared with the native soil compression strength to confirm a beneficial reaction between the lime and clay.

3.3 Lime Stabilized Soil Testing

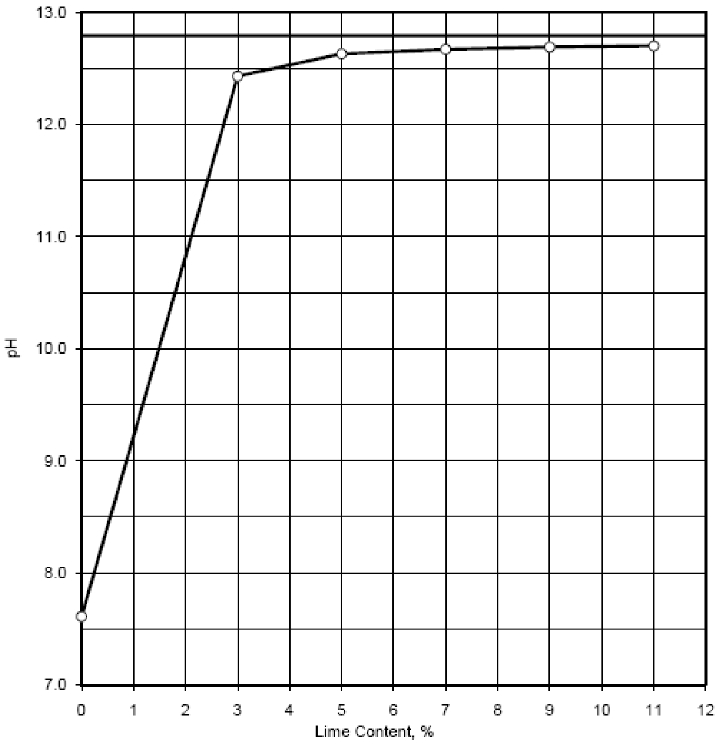
3.3.1 pH series

Eades and Grim (1966) developed a procedure to determine minimum required lime content. The procedure is based on the belief of adding sufficient lime to satisfy initial cation exchange reaction and still provide enough lime for the pH to remain high and induce pozzolanic reactions (Little 1995). The measurement of pH in the system becomes the focal point of the procedure.

Different adaptations have been made to the original process over the years. According to ASTM D 6276-99a, a series of specimens are prepared containing a range of percentages of lime content, with measurements of the pH levels in each specimen taken and compared. The minimum lime content required to achieve a pH value of 12.4 should be the lime content considered for the design. However, a pH of 12.4 is not always achieved. According to the ASTM 6276-99a:

> "If the highest measured pH is 12.3 or less then additional test samples using higher percentages of lime should be prepared and tested. If the highest measure pH is 12.3 and at least two successive specimens at increasing lime percentages yield values of 12.3, the lowest percentage of lime to give a pH of 12.3 is the approximate optimum lime percentage for stabilizing the soil. If the highest measured pH is less than 12.3, the test is invalid due to equipment or material error due to insufficient lime having been added (ASTM 2006)."

The ASTM D 6276-99a or Tex-121-E part III procedures can be followed for this test. Figure 3.1 shows the results of one test run in the NTE Segment 1. Choosing the lime content percent according to the Eades and Grim test is the first step in an effective lime stabilization design. This provides the engineer with a starting percentage, though further testing of the LSS properties is recommended.



Boring No. Bulk Sample 5
Sample Depth: 1-5
Sample Description: Dark Brown Clay
Liquid Limit: 65 %
Plastic Limit: 26 %
Plasticity Index: 39
- #200 Sieve 91 %

Lime	pH
0	7.61
3	12.43
5	12.63
7	12.67
9	12.69
11	12.7

Figure 3.1 Results from Eades and Grim Test (Ahmed & Ranasinghege 2012)

3.3.2 UCS Testing on Lime Treated material

The lime treated samples during the design and quality phases were tested in accordance with a modified version Tex-121-E part I and II. Samples were compacted using standard or modified proctor after being properly mixed with different percentages of lime. Standard proctor was conducted on tests during the design, according to ASTM D 698, and the modified proctor was conducted on tests during the construction, according to ASTM D 1557.

During the geotechnical investigation phase, 2 initial samples were prepared at the minimum required lime content, determined during the Eades and Grim test. In addition to these 2 samples, 2 other samples were prepared at 1 lime percent above optimum and 1 lime percent below optimum. After standard proctor compaction, the samples were cured for 5 days in an oven at 100°F and tested after the 5 day curing period. Compression strength for a sample cured at this temperature and for this time period should correspond roughly to the 28 day field compression strength. The 5 day curing period at 100°F is recommended by the Metropolitan Government Pavement Engineers Council. This testing, in the design phase, decides the percentage of optimum lime content for construction.

In the QA phase of the project, samples were mixed with the optimum lime percentage in the field and compacted, using the modified proctor, in the laboratory. These samples were molded to have a 4” diameter and be 6” tall. After molding, the samples were cured for 3 days, 7 days, and 15 days at 104°F. After curing, the samples were subjected to 24 hour capillary soaking, in an attempt to simulate field conditions. Following the soaking period, the samples were compressed until failure occurred. These UCS values served as a method to evaluate the resilient modulus of the LSS according to the Thompson correlation.

3.3.3 Resilient Modulus on Lime Treated Material

Resilient modulus tests were conducted in accordance with the AASHTO T 307-99 standard. Samples were cured in an accelerated fashion to simulate soil properties at 28 days field curing. This test estimates the resilient modulus of the material, which is the elastic modulus of a material but estimated from recoverable strain while cyclically loaded. Samples are loaded under confined stress conditions and then different deviator loads are applied cyclically for a period of time. After the cycle of loadings has completed, the deviator stress is increased and another cycle of loadings begin. This sequence is done at three different confining pressures: 2 psi, 4 psi, and 6 psi. Table 3.1 shows the recommended testing sequence.

Table 3.1 Testing Sequence for Resilient Modulus (AASHTO 2003)

Sequence No.	Confining Pressure	Max. Axial Stress	Cyclic Stress	Constant Stress	No. of Load Applications
	psi	psi	psi	psi	
0	6	4	3.6	0.4	500 - 1000
1	6	2	1.8	0.2	100
2	6	4	3.6	0.4	100
3	6	6	5.4	0.6	100
4	6	8	7.2	0.8	100
5	6	10	9.0	1.0	100
6	4	2	1.8	0.2	100
7	4	4	3.6	0.4	100
8	4	6	5.4	0.6	100
9	4	8	7.2	0.8	100
10	4	10	9.0	1.0	100
11	2	2	1.8	0.2	100
12	2	4	3.6	0.4	100
13	2	6	5.4	0.6	100
14	2	8	7.2	0.8	100
15	2	10	9.0	1.0	100

The results from these tests allow the engineer to evaluate the stiffness of the lime treated material. The resilient modulus of the LSS can be compared to the estimated untreated clay resilient modulus. This comparison helps evaluate the soil's reactivity with lime. If proper procedures are followed, these results can be compared to UCS testing results and UCS and resilient modulus correlations for the project soil can be formed.

3.4 Field Testing

3.4.1 Falling Weight Deflectometer (FWD)

The FWD is a non-destructive test used to estimate the resilient modulus of a pavement layer. The FWD is a device that drops a set of weights on a pavement layer surface and measures the deflection of the pavement with a set of geophones at different distances from the drop point. A typical distance between the geophones is 12 inches. Each test consists of three drops, the first drop is to settle the load plate and the next two drops are used to determine maximum deflection.

These deflection measurements are used in a complex back calculation process, which can be used to estimate the resilient modulus of different pavement layers. Pavement thicknesses must be known in order to measure the different modulus of the layers and the modulus of the layers must be known in order to determine layer thicknesses.

In the NTE Segment 1 project direct testing on the lime stabilized layer was conducted. Three impacts of 6,500 plf and 3 impacts of 9,000 plf were performed during testing. These forces along with the measured deflection were used to calculate the surface modulus. The Boussinesq formula for surface modulus, Equation 3.1, was used to determine the LSS modulus.

$$E_0 = \frac{2 \cdot (1 - \nu^2) \cdot \sigma \cdot a}{\delta_x} \tag{3.1}$$

where

E_0 = surface modulus (psi)

v = Poisson's ratio

σ = pressure of FWD impact load under loading plate (psi)

a = radius of loading plate (in)

δ_x = center deflection (in)

3.5 Summary

Properly following test procedures is extremely important in any type of engineering. These procedures have been reviewed, evaluated, and deemed as acceptable methods to replicate natural soil behavior in a laboratory setting, which is a very difficult task. This information aids the engineer in developing an efficient and detailed design.

Chapter 4

Geotechnical Investigation

4.1 Introduction

The NTE Segment 1 design team had to consider expansive clay while determining the flexible pavement design. The most practical method for dealing with the expansive clay was lime treatment, which was common practice in the area. An extensive geotechnical investigation was conducted through sampling and testing, which was used to identify the physical properties of the project soil. In regards to pavement design and ground improvement, Atterberg limit tests, swell tests, UCS, sulfate tests, and sieve analysis were completed to aid in the design process.

Lime series tests were also carried out on 8 samples of project soil collected along the alignment. The results from these tests would give an indication of the soil's reactivity with lime. These tests included pH series (originally developed by Eades and Grim), PI series, UCS, standard proctor, and resilient modulus.

The test results obtained from the lime series testing, during the GI, clearly indicated that the clay gained significant strength and stiffness following treatment. These results validated the assumed LSS structural contribution to the pavement structure.

The GI for the NTE Segment 1 began in April 2010 and terminated in late 2010. Over the period of 6 months more than 330 boreholes were drilled with continuous hollow-stem sampling, totaling over 13,000 linear feet of drilling, including soil and rock. The initial layout of the boreholes followed the direction of TxDOT *Geotechnical Manual August 2006*. This chapter summarizes the results of the GI which were used in the design process.

4.2 <u>GI Testing and Results</u>

4.2.1 Atterberg Limits

Perhaps one of the most important series of tests conducted on the native soil was the Atterberg limit tests, which gives the engineer an idea of the type of clay in the area and the clay's plasticity. Over 530 Atterberg limit tests were conducted on the different soil samples obtained from the project area, the results of which are summarized in Figure 4.1. Values that fall on the right side of the vertical, dashed line at LL equal to 50 are typically highly plastic (fat clays) and values to the left of this line express low to medium plasticity (lean clays). Both fat and lean clays were found in the project area. It was discovered that 95.9% of the Atterberg tests showed clay with a PI greater than 15. This suggests, pending sieve analysis, that the majority of the soil would be reactive with lime.

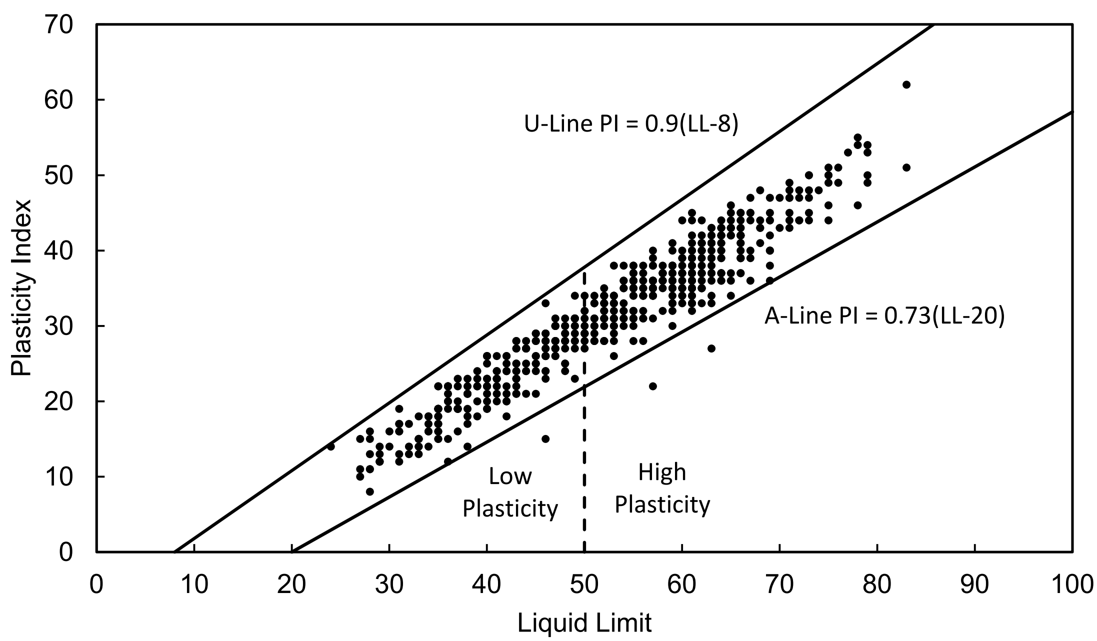


Figure 4.1 Casagrande's Plasticity Chart for Segment 1 of NTE

According to the plasticity chart, the soil in the project location was clay with varying plasticity. The variation of plasticity along the alignment would affect the required depth of lime treatment. Figure 4.2 and Figure 4.3 show the PI values along the I-35 and I-820 alignment with geological formations limits provided. It is common for these values to vary according to geologic formation. However, the PI values in this area seemed to vary within the different formations sporadically. A slight decrease in the plasticity can be seen as the project moves farther east. The inconsistent variation in PI hindered the determination of an optimized construction thickness for the LSS. In this case, conservative assumptions of plasticity were required to designate varying LSS thicknesses from section to section.

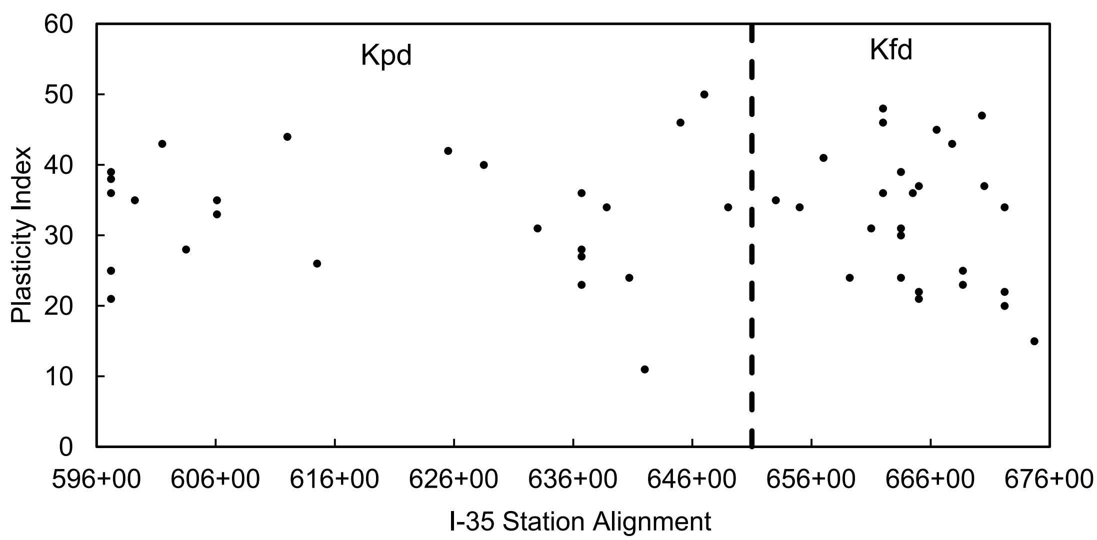


Figure 4.2 PI Values along the I-35 Alignment

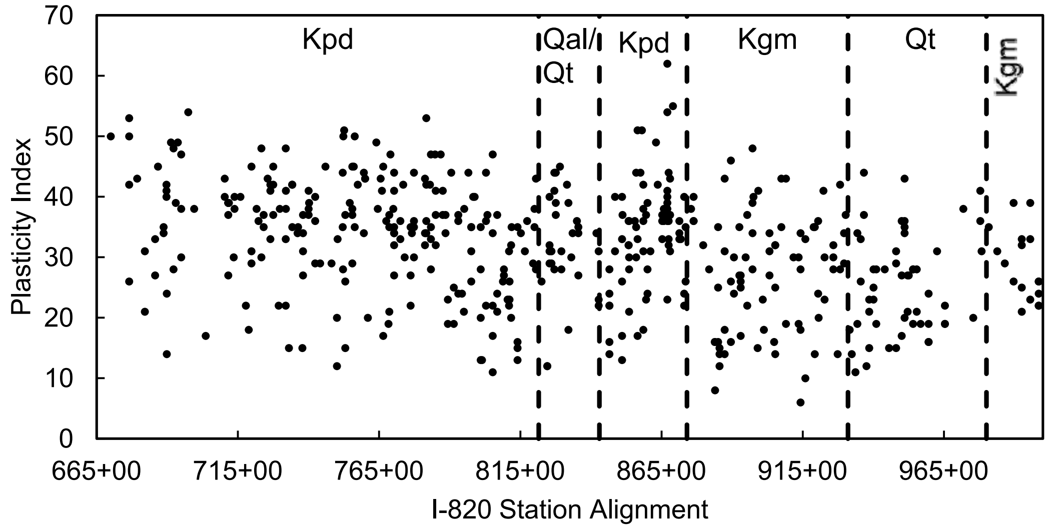


Figure 4.3 PI Values along the I-820 Alignment

According to the Mitchell 1976 and Skempton 1953 mineralogy classification presented in Das (2010), Table 4.1, and the results from a statistical analysis of the PL and LL of the clay, Figure 4.4, the clay in the project area likely consisted of Kaolinite but with high liquid limit values. The high liquid limit values increased the likelihood of the material having expansive qualities.

Table 4.1 Mitchell 1976 and Skempton 1953 Mineralogy (Das 2010)

Mineral	**LL**	**PL**	**Activity**
Kaolinite	35-100	20-40	0.3-0.5
Illite	60-120	35-60	0.5-1.2
Montmorillonite	100-900	50-100	1.5-7.0
Halloysite (hydrated)	50-70	40-60	0.1-0.2
Halloysite (dehydrated)	40-55	30-45	0.4-0.6
Attapulgite	150-250	100-125	0.4-1.3
Allophane	200-250	120-150	0.4-1.3

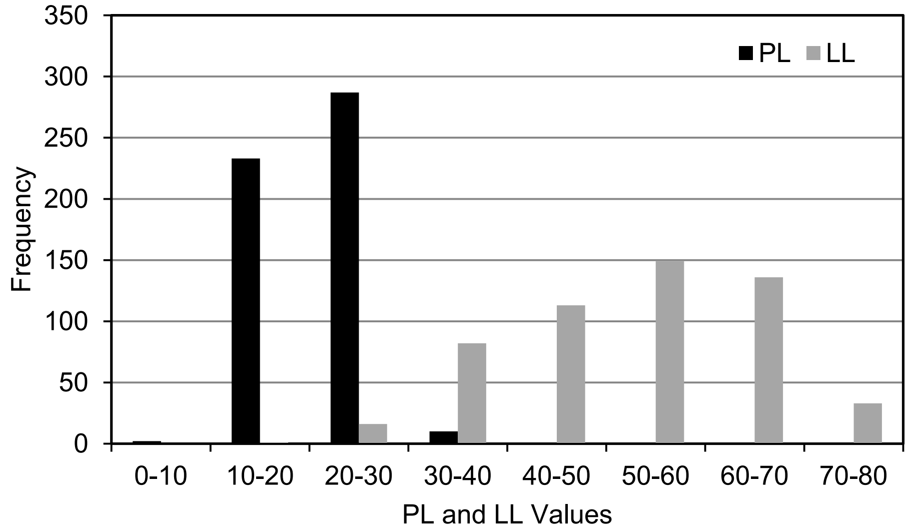


Figure 4.4 Histogram of PL and LL in Project Area

Typically Kaolinite minerals have relatively strong intermolecular bonds, low plasticity, and low activity when compared to other clay minerals. However, in this area, the clay PI values are quite large. This may suggest significant amounts of montmorillonite could be present in the soil. Unfortunately, no mineralogy tests were conducted on the soil and the clay mineralogy can only be speculated. In order to further evaluate possible swell potential, swell tests were conducted.

4.2.2 Swell Tests

Swell tests were used to identify soil swell potential. Based solely on PI values and local experience, the clay was assumed to have expansive characteristics. Swell tests were conducted to verify that assumption. Figure 4.5 summarizes the swell indexes found along the I-820 alignment. Figure 4.6 expresses the relationship between swell index and PI.

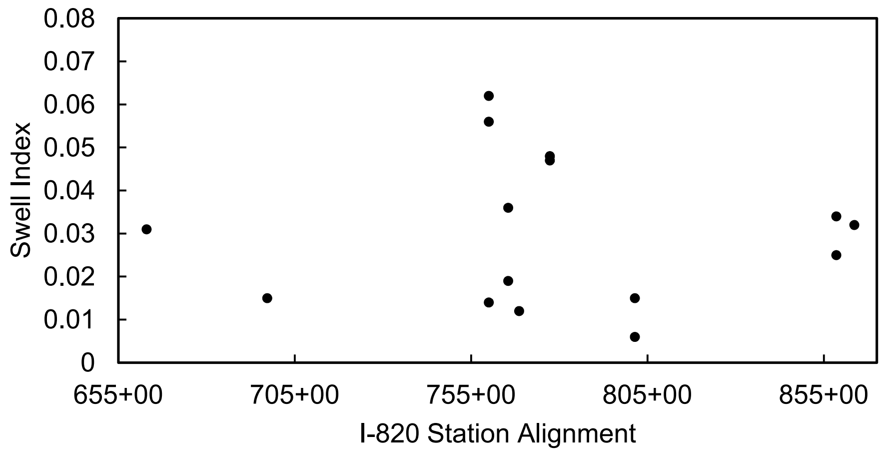


Figure 4.5 Swell Test Results along I-820 Alignment

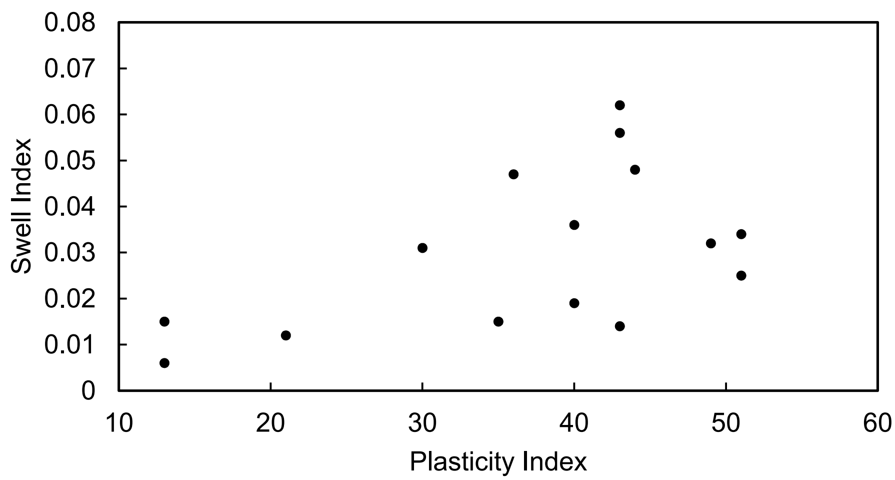


Figure 4.6 Swell Index Relationship with PI

Typically, as the PI increases, the swell index of the soil should increase. However, as shown in Figure 4.6 that relationship is not very clear. The swell index for some samples remained low despite an increase in PI. This may be due to varying percentages of montmorillonite and kaolinite minerals. The samples with low indexes could consist mainly of the kaolinite mineral; whereas the samples with high indexes could have a high percentage of montmorillonite. This would explain the inconsistency of swell with PI.

Based on the overall results from the swell tests, it was verified that pockets of soil in the area had highly expansive properties. These results confirmed the requirement of lime modification. Following the confirmation of swell potential, the level of reactivity between the native clay and different lime percentages was investigated.

4.2.3 Soluble Sulfate Testing

Soluble sulfates can interfere with the chemical reactions between the clay and lime. If the amount of sulfate in the soil is high enough, the sulfate can react with the lime and clay particles and cause significant heaving. This possibility of this potential heaving may require extended mellowing periods or alternative ground improvement methods.

During the GI, the project area was tested for sulfate contents. In a few locations, the amount of sulfates was found to very high. However, these areas were scarce throughout the project. The overwhelming majority of the tests found a sulfate content less than 3,000 ppm. This level of sulfates was deemed acceptable and required no alterations to the design or standard construction process. As a precaution, additional sulfate tests would be conducted during construction of the LSS layer. If high sulfates were found during construction, the soil in the area would be excavated and replaced with suitable material.

4.2.4 Sieve Analysis

Percent passing the number 40 and number 200 sieves are required pieces of information when designing pavement over expansive soil. The percent passing No. 40 is used in the evaluation of potential swell. A greater percent passing the number 40 sieve will lead to a larger PVR calculation. The average percent of soil passing the number 40 sieve was determined to be 83 percent.

Percent passing the No. 200 sieve and the soil PI allow the engineer to estimate the soil's reactivity with lime. A percent passing the no. 200 sieve and minimum PI of 15 suggests clay is reactive (TxDOT 2005). As graphically represented in Figure 4.2 and Figure 4.3, the majority of the soil had a PI greater than 15; 92 percent of the sieve analysis conducted showed 25% or greater passing the no. 200 sieve.

4.2.5 pH and PI Series

Eight different sets of bulk samples were collected along the I-35W and I-820 alignment. Unfortunately, the records for the exact origin of these could not be found. However, it is known, that these samples were collected at equivalent intervals along the project alignment. These samples served as a representation of the soil within the project limits. The samples were taken to the lab and mixed with different lime percent contents of hydrated lime.

The initial reactions between lime and clay caused the plasticity of the clay to reduce, typically by increasing the PL and decreasing the LL. The reduction of plasticity was evaluated by conducting PI series testing. The PI of the lime treated material was measured at different lime content percentages. The results of the PI series testing can be seen in Figure 4.7. As lime is added to the natural clay the PI values reduce significantly. Even small amounts of lime cause the plasticity of the clay to reduce significantly.

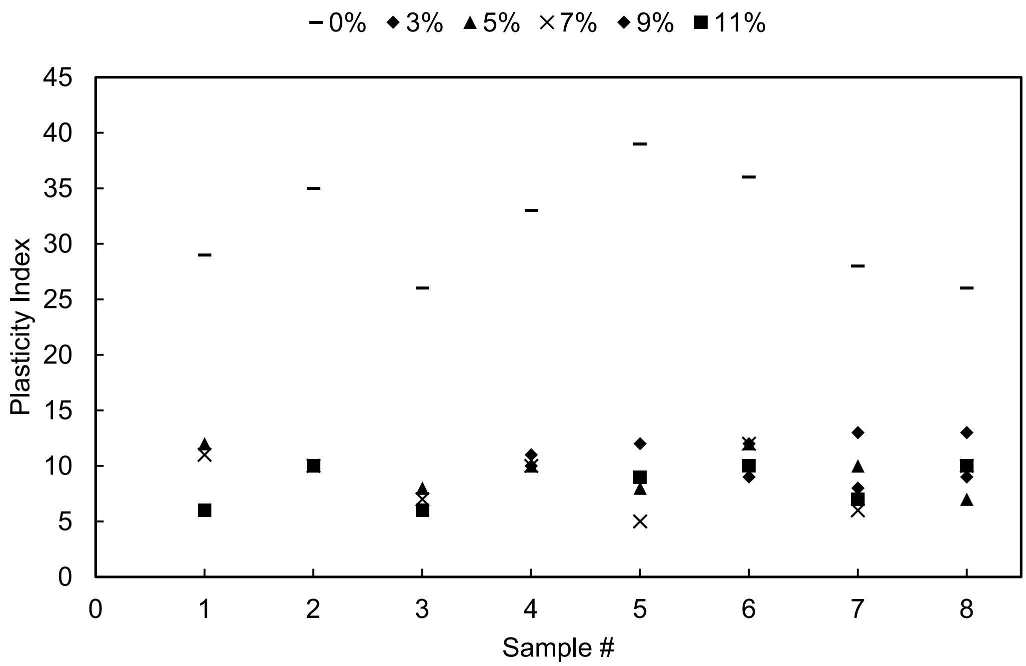


Figure 4.7 PI Series Results

Identifying the required minimum amount of lime for pozzolanic reactions to occur was estimated by pH testing. The addition of lime to the clay causes an increase in pH; the ideal pH value for the mixture is 12.4. However, at times the pH did not reach this value no matter the amount of lime added to the system. In these situations, the lime content that expressed a minimum change in pH was assumed to be the minimal lime content. This content was amount required to modify the clay and create long-term pozzolanic bonding.

Figure 4.8 shows the results of the pH tests run on the 8 different bulk samples. Different lime contents were added to the clay and the pH of each lime percent was measured according to the specified test procedure. Based on the results of these tests, a preliminary minimum percent lime was determined to be 6%.

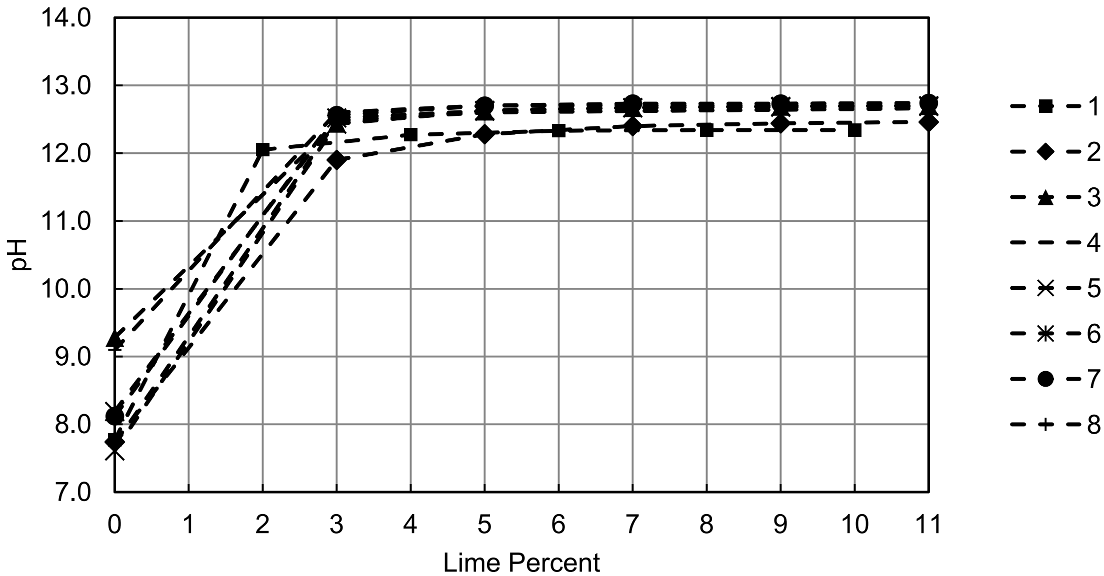


Figure 4.8 pH Values with Lime Percent Added

4.2.6 UCS Testing on Native Clay

Unconfined compression testing can give the engineer a good idea of the physical strength of the soil. UCS testing was conducted on the native soil during the GI. The interpreted results excluding any outliers can be seen in Figure 4.9 and Figure 4.10. The average compression strength of the clay material in the project was around 40 psi or 5760 psf, which corresponds to undrained shear strengths of 20 psi or 2880 psf.

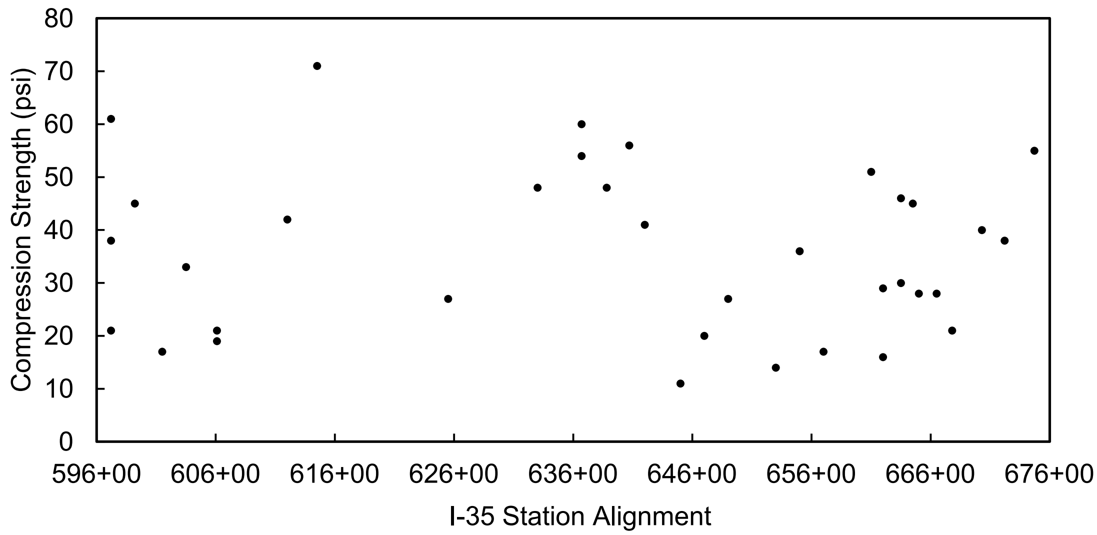


Figure 4.9 UCS Testing on Native Soil along I-35 Alignment

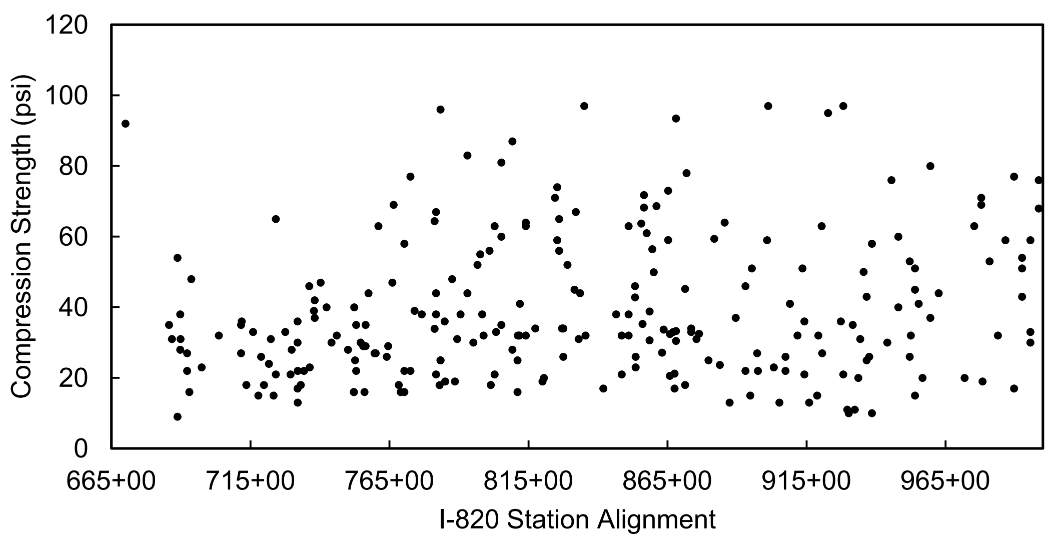


Figure 4.10 UCS Testing on Native Soil along I-820 Alignment

4.2.7 UCS Testing on Lime Treated Clay

Each of the eight different bulk samples were subjected to UCS testing after being mixed with the preliminary 6% minimum design lime content, 2 percent plus and 2 percent minus the preliminary minimum lime content. These samples were compacted using standard proctor compaction and cured in the oven at 100°F for 5 days. This method of curing generally underestimates the 28-day strength (Mooney & Toohey 2010). The results of these tests can be seen in Figure 4.11. Based on these results and the type of accelerated curing, a lime percent of 6% was deemed acceptable for the design of the LSS layer for the NTE Segment 1. However, a closer look at the Figure 4.11 shows that samples mixed with a higher percentage of lime have lower strength results. This fact made it essential that the lime was mixed appropriately in the field. If too much lime was mixed into the soil, the strength gain of the mixture may not reach the desired level.

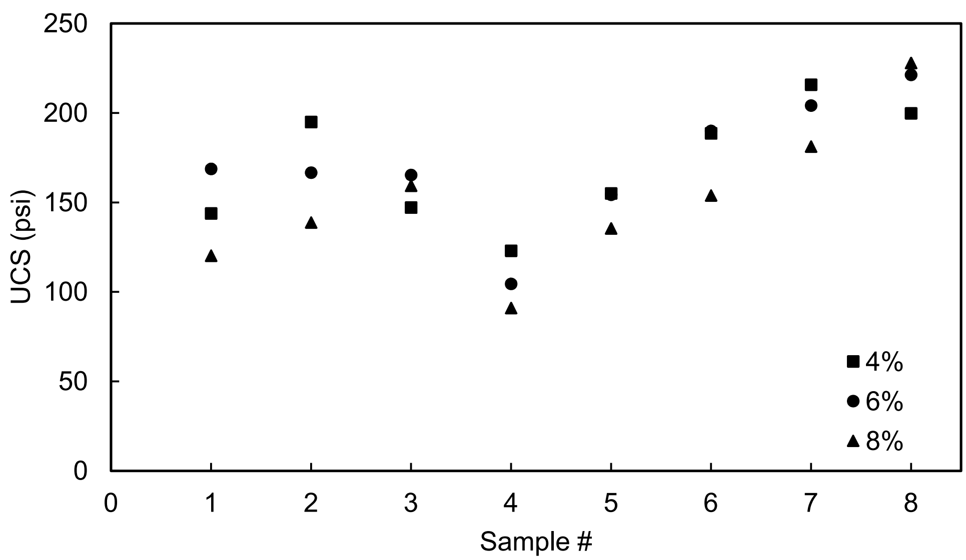


Figure 4.11 UCS Testing with Lime Percent

Sample 4 shows lower UCS test results than the other samples. Nevertheless, this value was greater than the natural soil's UCS value, 160% greater. In fact, it also must be mentioned that the 5 day accelerated curing typically provide conservative 28-day compression strength (Mooney & Toohey 2010). The actual 28-day strength for these samples could be higher after curing in the field for 28 days. Nonetheless, the average UCS strengths of the soil after treatment with 6% lime was 175 psi, which was 335% greater than the untreated clay average compression strength of 40 psi.

4.2.8 Resilient Modulus on Lime Treated Clay

In order to further confirm the clay's reactivity with lime and that the required pozzolanic reactions would occur, resilient modulus testing was conducted on the same 8 bulk samples. Unfortunately, the information on the accelerated curing periods for these tests was not available. However, based on common practice and how the results correlated with published literature, it was assumed that the curing process matched that of the UCS testing. The clay samples for these tests were mixed with 7% lime.

Resilient modulus results at 2 and 4 psi confining pressure and deviator pressure of 6 psi are considered representative to the field resilient modulus for pavement design (Little 2000). Figure 4.12 expresses the results of the resilient modulus testing. These results show the lime treated soil achieved the required 25,000 psi required to be assigned a structural number of 0.12.

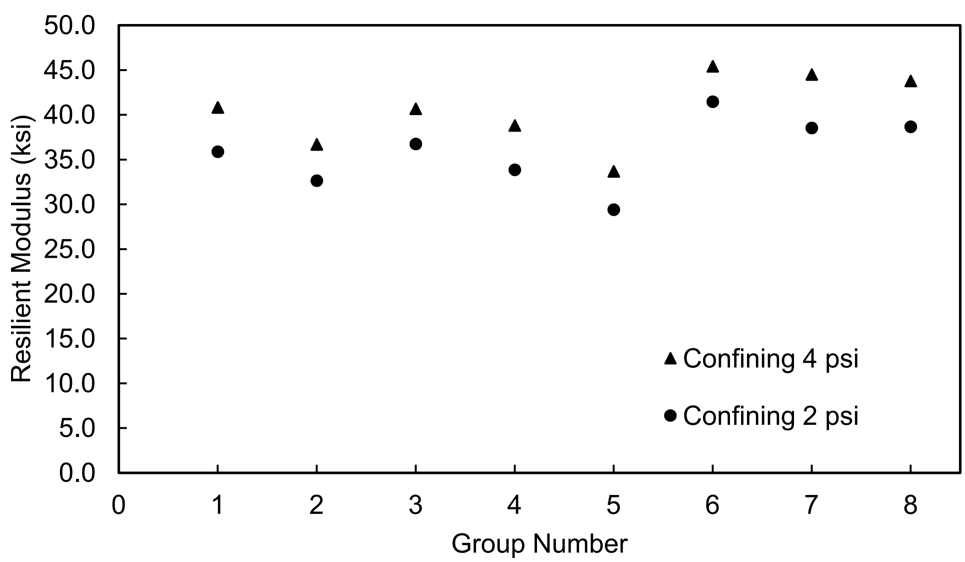


Figure 4.12 Resilient Modulus Results for Bulk Samples

Since UCS and resilient modulus testing was conducted on the same samples of soil and those samples were subjected to the same curing periods, the test results can be used to review the accuracy of the Thompson correlation. Given that the resilient modulus samples were prepared at 7% lime, the interpolated UCS values between 6% and 8% lime content were used in this comparison. As seen in Figure 4.13, the Thompson (1966) correlation serves as a lower boundary, which was stated by Bearce et al. (2013). The observation from Little et al. (1994) splits the test data; this would be an aggressive estimate of resilient modulus from UCS. This observation validates the correlation of UCS strength to resilient modulus for the project clay.

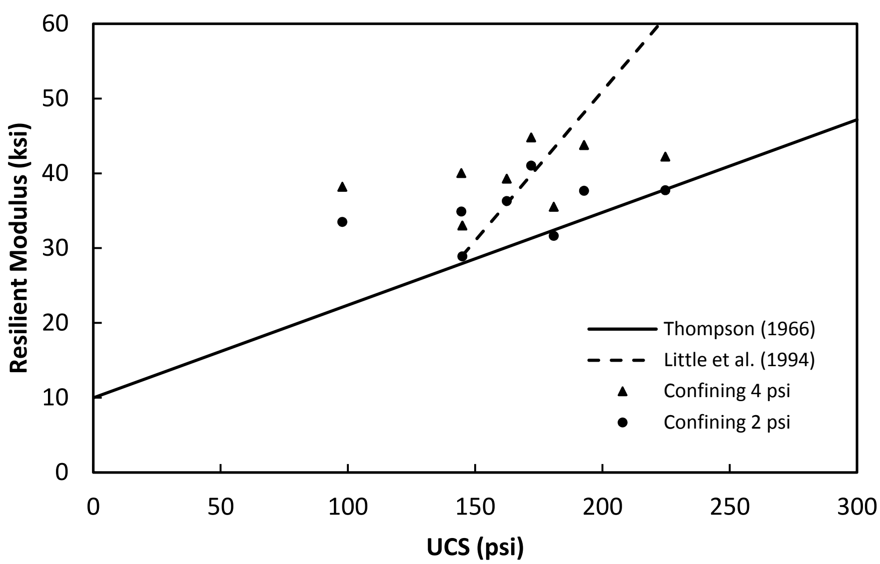


Figure 4.13 UCS and Resilient Modulus with Thompson Correlation and Little et al. Observation

4.3 Summary

The geotechnical investigation program confirmed what local experience had suggested, the material in the project location varied from highly to medium expansive clay. In addition to identifying the expansivity of the clay, testing established that no significant sulfate contents were present. The minimum optimum lime content of 6% was determined to effectively react with the in-situ clay material. The UCS and resilient modulus testing confirmed the required strength and stiffness would be achieved after lime treatment and curing.

Chapter 5

Lime Stabilized Soil and Pavement Design

5.1 Introduction

Based on the results of the GI, the design for the pavement and LSS layer was formulated. During the design, the thicknesses of the LSS structural layer and remaining pavement layers were designed based on provided traffic data by the client and according to the AASHTO 1993 methodology. LSS thickness increased in most locations due to the expansivity of the clay. The LSS was designed to reduce the predicted PVR values, in addition to its structural contribution.

5.2 LSS Design

The LSS layer served two functions: reduce the predicted PVR in the area, and part or all of the LSS thickness must serve as a sub-base for the flexible pavement. In some locations, the LSS thickness was no greater than the thickness required for pavement design. This occurred where the PVR in a given area was negated, solely, by the overall pavement thickness, which included the structural LSS. However, due to the high plasticity of the soils in the NTE Segment 1, this situation did not occur often.

Tex-124-E was used to estimate the expected PVR in the NTE Segment 1. During this process, a statistical analysis of the data gathered from the GI phase was used to determine the input parameters required by the procedure. Percentiles and averages of the PI, LL, and the percent passing #40 sieve values were used to predict the PVR along the alignment.

The alignment was segregated into different sections, with section limits based on PI values; areas appearing to have similar PI values were grouped in the same section. The PI 75th percentile value of each section was used in the PVR analysis for the entire active zone. The soil was also assumed to be in a dry state, which was a conservative assumption. A 93% passing was assumed for the percent passing #40 criteria, which was greater than the 83% average found during the GI phase. During the design process, a maximum active zone of 10 feet was assumed in the area, which was based on engineering judgment.

TxDOT recommends PVR for any ML or GPL should not be greater than 1.5” and PVR for any FR should be less than 2.0” (TxDOT 2011). If material was brought to the project for fill locations, a statistical analysis was completed based on the PI from the new material and the in-situ soils. The PVR was then re-evaluated and the appropriate PVR was chosen for the area.

Table 5.1, Table 5.2, and Table 5.3 summarize the design values used in the PVR analysis, the calculated PVR, and the treatment thicknesses required to reduce the PVR to an acceptable level. The pavement thickness was considered to be part of the treatment thickness. In some areas, the expected active zone was less than 10 feet due to a shallow rock table. This can be noted in the low PVR values found in the following tables at stations 758+00 to 768+00, 792+00 to 812+00, and 846+00 to 858+00.

Table 5.1 GPL PVR Design Data and Results (Ahmed & Ranasinghege 2012)

General Purpose Lanes							
Section	Subsection		LL	PI	% Passing #40	PVR (in)	Treatment Thick. (ft)
	Begin	End					
I-35W Northbound	590+00	670+00	60	42	93	3.77	4.0
I-35W Southbound	590+00	670+00	64	48	93	4.65	5.0
I-820	670+00	720+00	64	48	93	4.65	5.0
	720+00	758+00	64	48	93	4.65	5.0
	758+00	768+00	62	40	93	2.11	1.0
	768+00	792+00	62	40	93	3.51	3.5
	792+00	812+00	62	40	93	2.09	0.9
	812+00	846+00	62	46	93	4.33	4.6
	846+00	858+00	62	38	93	1.96	0.7
	858+00	920+00	60	35	93	2.92	2.8
	920+00	955+00	62	40	93	3.47	3.7

Table 5.2 ML PVR Design Data and Results (Ahmed & Ranasinghege 2012)

Managed Lanes							
Section	Subsection		LL	PI	% Passing #40	PVR (in)	Treatment Thick. (ft)
	Begin	End					
I-35W Northbound	590+00	670+00	60	42	93	3.77	4.0
I-35W Southbound	590+00	670+00	64	48	93	4.65	5.0
I-820	670+00	720+00	64	48	93	4.65	5.0
	720+00	762+00	64	48	93	4.65	5.0
	762+00	777+00	62	40	93	3.12	2.8
	777+00	791+00	62	40	93	3.42	3.9
	791+00	814+00	62	40	93	2.09	0.9
	814+00	844+00	61	46	93	4.33	4.1
	844+00	851+00	61	38	93	1.97	0.7
	851+00	920+00	60	35	93	2.92	2.8
	920+00	983+00	62	40	93	3.49	3.7

Table 5.3 FR PVR Design Data and Results (Ahmed & Ranasinghege 2012)

Frontage Roads							
Section	Subsection		LL	PI	% Passing #40	PVR (in)	Treatment Thick. (ft)
	Begin	End					
I-35W	590+00	670+00	60	42	93	3.87	3.0
I-820	670+00	715+00	64	46	93	4.38	3.5
	715+00	755+00	64	42	93	3.87	3.0
	755+00	850+00	61	38	93	3.27	2.2
	850+00	962+00	60	35	93	2.86	1.5

5.3 Pavement Design

The design of the pavement had to consider the AASHTO 1993 methodology, design and build contract requirements, construction phasing, and any environmental concerns. The flexible pavement design was completed assuming a pavement design life of 20 years, absent of any major rehabilitation.

The pavement structure consisted of 4 different layers, stone matrix asphalt (SMA), hot mix asphalt (HMA), flexbase (FB), and lime stabilized soil (LSS). The first three items have standard structural numbers. The LSS structural number was based on the layer's expected stiffness and strength. Table 5.4 summarizes the coefficient values used in the design.

Table 5.4 Layer Structural Coefficients

Layer	Structural Coefficient	Modulus (psi)
SMA	0.38	350,000
HMA	0.42	450,000
FB	0.14	35,000
LSS	0.12	25,000

The structural coefficient value of 0.12 for the LSS subbase was estimated using the AASHTO 1993 structural coefficient equation for a granular subbase material, Equation 5.1 (AASHTO 1993). In order to obtain a structural coefficient value of 0.12, the LSS structural layer would need to have a resilient modulus of at least 16,800 psi. The required resilient modulus for the LSS was 25,000 psi. This target value satisfies the required value needed for a 0.12 structural coefficient; in fact, the 25,000 psi resilient modulus would be more reflective of a 0.16 structural coefficient. The conservative value of 0.12 served as an added safety precaution.

$$a_3 = 0.227 \times \log_{10} E_{SB} - 0.839 \tag{5.1}$$

where

a_3 = granular subbase structural coefficient

E_{SB} = granular subbase resilient modulus (psi)

Other variables required to determine the layer thicknesses included: Reliability, Standard Deviation, PSI initial, PSI Final, Subgrade Resilient Modulus, and Drainage Coefficient. AASHTO 1993 and TxDOT Pavement Design Manual recommendations were used to determine these variables. The final PSI of the frontage roads was designed to be 0.5 less than the managed and general purpose lanes. Table 5.5 summarizes the values used in the structural number and ESAL pavement equation, (.

Table 5.5 AASHTO Pavement Equation Inputs

Parameter	Value
Reliability	95%
Standard Deviation	0.45
PSI Initial	4.5
PSI Final for ML and GPL	3.0
PSI Final for FR	2.5
Subgrade Resilient Modulus (psi)	5000
Drainage Coefficient	1.0

In order to determine the overall required structural number for the pavement system, the number of expected ESALs during the pavement life was required. These ESAL values were provided in the design build contract and were based on traffic data. The expected 20 year ESALs on the GPL, ML and FR were estimated to be approximately 37 million, 13 million, and 8 million, respectively.

Using the overall structural number and the method discussed previously, layer thicknesses were calculated. In order to design a pavement for constructability, some layer thicknesses were increased to create a uniform pavement system. The summary of the general thicknesses can be seen in Table 5.6.

Table 5.6 General Pavement Thicknesses (Ahmed & Ranasinghege 2012)

Roadway	SMA (in)	HMA (in)	FB (in)	LSS (in)	Total (in)
GPL	1.25	8	8	20	37.25
ML	1.25	6.5	8	20	35.75
FR	1.25	5.75	8	20	35.00

5.4 Summary

The design of the LSS contributes to the structural soundness of the pavement and also prevents significant differential pavement movement caused by expanding soils. The LSS layer reduces PVR but also creates a water barrier between the surface and the natural soil under the LSS, which will help prevent the native soil from becoming saturated and expanding; this is just another benefit of the LSS layer.

Figure 5.1 and Figure 5.2 express the different thicknesses of HMA and FB required with and without the use of structural LSS required for different ESAL loads. During the HMA analysis, FB thickness remained constant at 8 inches; HMA thickness remained constant at 8 inches during the FB analysis. Structural LSS thickness remained 20 inches during both analyses. The figures show a reduction of HMA that ranges from 4.5” to 5.75” and a reduction of the FB that ranges from 9.5” to 17”. This reduction in thickness reflects a significant cost savings, which stems from recognizing the beneficial effects LSS has on the pavement structure.

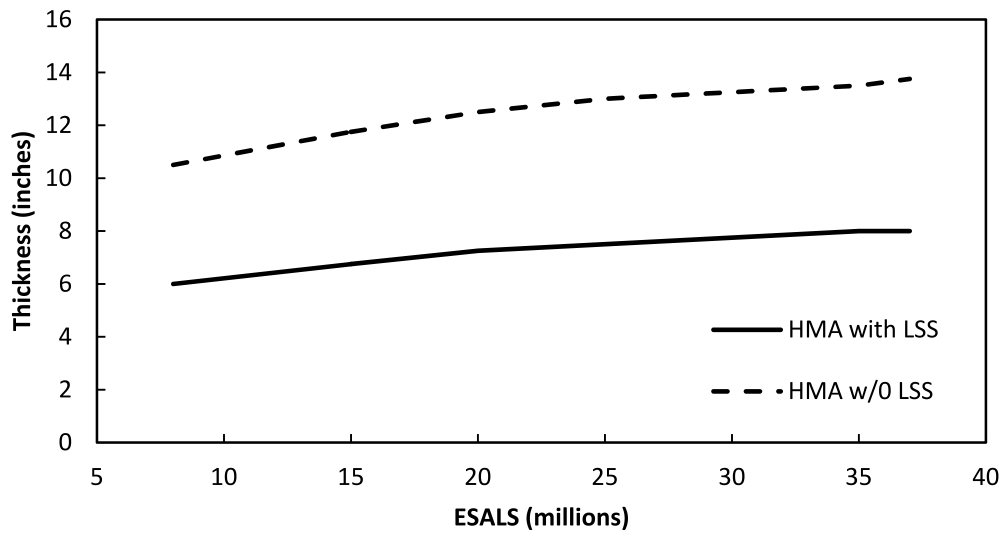


Figure 5.1 Thickness of HMA with and without the Structural LSS

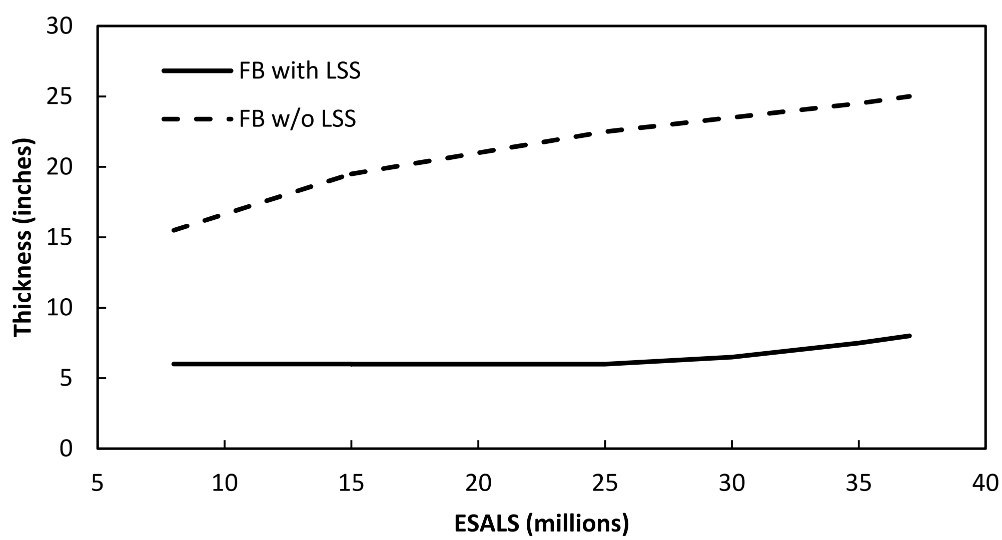


Figure 5.2 Thickness of FB with and without the Structural LSS

The contribution of the LSS to the pavement structure reduces the thicknesses of the more expensive layers, such as the flexbase or the HMA. If LSS was not used and played no part in the pavement design, the total thickness of flexbase could have increased by 200% in some areas. The LSS layer made a valuable contribution to the pavement structure. In order to confirm the engineering properties of the LSS, an extensive QC/QA program was required to guarantee the assumed strength and stiffness characteristics.

Chapter 6

Quality Assurance and Control Program

6.1 Introduction

In order to consider the LSS as a part of the pavement structure, the modification and the pozzolanic chemical reactions between the clay and lime had to be verified. An initial limited verification was conducted during the design process by pH, PI series, UCS and resilient modulus testing. These tests showed lime reacted well with the native clay and that the treated soil meets required strength and stiffness criteria. However, due to variation of soil along the 7 mile alignment and other complications during construction, the in-situ soil and the LSS were tested during and after construction. This verification testing was part of the QA program and included but was not limited to Atterberg limit, sulfate, UCS, and FWD testing.

Testing, however important, is not the only part of the verification process. The type of lime, distribution of lime, preparation of native soil, mixing of the lime with soil, compaction of the treated soil, moisture content of the soil, and curing of the LSS are just a few of the major aspects of lime stabilization that must be supervised during construction. During the QC program, the construction process must be observed and conditions in the field should mimic, as closely as possible, the ideal conditions in the lab; just as conditions in the lab should mimic conditions in the field. If the construction of the LSS layer is not done properly, the LSS will not achieve the physical properties required to validate the design assumptions.

The QC program was based on TxDOT Specifications for Construction and Maintenance of Highways, Streets and Bridges Item 260 Lime Treatment (Road-Mixed). The major aspects of the specification will be discussed in this section. The basic specifications in Item 260 were not altered for this QC program, but additional requirements were added to verify the soil's assumed PI values, which were used to determine the required treatment thickness, and evaluate the strength and stiffness of the LSS.

6.2 LSS Quality Control

The NTE Segment 1 LSS layers were constructed based on a "lot" system. A lot could not consist of a weight greater than 6,000 tons nor could the lime mixing process of a lot be longer than 1 day. Each lot had a specified location and a unique coding system that identified the lot based on its location. Each lot consisted of multiple LSS "lifts", which are layers of LSS of pre-defined thicknesses. LSS layers greater than 12 inches thick were not constructed. Since the minimum thickness of LSS in the project was 20 inches or greater, the total LSS thickness was generally constructed in stages or lifts of 10 inches.

An additional requirement added to the construction process was the validation of the PI in the subgrade material. This step was added to verify the design PI used to estimate the PVR of the active zone in the area, which served as the basis of the treatment thickness. The quality program required 9 native soil samples to be gathered per lot, at least 5 days before any construction was conducted, with 3 sets of 3 samples collected at random locations at specified depths within the proposed lot. At each location, samples were collected at 1, 3 and 5 feet below the bottom of the future LSS layer (1, 3, and 5 feet below the top of the future untreated soil), Figure 6.1. These samples were tested to determine the PI values, which were used to confirm the LSS thickness required to counter the expected PVR. After verifying the LSS thickness, construction of the LSS commenced.

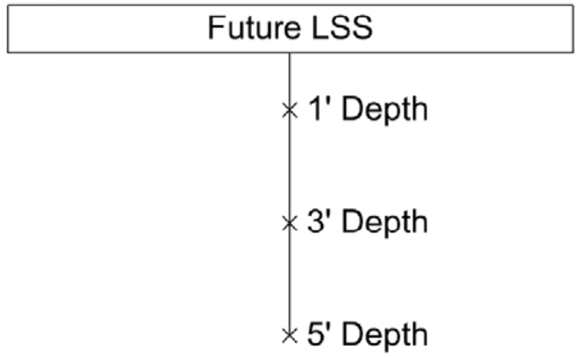


Figure 6.1 Required Sample Depths for PI Validation

Before construction began, the type of lime was chosen for stabilization. Lime could have been transferred to the site as dry hydrated lime, dry quicklime, or slurry lime. Each type of lime has its advantages and disadvantages. However, slurry lime was used for the construction of the LSS in the NTE Segment 1, distributed using slurry tanks equipped with agitation devices.

When beginning the mixing process, the initial step was to make sure the surface of the native soil was scarified and partially pulverized. The soil was not to contain foreign materials, such as roots, stones, or debris, larger than 3 inches and 100% of the soil was to pass through the 2.5 inch sieve. Pulverization increases the quantity of surface area; the greater the exposed soil surfaces area, the better the lime will react. Scarification helps reduce the amount of runoff and pooling, which keeps the lime evenly distributed.

After the soil was properly scarified and pulverized, lime was added to the system. A qualified supervisor was to be on site to determine that the lime was being spread properly. If the lime is not distributed evenly, some soil may receive less than the design minimum lime percentage of 6% and the soil-lime mixture may not have the proper reaction. Lime was not to be applied unless the ambient temperature was at least

35°F and rising, or greater than 40°F. Figure 6.2 shows the placement of lime slurry for the first lift of the I-35 northbound frontage road, Lot 13.

Figure 6.2 Lot 13 Lime Slurry Placement

Following the addition of lime, the soil and lime was to be thoroughly mixed within 6 hours of lime application. Rotary mixers were used to insure the entire area was properly mixed with lime. When mixing was finished, the soil was shaped, leveled, and lightly compacted with a roller. Figure 6.3 shows the lime being mixed into the soil at Lot 13.

Figure 6.3 Lot 13 Mixing with Rotary Mixer

After light compaction, the lime soil mixture was allowed to mellow. Mellowing allows time for cation exchange and chemical reactions. It also serves as a way to mitigate the deleterious effects of any organics and sulfates that may be in the system; the amount of these elements present may affect the duration of the mellowing period. Mellowing typically can range from 1 to 7 days or it may not even be required, if the soil has low plasticity or when drying or modification is the goal. Soils in the NTE Segment 1 did not have a high enough sulfate content to require additional mellowing past 24 hours, and any organic material encountered was to be removed before mixing. Construction recommendations did not allow for mellowing longer than 9 days. If mellowing exceeded 9 days, the lot would be reworked with an additional 2% of lime.

Immediately following the mellowing period, remixing of the soil began. The soil was remixed and pulverized until 100% of non-stone material passed the 1 inch sieve and at least 60% of non-stone material passed the number 4 sieve. If required, water was to be added to insure that final compaction met specifications. Figure 6.4 shows the remixing of a Lot along I-820 westbound frontage road.

Figure 6.4 Lot during Remixing

Directly after the soil was remixed, the mixture was to be compacted using vibratory or pneumatic rollers, a combination of the sheepsfoot and light pneumatic vibratory padfoot rollers, or tamping foot rollers. The area was to be compacted to the pre-approved specified density which was typically 95% of the maximum density.

After final compaction, the lot began the curing process, during which the pozzolanic reactions occur and the soil becomes more cementatious. The soil gains significant strength in this stage. Steps were taken to prevent external water from penetrating the system during the curing process and any water that gathered around the system was to be removed. Drainage paths were dug around the system in order to avoid pooling water. Figure 6.5 shows a finished lot after compaction along the 35W northbound frontage road.

Figure 6.5 Finished Lot Compacted and Curing

Certain issues arose during construction which had to be addressed by the quality team, such as rainfall, delays in mixing, improper lime percentage, traffic issues, etc. Occasionally, the engineer or quality team requested depth checks of the LSS layers, which guaranteed that the lime thicknesses were correct. The check was done with Phenolphthalein, which reacts and turns purple when in contact with lime treated material.

The construction of the LSS was done according to the approved protocol to guarantee the required reactions would take place. The process was supervised from beginning to end. In this process if shortcuts or corners are cut, the entire process could have been compromised, which would have resulted in the replacement of the LSS. During and after construction, samples were taken and tested to observe the reactions between the lime and clay.

6.3 Quality Assurance

In addition to standard practice outlined in Item 260, continual testing requirements were incorporated during lime placement. Since the LSS would be considered part of the structural makeup of the pavement, stricter requirements for the soil mixing process were defined. The additional tests, standards, and the required frequency can be seen in Table 6.1.

Table 6.1 Continual LSS and Native Soil Testing

Test Purpose	Test Standard	Frequency
Identify soluble sulfates in native soil	Tex-145-E	1 every 500 feet in cuts. At the engineer's discretion in fills
Determine PI for native soil before treatment and LSS after initial mixing	Tex-104-E Tex-106-E	1 per lot
Determine percent passing number 200 sieve	Tex-111-E	1 per lot
Test the quality of the Lime	Item 260	Chemical analysis for each source
Test the quality of the water	Item 260	Approved source
Verify depth of lime treatment	Tex-140-E	Engineer's discretion
Determine moisture content	Tex-103-E	1 per lot
Determine density	Tex-115-E	3 per lot
Check gradation of LSS after mellowing	Item 260	1 per lot
Determine grading of LSS layer	Item 260	1 check every 50 feet

An extensive UCS testing program was implemented to estimate the strength and stiffness of the LSS and monitor the lime/clay reaction. Each lot was divided into a number of sublots based on the total tonnage of material; lots of 4000 to 6000 tons of material were divided into 3 sublots, lots of 2000 to 4000 tons of material were divided into 2 sublots, and lots with less than 2000 tons of material were not divided. Samples were collected from each sublot per lift for UCS testing after the lime was added and final mixing was completed following the mellowing period.

Enough soil was collected in each sublot to run 1 UCS test after 4 days, 5 UCS tests after 8 days, and 5 additional UCS tests at 16 days, if required. 16-day tests were only conducted if the 8-day test results were lower than expected. The samples were collected randomly throughout the sublot. These samples were compacted using the modified proctor standard, cured at 104°F for n-1 days, subjected to capillary soaking for 1 day, and then tested on the n^{th} day.

The sublot was considered passing if the average UCS values were equal to or above 125 psi and no more than 2 of the 5 test values were less than 100 psi. If the sublot was deemed unsuitable after initial testing, the additional samples that were subjected to 15 days of curing and 1 day of soaking were tested. If the average of these values was greater than 125 with no more than two values less than 100 psi, the sublot was considered passing.

In circumstances where a lot was deemed unsuitable after sublot analysis, additional testing took place. This testing included Atterberg, sulfate content, and pH. Furthermore, it was recommended that FWD testing should be conducted on the LSS after 30 days of field curing. If the lot was still considered unsuitable after the analysis of the additional tests and the lift was considered part of the pavement structure, the lift would be reworked with 2% additional lime until the LSS passed the predefined quality check. If the LSS did not serve a structural purpose but only served in reducing PVR, other steps were to be considered.

6.4 QA UCS Testing Results

6.4.1 Introduction

During the time of this study, Segment 1 of the NTE was still under construction. The testing summarized in this section was collected between the start of construction to October 23, 2013. Over 5000 UCS tests were conducted during the QA program in the NTE Segment 1 over a period of approximately 2.5 years. These tests were filed according to specific codes that served as identification. The alignment was split into 10 subsegments, with the UCS test results for 8 of the 10 subsegments along the alignment reported in this section. Two of the subsegments did not have enough testing completed to be considered in this study.

The location and UCS results of each sample in each sublot was recorded and summarized with data from the same subsegment. The results have been summarized graphically and in tabular form. These UCS values were correlated using the Thompson (1966) correlation to estimate the resilient modulus. The summarized UCS testing corresponds to tests run at 3, 7, and 15 days oven curing and 1 day soaking.

In addition to UCS results, resilient modulus correlated values are summarized graphically for each subsegment. These results are representative of UCS tests conducted at 8 and 16 days. The 8-day test results represent field curing at 28 days and moisture conditions. 16-day testing was only conducted when 8-day testing produced questionable results. For this reason, it is difficult to find a UCS to time relationship for the data according to subsegments. An overall project relationship can be defined by discarding data that does not have UCS results for 4, 8, and 16-day curing. Due to the elimination of the well performing 8 day testing data, the UCS verses time relationship is on the conservative side.

6.4.2 Subsegment A

This subsegment was located along the I-35 alignment between stations 576+05 to 671+97. The subsegment is located in the Fort Worth and Duck Creek (undivided) and the Denton, Weno, Pawpaw (undivided) formations. According to the geologic formation and soil classification data, the soil in this area should be predominantly clay with medium to high PI values. After the geotechnical investigation this assumption was confirmed. The average PI in the subsegment was 33, the average UCS compression strength of the natural soil was 42 psi, and the average percent passing the No. 200 sieve was 81%.

After 3 days of accelerated curing and 1 day of soaking, the average compression strength of the lime stabilization soil was 340% greater than that of the natural soil. Following 7 days of curing and 1 day of soaking, the LSS displayed average compression strength 590% greater than the average natural soil compression strength. This data suggests lime stabilization was effective in the area. Table 6.2 summarizes the data gathered from Subsegment A.

Table 6.2 Summary of Test Data from Subsegment A

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	98	19	18.5%	185
8-Day	490	16	3.2%	291
16-Day	445	4	0.8%	387

Using the Thompson correlation, Equation 2.3, only 4 of the 445 samples tested at 16 days had a resilient modulus less than the ideal 25,000 psi value. When looking at each sublot UCS results, only 1 sublot of 99 failed to meet the QA requirement at 8 days and no sublot failed at 16 days. Based on the QA criteria, 100% of lots passed in this subsegment. Figure 6.6 present a graphical review of the UCS testing along the alignment. Figure 6.7 and Figure 6.8 express a graphical summary of the expected resilient modulus values based on the Thompson correlation and UCS results.

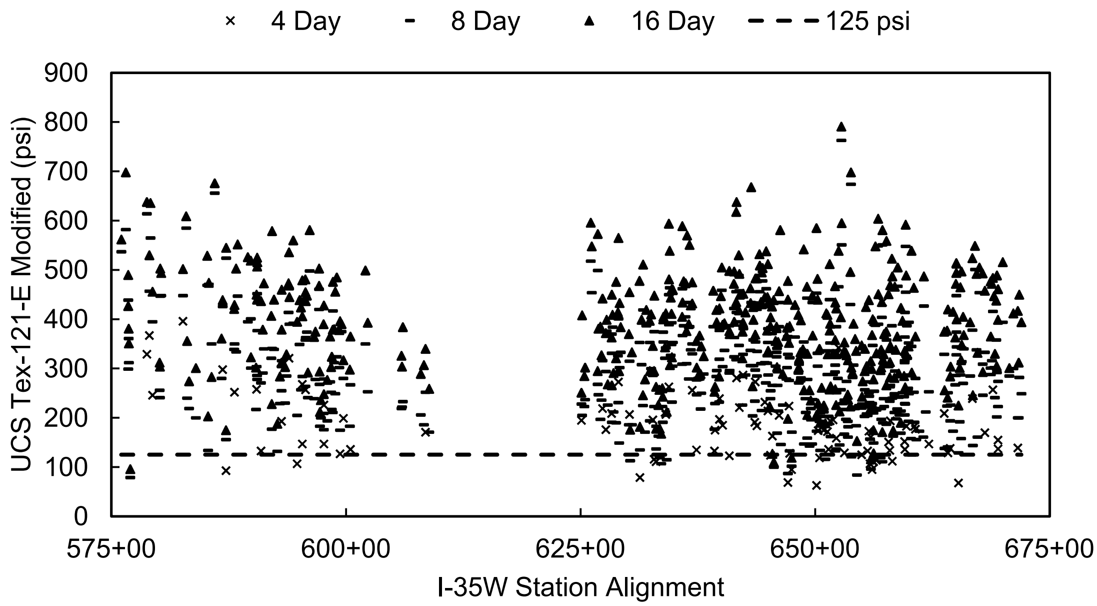


Figure 6.6 UCS Results for Subsegment A

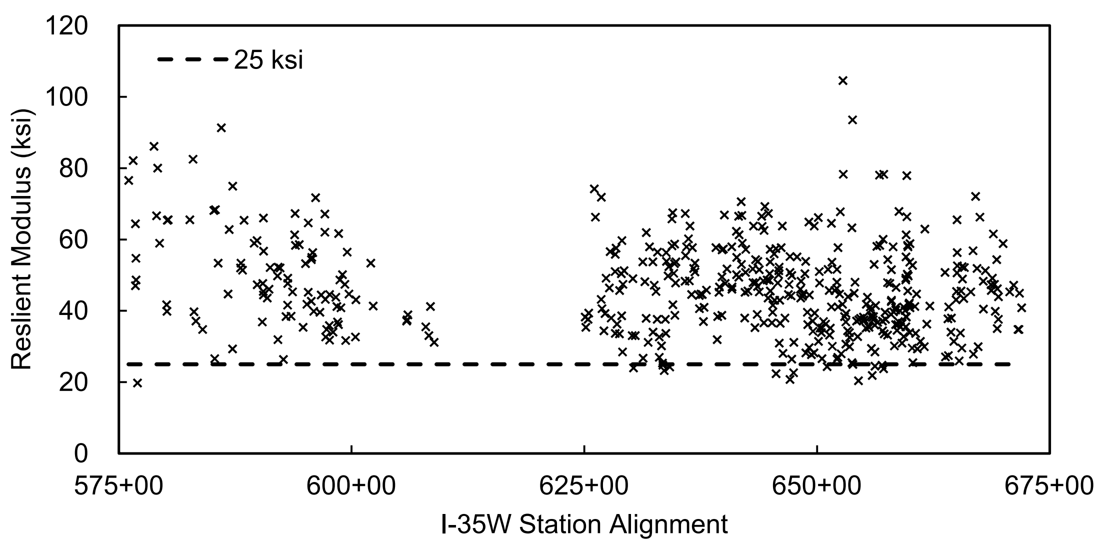


Figure 6.7 Subsegment A Resilient Modulus Correlation from UCS at 8 Days

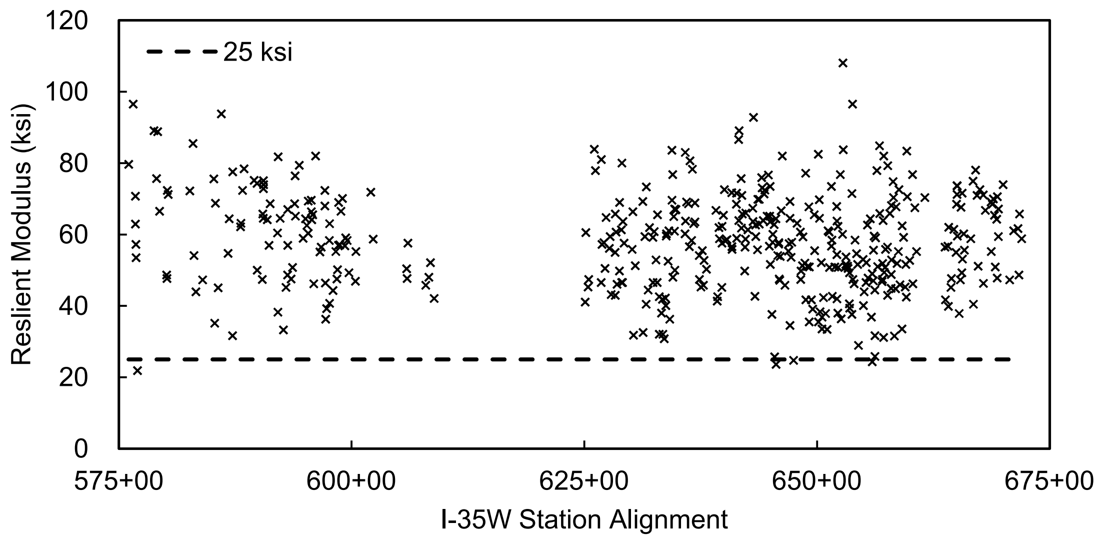


Figure 6.8 Subsegment A Resilient Modulus Correlation from UCS at 16 Days

6.4.3 Subsegment B

Subsegment B was located along the I-820 alignment between stations 674+00 to 731+00. This subsegment was the farthest east along I-820 and fell completely into the Denton, Weno, Pawpaw (undivided) formation. The geotechnical investigation showed an average PI of 37, UCS of 38 psi, and percent passing No. 200 of 79%. 1,358 UCS tests were completed in this subsegment. Only 3 tests failed to reach the 25,000 psi resilient modulus after 15 days curing and 1 day of soaking. After the 7 day curing period, the average UCS result was 520% greater than the untreated clay average UCS. Table 6.3 summarizes the UCS data in Subsegment B.

Table 6.3 Summary of Test Data from Subsegment B

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	178	71	39.5%	151
8-Day	890	31	3.4%	237
16-Day	290	3	1.0%	320

When evaluating UCS results with the QA criteria, 4 out of 178 8-day sublots failed and no sublots failed at 16-days. There were no lots that failed in Subsegment B. Figure 6.9 summarizes the UCS results, and Figure 6.11 and Figure 6.10 summarize the expected resilient modulus.

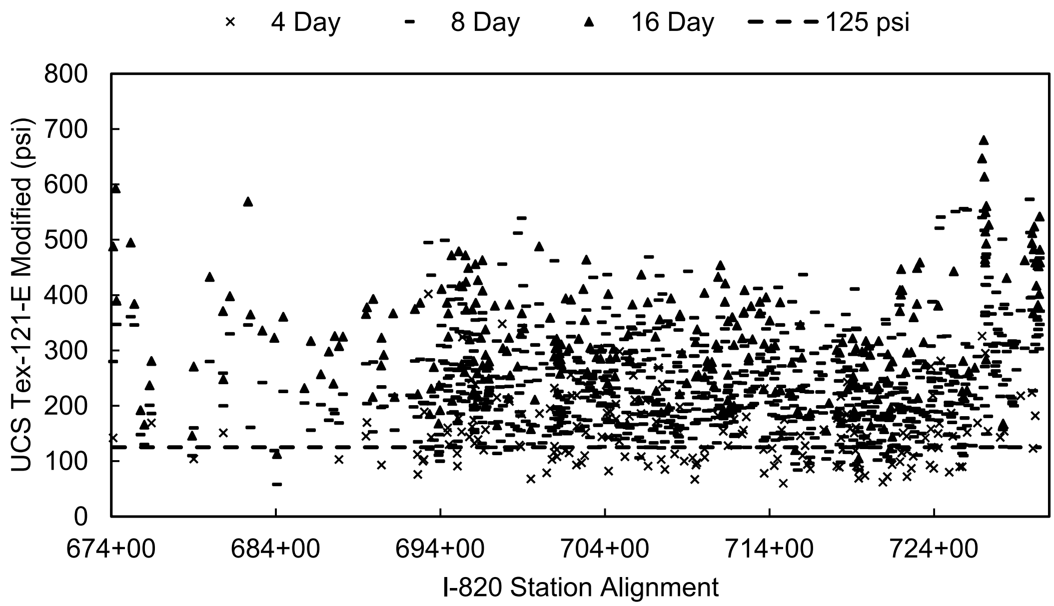


Figure 6.9 UCS Results for Subsegment B

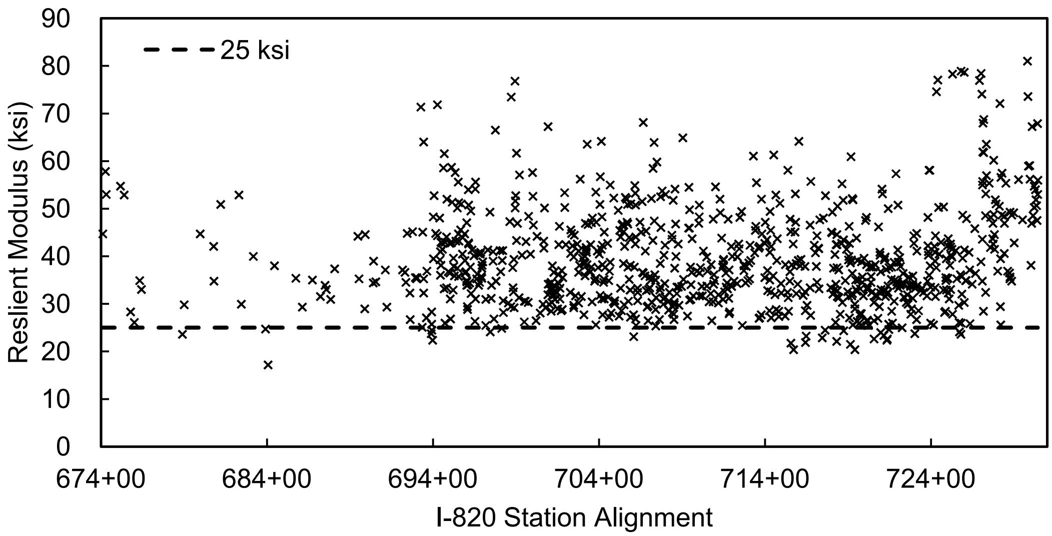


Figure 6.10 Subsegment B Resilient Modulus Correlation from UCS at 8 Days

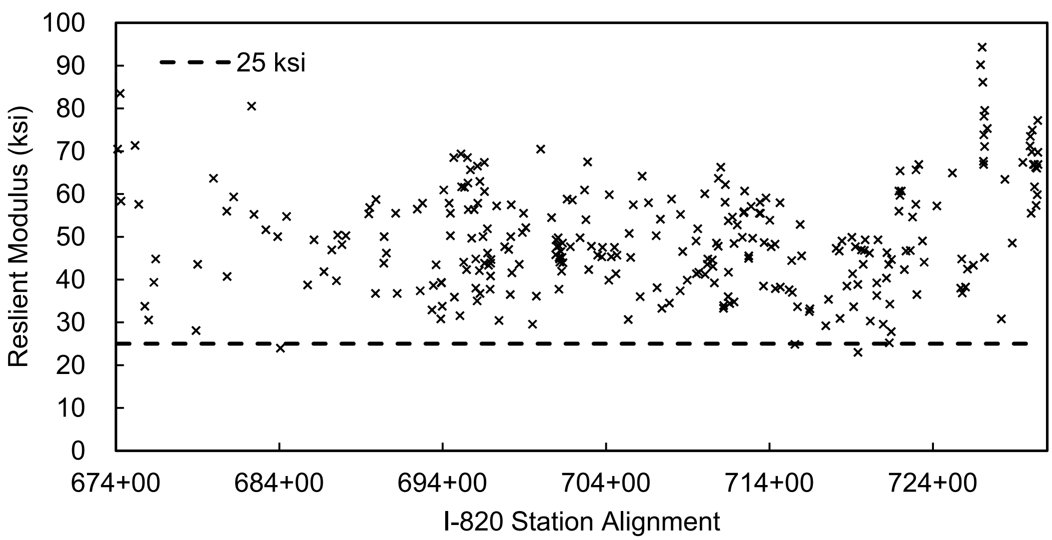


Figure 6.11 Subsegment B Resilient Modulus Correlation from UCS at 16 Days

6.4.4 Subsegment C

This subsegment was located between stations 731+00 to 787+00 and overlapped with Subsegment D. This subsegment is located completely in the Denton, Weno, Pawpaw (undivided) formation. The geotechnical investigation revealed a natural soil with an average PI of 35, UCS of 43, and percent passing the No. 200 sieve of 79%. 1294 UCS tests were conducted in this subsegment. Table 6.4 summarizes the UCS data collected on the LSS in the subsegment. The average compression strength of the LSS at 7 day curing and 1 day soaking was 425% greater than the average untreated UCS.

Table 6.4 Summary of Test Data from Subsegment C

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	165	61	36.5%	155
8-Day	825	46	5.5%	226
16-Day	304	1	0.3%	284

When considering the QA criteria, 5 sublots of 165 failed at 8-day testing and no sublots failed at 16-day testing. According to the QA recommendations, no lots failed in Subsegment C. Figure 6.12 summarizes the UCS results; Figure 6.13, and Figure 6.14 summarize the expected resilient modulus using the Thompson correlation.

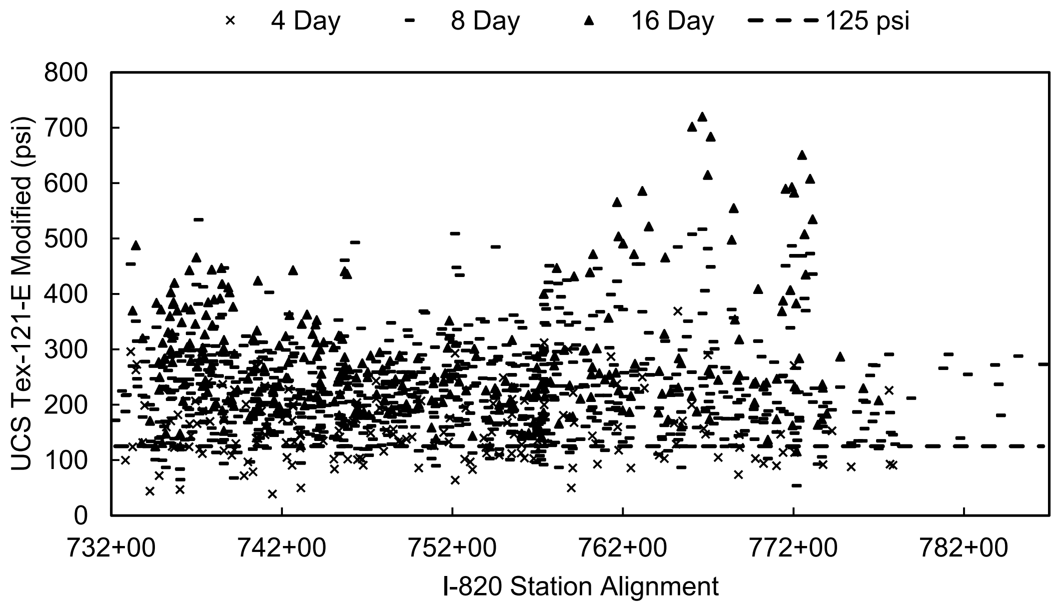


Figure 6.12 UCS Results for Subsegment C

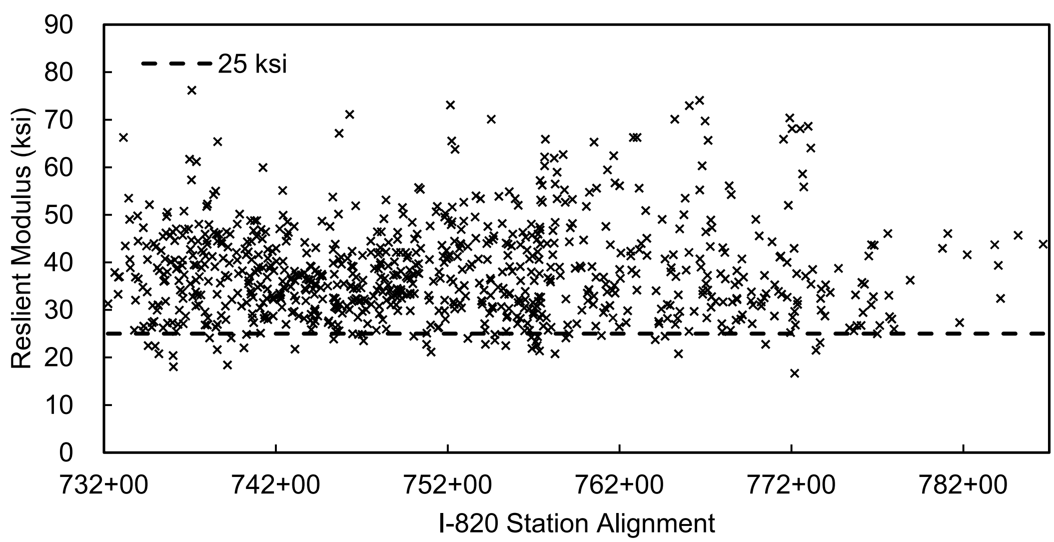


Figure 6.13 Subsegment C Resilient Modulus Correlation from UCS at 8 Days

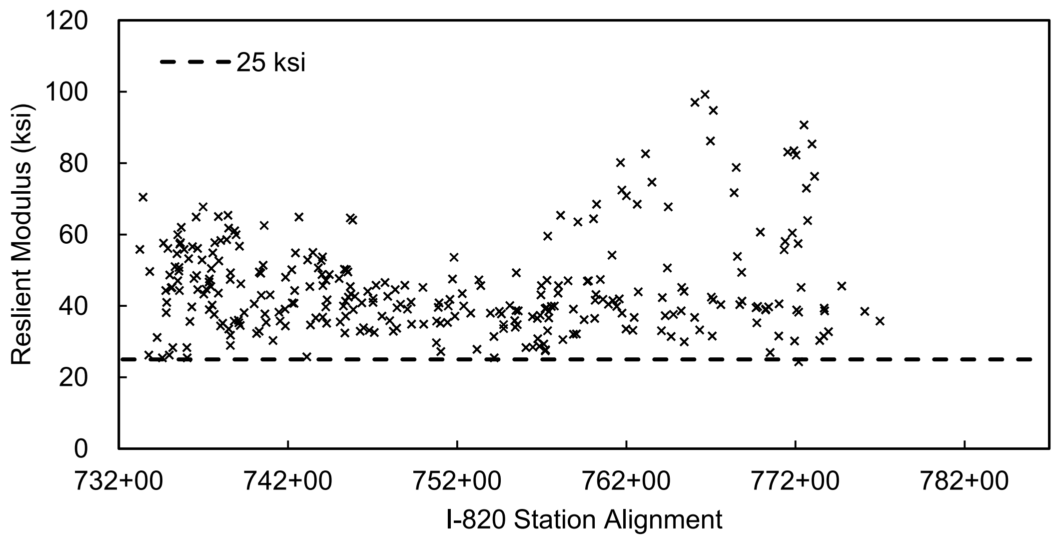


Figure 6.14 Subsegment C Resilient Modulus Correlation from UCS at 16 Days

6.4.5 Subsegment D

This subsegment began at station 773+00 and ended at 812+00 along I-820. The beginning of this subsegment overlapped with Subsegment C. The subsegment is located in the Denton, Weno, Pawpaw (undivided) formation. The natural soil had an average PI of 32 and UCS of 56 psi. Not enough data on the percent passing 200% No. sieve was collected to provide any intelligible data on this subsegment. Table 6.5 summarizes the UCS data for the LSS in the subsegment. The average 8 day UCS value for the LSS was 300% greater than the average UCS of the untreated soil.

Table 6.5 Summary of Test Data from Subsegment D

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	65	24	36.7%	147
8-Day	325	39	12.0%	229
16-Day	165	10	6.0%	266

According to the UCS testing, a problem area developed during the construction of the LSS, around station 793+00 in the LSS layer designated for the general purpose lane and just west of the Union Pacific Railroad. Several of the UCS tests in this area failed the 8-day testing; unusually low UCS values occurred at the 16-day testing. This area can be seen in Figure 6.15.

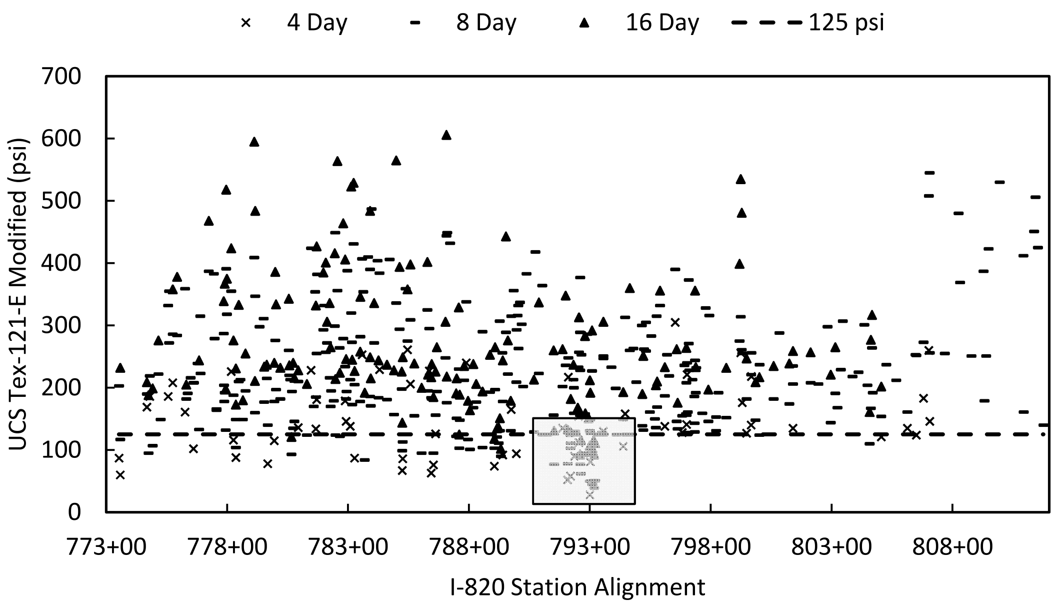


Figure 6.15 UCS Results for Subsegment D

When the 8-day and 16-day values were used to estimate the resilient modulus, the number of failing values significantly decreased. 10 of the 165 test ran at 16-days fail to meet the 25,000 psi required for resilient modulus; only one sublot failed at 16-days, when compared to the QA criteria. Based on additional testing conducting in this area for PI, pH, and sulfates, the obvious increase in strength with time, and the overall evaluation of the lot, the Lot was determined to be acceptable by the engineer in charge. All lots in Subsegment D were deemed passing based on QA criteria. Figure 6.16 and Figure 6.17 summarizes the resilient modulus values expected from the UCS test results.

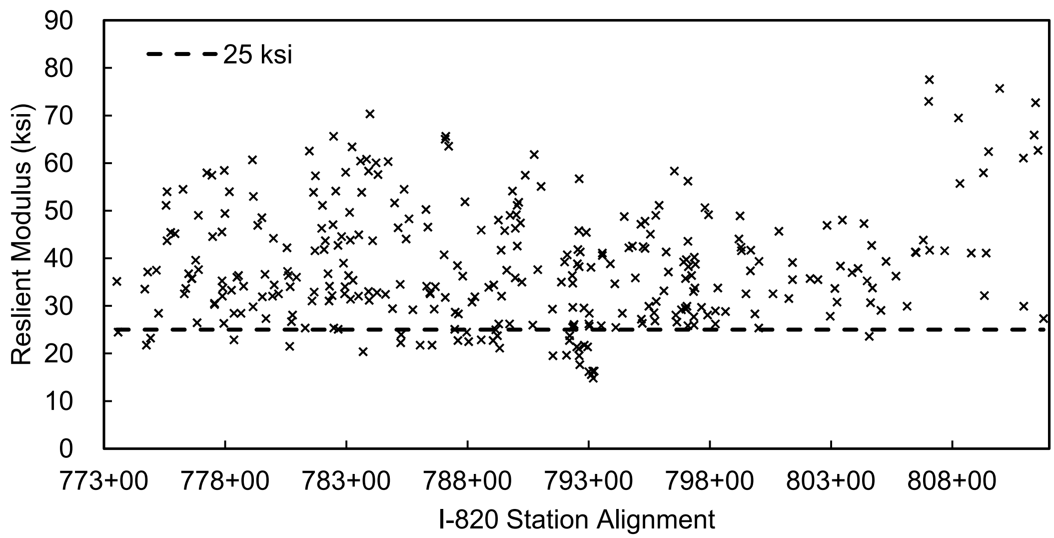


Figure 6.16 Subsegment D Resilient Modulus Correlation from UCS at 8 Days

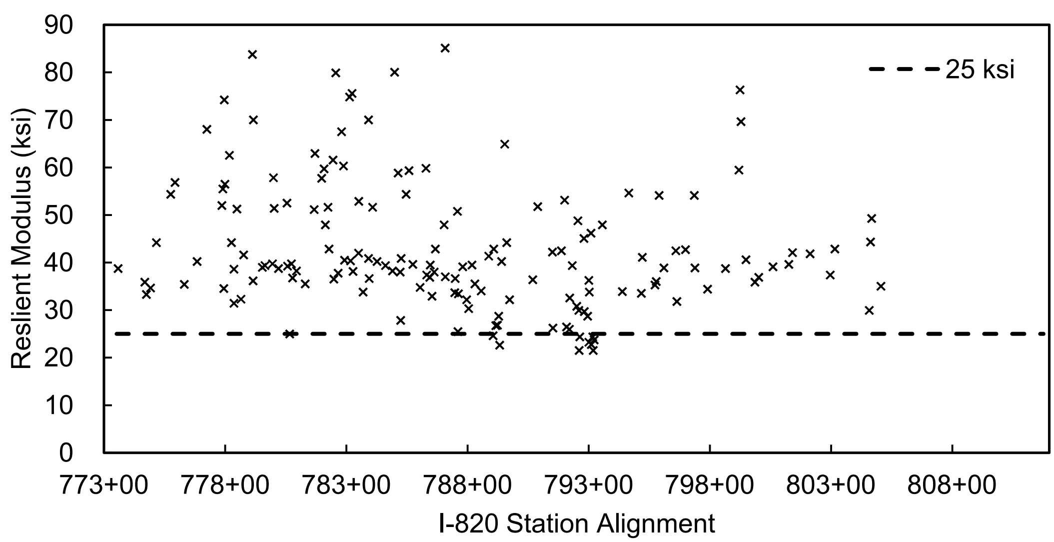


Figure 6.17 Subsegment D Resilient Modulus Correlation from UCS at 16 Days

The low UCS results in this area were an abnormality for the NTE Segment 1. An analysis in the geological formation and borehole information would suggest the soil in the area was a prime candidate for lime stabilization. The only other possible explanation for the low UCS results was poor field construction. Historical documents state on the day of mixing there was a small amount of rainfall in the area. This rainfall could have increased the moisture content of the soil well above optimum, which would have had a deleterious effect on the mixing and reaction process. During the construction process, weather forecasts must be constantly reviewed and no lime should be placed on a day with heavy rainfall.

6.4.6 Subsegment E

This subsegment, located between stations 814+00 and 847+00 along I-820, crossed through three different geologic formations: Denton, Weno, Pawpaw; the Quaternary Alluvium; and the Quaternary Terrace. The subsegment also passed through two small water conduits: Big Fossil Creek and Singing Hills Creek. The presence of the Quaternary Terrace suggests the Trinity River once passed through this location. Extra care should be taken in locations such as this, as water tends to carry foreign soil into an area, which may not be conducive with lime stabilization.

The geotechnical investigation in the location of the two Quaternary formations showed no major deviation from the typical soil values found in the NTE Segment 1 location. The averages for PI, UCS, and percent passing the No. 200 sieve in the total subsegment were 30, 48 psi, and 78%, respectively.

Table 6.6 summarizes the UCS data collected in Subsegment E. The average UCS at 8 days for the LSS layer was 440% greater than the average UCS of the untreated soil in the subsegment. No UCS value tested at 16 days failed to achieve the 125 psi threshold.

Table 6.6 Summary of Test Data from Subsegment E

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	48	12	24.1%	160
8-Day	240	5	2.0%	261
16-Day	15	0	0.0%	227

No resilient modulus value failed to reach the 25,000 psi threshold at 16 days. Furthermore, only one sublot failed at 7-day testing, based on QA criteria with an average of 124 psi test result; this sublot passed when re-evaluated at 16 days. All lots in this subsegment were considered passing based on QA criteria. Figure 6.18 summarizes the data gathered from UCS testing. Figure 6.19 and Figure 6.20 summarize the resilient modulus correlation in Subsegment E.

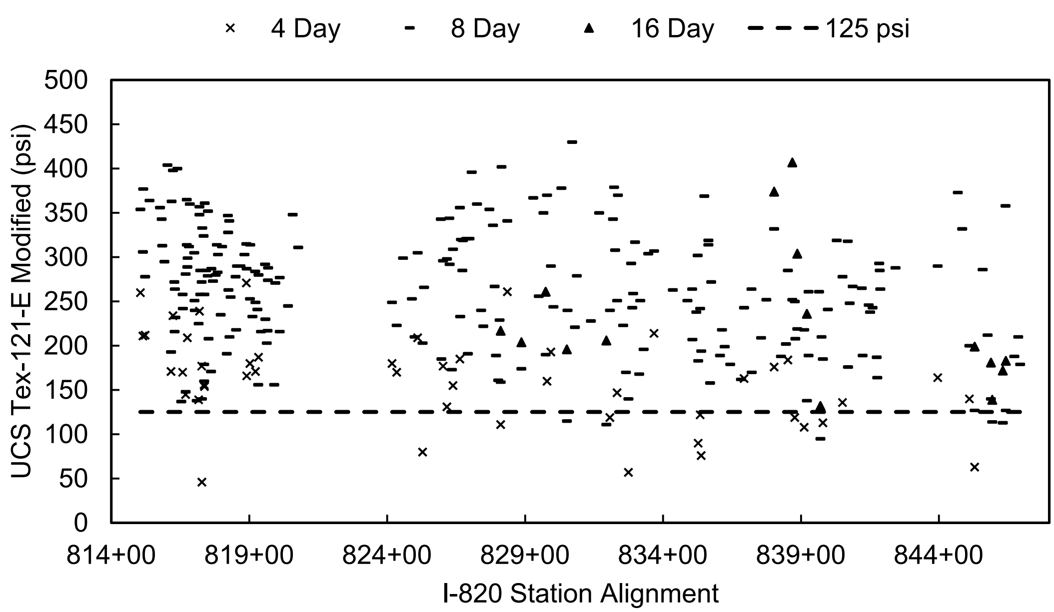


Figure 6.18 UCS Results for Subsegment E

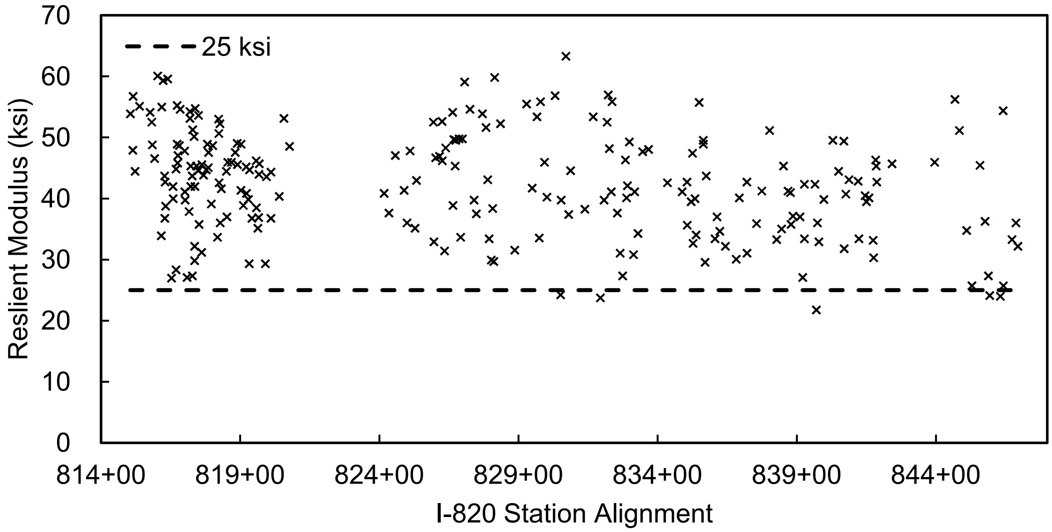


Figure 6.19 Subsegment E Resilient Modulus Correlation from UCS at 8 Days

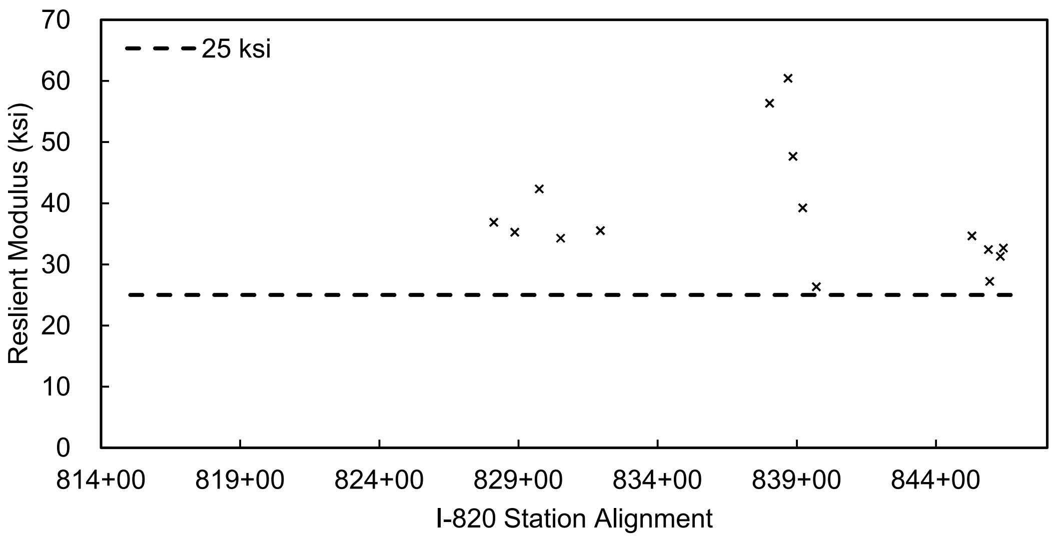


Figure 6.20 Subsegment E Resilient Modulus Correlation from UCS at 16 Days

6.4.7 Subsegment F

This subsegment was located between stations 845+00 to 889+00 along I-820, partly in the Denton, Weno, Pawpaw (undivided) formation and in the Mainstreet and Grayson (undivided) formation. The average PI, UCS, and percent passing No. 200

sieve in the subsegment were 32, 46 psi, and 74%, respectively. As in the majority of subsegments, no problem areas in this subsegment were observed. Over 580 UCS tests were conducted at 4-day, 8-day, and 16-day. Table 6.7 summarizes the UCS data collected in the subsegment. The average UCS value at 8 days for the LSS layer was 330% greater than the average compression strength for the untreated soil in the area.

Table 6.7 Summary of Test Data from Subsegment F

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	70	40	56.8%	133
8-Day	350	33	9.3%	200
16-Day	160	5	2.7%	275

Considering 40 out of 70 4-day tests failed, which is over half of the tests, the results expressed a considerable increase in strength over time. As before, no sublots failed the QA criteria at 16-day testing and only 4 tests failed to meet 25,000 psi resilient modulus at 16 day. All lots passed the QA criteria in Subsegment F. Figure 6.21 summarizes the UCS testing, and Figure 6.22 and Figure 6.23 summarize and resilient modulus correlations for Subsegment F.

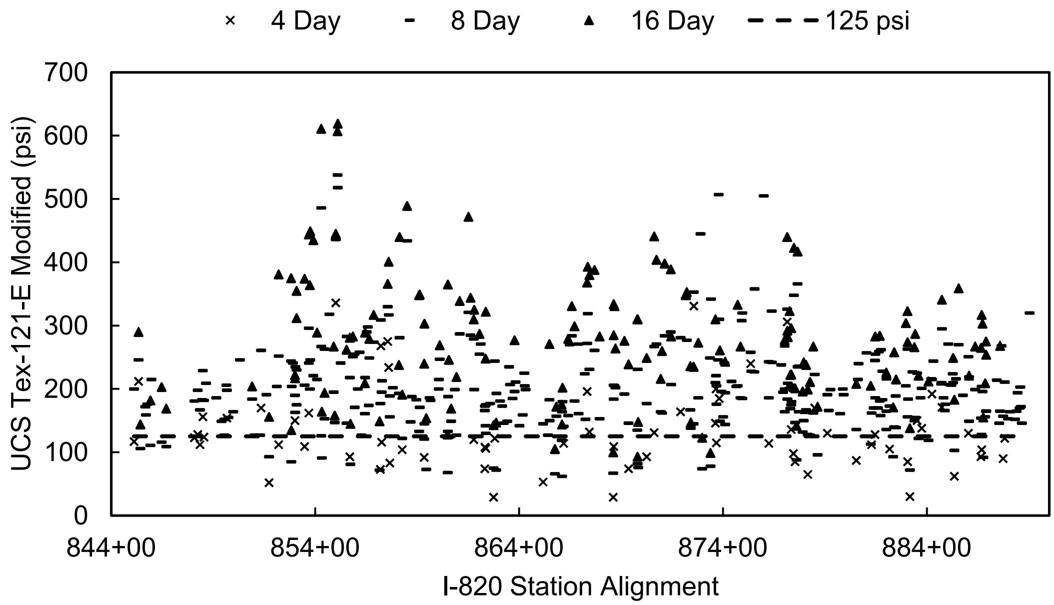


Figure 6.21 UCS Results for Subsegment F

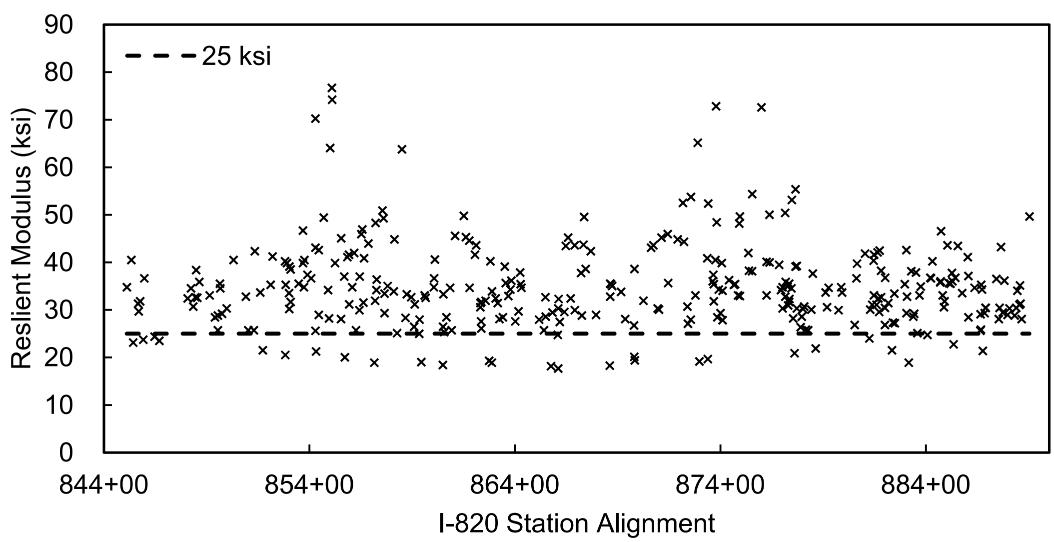


Figure 6.22 Subsegment F Resilient Modulus Correlation from UCS at 8 Days

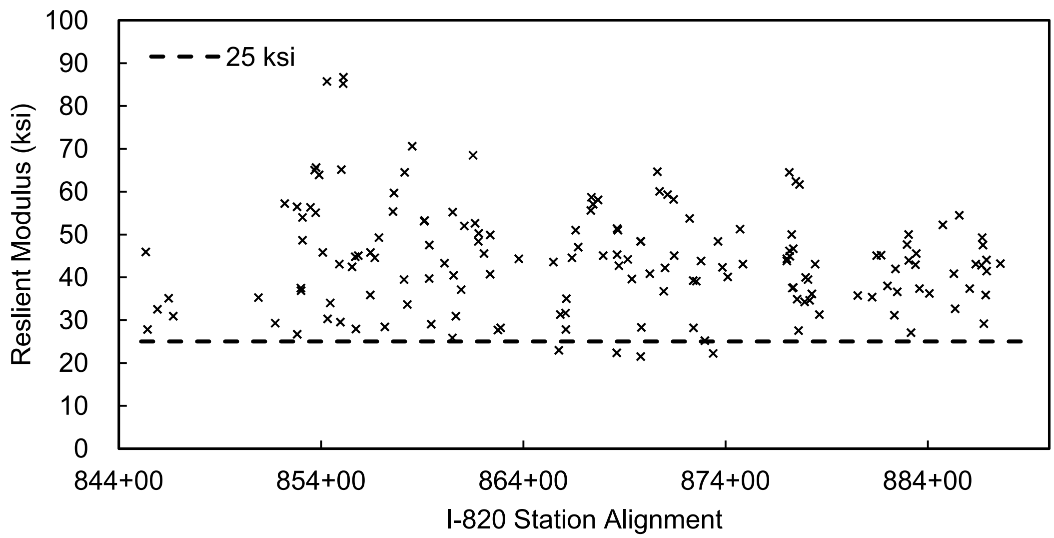


Figure 6.23 Subsegment F Resilient Modulus Correlation from UCS at 16 Days

6.4.8 Subsegment G

Subsegment G was located between stations 889+00 and 924+00. The subsegment was located in two different geologic formations: the Mainstreet and Grayson (undivided) and the Quaternary Terrace. The geotechnical investigation in this are revealed soil with average PI, UCS, and percent passing No. 200 sieve of 29, 39 psi, and 60%, respectively.

The UCS test results in this subsegment demonstrated a high amount of strength gain quickly. Results from the UCS testing can be seen in Table 6.8. At the time of this study, 511 UCS tests have been conducted in this area. Of the 355 UCS tests at 8-day, 4 tests failed to achieve the 125 psi threshold. At the 16-day testing, no UCS test result failed to achieve the 125 psi strength and no result failed to achieve the resilient modulus value of 25,000 psi when correlated. The average UCS for the LSS at 8 days was 670% greater than the UCS for the untreated soil in the area.

Table 6.8 Summary of Test Data from Subsegment G

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	71	14	18.5%	186
8-Day	355	4	1.0%	302
16-Day	85	0	0.0%	475

The rapid development in strength may be contributed to a change in soil of geological formations. Subsegment G and H are found in different geological formations than the rest of the NTE Segment 1. The mineralogy in these formations may explain the significant strength increase with time.

All sublots passed the QA criteria after the 8-day testing. The LSS reactions in this and the subsequent subsegment were the strongest and developed the fastest in the project. All lots passed the QA criteria in this subsegment. Figure 6.24, Figure 6.25, and Figure 6.26 graphically summarize the data gathered for this subsegment.

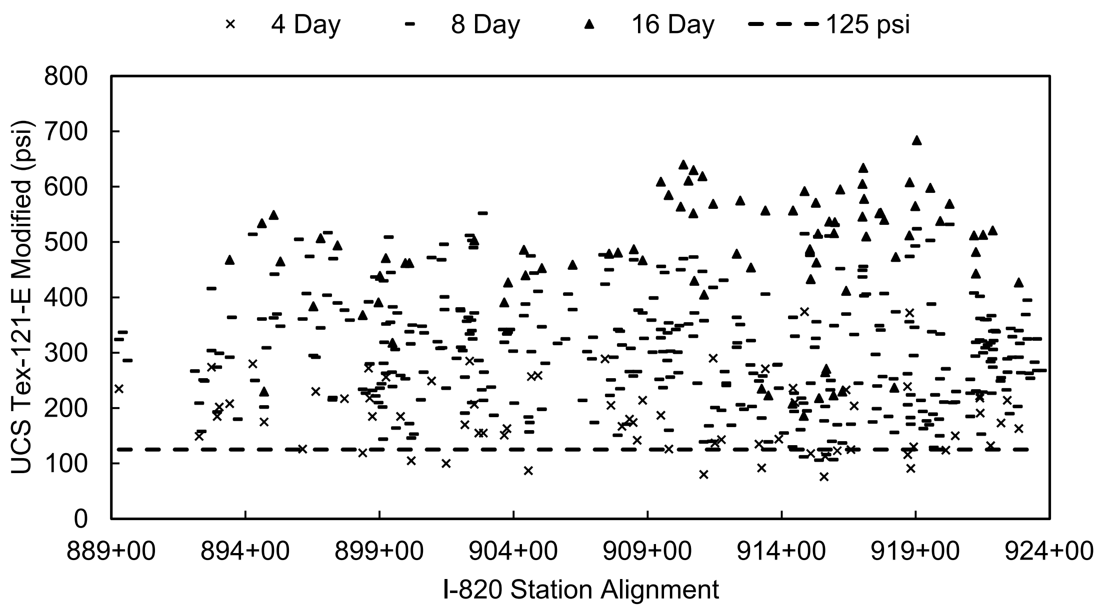


Figure 6.24 UCS Results for Subsegment G

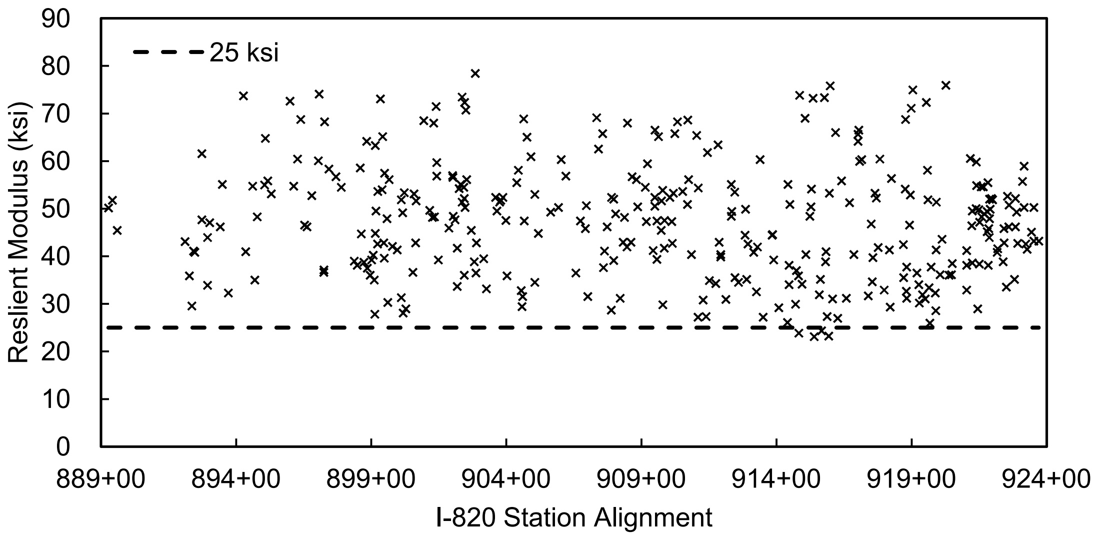


Figure 6.25 Subsegment G Resilient Modulus Correlation from UCS at 8 Days

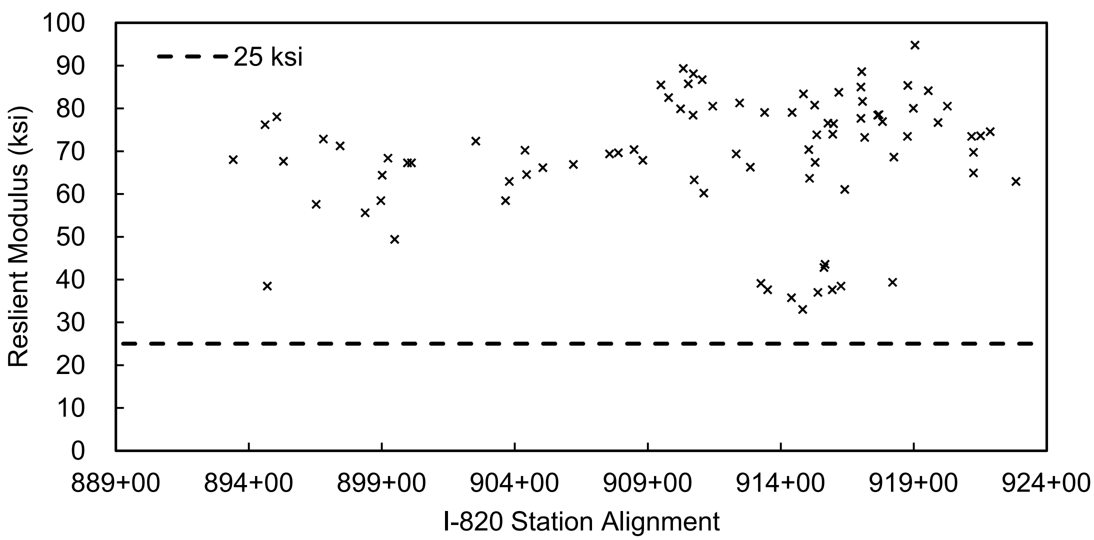


Figure 6.26 Subsegment G Resilient Modulus Correlation from UCS at 16 Days

6.4.9 Subsegment H

This subsegment was the final area of analysis in this study. The subsegment was located at the far east end of Segment 1 between stations 924+00 and 961+00. The majority of this subsegment fell in the Quaternary Terrace formation with a small amount located in the Mainstreet and Grayson (undivided) formation. The soil data gathered from the geotechnical investigation showed soil with slightly different characteristics than found in the rest of the project area. The soil had a lower average PI and percent passing No. 200 sieve, 26 and 54%, respectively, than the other subsegments. However, the values were still within the acceptable limits for lime stabilization. The average UCS value in the subsegment was found to be 42 psi.

The summary of UCS data on the LSS can be seen in Table 6.9. This subsegment along with previous Subsegment H showed the strongest lime reactions in the NTE Segment 1. The average 8-day UCS value was 650% greater than the average untreated UCS value. Of the 470 8-day UCS tests, 1 test failed to reach the 125 psi threshold. All values reached the target resilient modulus of 25,000 psi by the 16-day UCS testing.

Table 6.9 Summary of Test Data from Subsegment H

Curing	No. of Tests	No. < 125 psi	Percentile < 125 psi	Average (psi)
4-Day	94	12	11.8%	203
8-Day	470	1	0.2%	315
16-Day	26	0	0.0%	461

No sublots failed the 8-day testing and all lots were considered passing in this area. Figure 6.27, Figure 6.28, and Figure 6.29 summarizes the UCS test and resilient modulus correlated data for the subsegment.

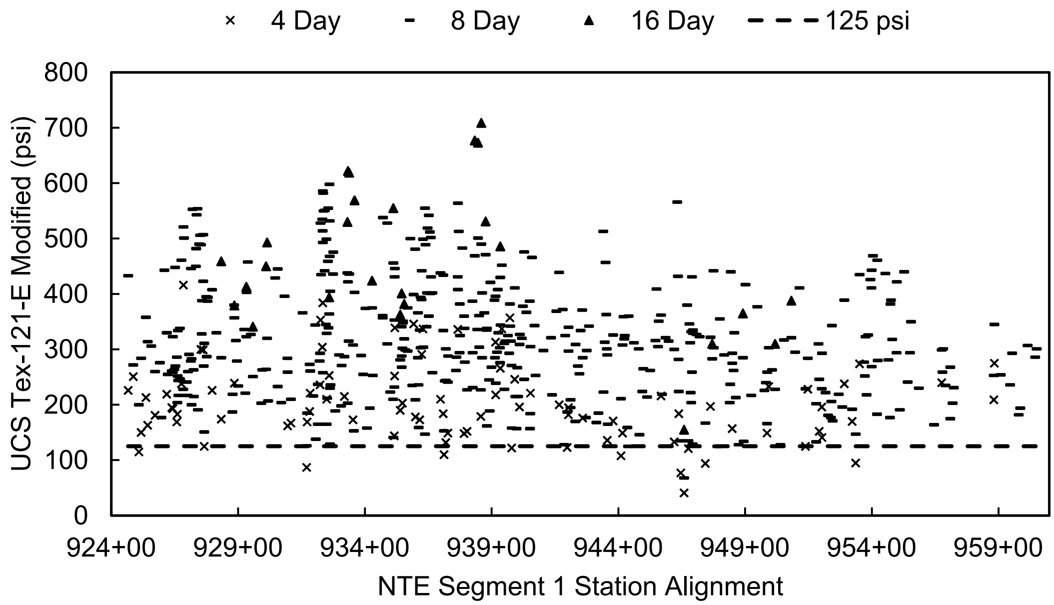


Figure 6.27 UCS Results for Subsegment H

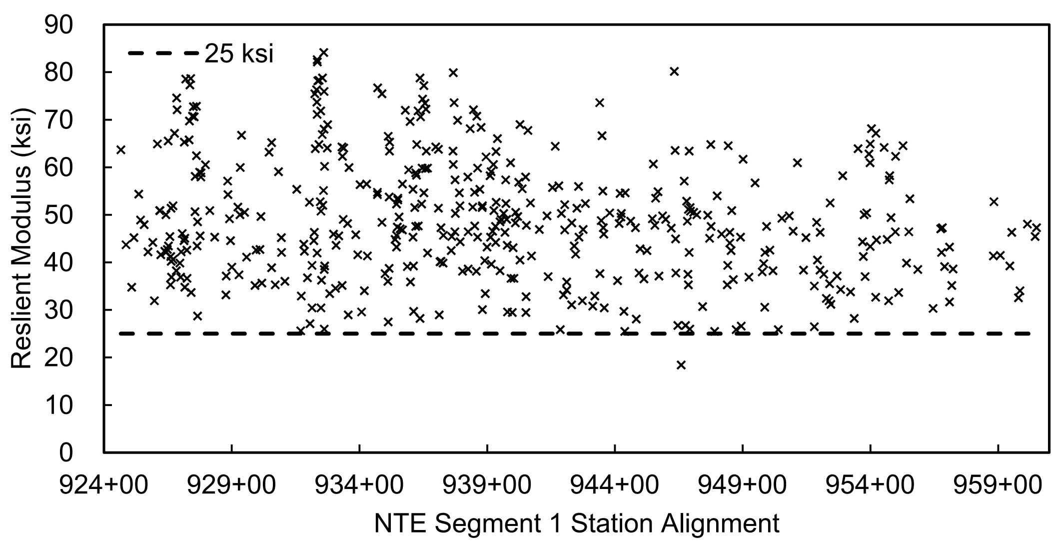


Figure 6.28 Subsegment H Resilient Modulus Correlation from UCS at 8 Days

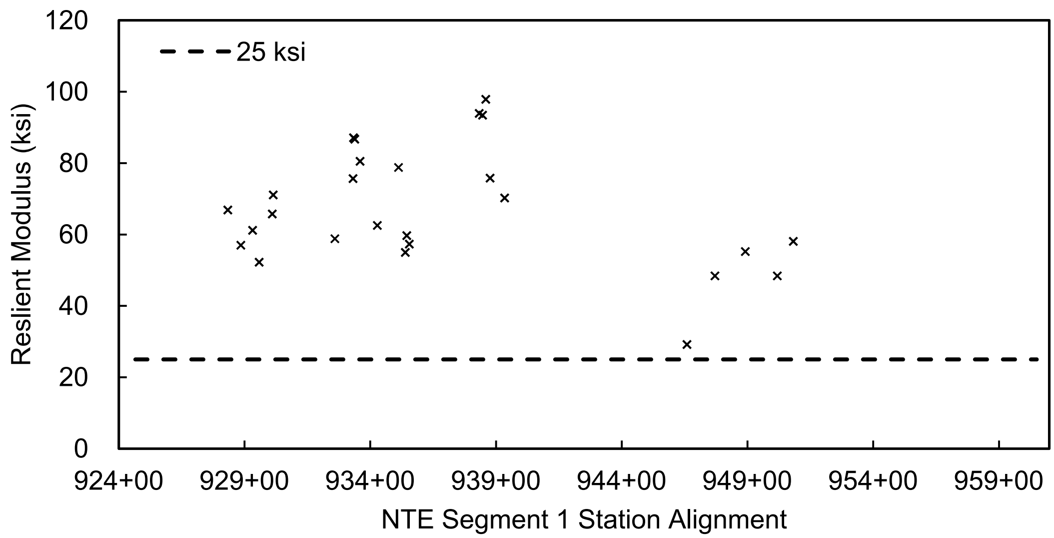


Figure 6.29 Subsegment H Resilient Modulus Correlation from UCS at 16 Days

6.4.10 Summary

The UCS data gathered during the QA investigation confirms the reactivity of the soil with lime in the NTE Segment 1. With increases of strength from 300% to 670%, the LSS in the project area gains significant strength with time. This strength increase with time is confirmed further in Figure 6.30. The figure expresses the average UCS value for the LSS with 1 standard deviation increase and decrease with curing time. Also shown is the average untreated natural clay UCS value. It should be noted again that UCS tests were conducted at 16 days only if the results from 8 day tests were questionable. Due to this reason, the average UCS test result may be lower at 16 day than at 8 days, or the percent of UCS tests below 125 psi may be greater at 16 days than at 8 days.

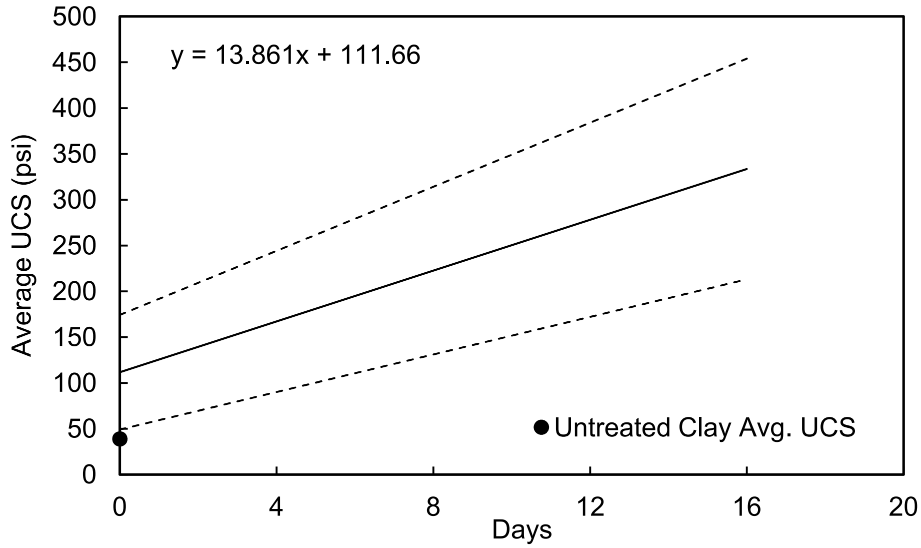


Figure 6.30 NTE Segment 1 Average USC with Time

Table 6.10 Summary of QA Data for NTE Segment 1

Sub-segment	Percent UCS < 125 psi			UCS Average (psi)			
	4 day	8 day	16 day*	In-situ	4 day	8 day	16 day*
A	18.5%	3.2%	0.8%	42	185	291	387
B	39.5%	3.4%	1.0%	38	151	237	320
C	36.5%	5.5%	0.3%	43	155	226	284
D	36.7%	12.0%	6.0%	56	147	229	266
E	24.1%	2.0%	0.0%	48	160	261	227
F	56.8%	9.3%	2.7%	46	133	200	275
G	18.5%	1.0%	0.0%	39	186	302	475
H	11.8%	0.2%	0.0%	42	203	315	461
Overall	31.6%	4.5%	1.6%	39	164	254	332

* Only samples showing questionable results at 8 days were tested at 16 days

6.5 QA FWD Testing

FWD testing was not required in the NTE Segment 1, since no lots failed the initial evaluation. However, FWD tests were conducted in some area as an extra means of evaluation. These test results confirmed the strength and stiffness shown by the UCS testing results.

Figure 6.31 shows the data collected from FWD testing along the I-820 alignment. These tests were conducted on LSS designated for the managed lane. These values are the average surface modulus values for the 6 drops at each station location.

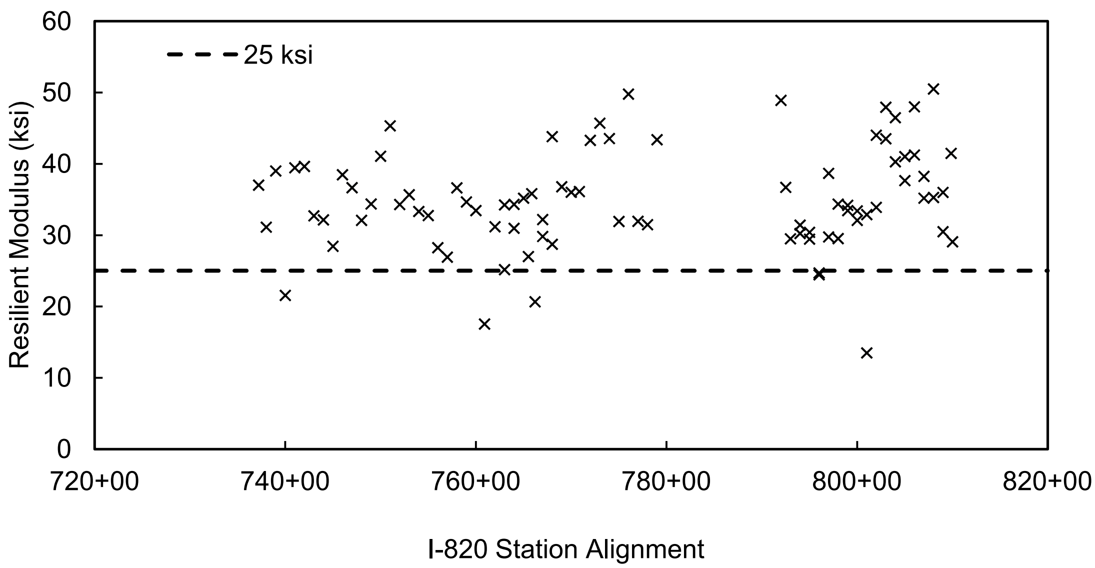


Figure 6.31 FWD Data along I-820

The curing periods for these areas vary from 4 to 9 field days. The thicknesses also vary 46.4 inches to 20 inches. Even at the limited curing period of 4 to 9 days in the fields a significant increase in stiffness can be seen. Unfortunately, additional FWD data was not available. Due to the quantity of UCS testing conducted, additional FWD tests were not required.

6.6 Summary

The organization and data collection for testing of the LSS in the NTE Segment 1 quality program is a prime example for any agency wanting to utilize LSS as a pavement member. The program confirmed the assumed strength and stiffness characteristics of the LSS. This program is an ongoing process that begins in the initial steps of the project and may not end until years after the final days of construction.

At the time of this study, only one lot had to be reconstructed due to poor construction. No lot was deemed failing due to poor reactions with lime. The testing showed an increase in strength and, therefore, an increase in stiffness over time. The QA and QC programs were executed professionally and effectively by the project quality staff.

Chapter 7

Summary, Conclusions and Recommendations

7.1 Summary

During the review of the design data and quality data, it was apparent that the soil in the NTE Segment 1 was reactive with lime. In a few cases, the LSS did not achieve the required target strength; however, it was obvious there was a continuous increase in strength over time. Pozzolanic reactions continue to take place for a period of time after final compaction, which can be seen in Figure 6.30, and the strength of the LSS will increase, leading to greater stiffness. For this reason, construction and preservation of the layers will be important and should not be overlooked during construction and long-term maintenance.

If construction of the LSS is not executed properly, reactions between the lime and soil could be hindered or may not even develop. The use of a LSS subbase is a risky process, as construction of the pavement cannot always wait for all testing to be completed. If a section fails inspection, an entire pavement section could be excavated and reconstructed. However, if the design, construction, and QC program are executed properly, the probability of failing sections decreases to a negligible value.

As stated previously, the 7 day curing at 104°F typically overestimates the 28-day field strength of the LSS. However, the process does not induce a stronger bond within the LSS. The accelerated curing only expedites the bonding that would occur naturally. It is clear the strength of the lime-soil mixture increases with time.

UCS test data is further summarized in Table 7.1. This table shows the 50th, 80th, and 90th percentile values (percent of values above the 125 psi target value) of the 4-, 8- and 16-day tests conducted in each subsegment. Nearly all the 90th percentile values at

8 days achieved the 125 psi target, with Subsegment D falling slightly below the target. At 16-day testing, the 90th percentile values were above the 125 psi target in all subsegments.

Table 7.1 Summary of UCS Test Data with Percentiles

Sub-Segment	UCS (psi)								
	50th			80th			90th		
	4 Day	8 Day	16 Day	4 Day	8 Day	16 Day	4 Day	8 Day	16 Day
A	175	282	390	127.4	203	294.8	111	166.6	246.4
B	144.5	222	304.5	99.4	170	230.8	88.4	144.9	197.9
C	144	215	255.5	103	160	202.2	90.4	136	174
D	136	212	238	87.8	146	189.8	74.8	118.6	146.8
E	165	263.5	204	119	198.4	179.2	87	170	152.2
F	118.5	191	270	89.4	149	195.6	72.2	126.9	148.9
G	180	302	487	126	210.8	410.6	113	175.8	248.2
H	194	303	437	148.6	222	365	122.3	185	325.5
Overall	152	237	310	108	172	222	89	144	222

If the LSS is allowed to cure properly, the mixture will gain strength and continue to gain strength for an unknown period of time. The total time of strength gain for each mixture is unique, but it is estimated that a UCS test carried out after 14 days of accelerated curing at 104°F curing achieves approximately one half to three quarters of its strength (Little 2010). This means that even the 16-day tests still have not achieve full strength capacity. This makes construction and maintenance especially critical, since disturbance from water could hinder the lime/clay reaction.

7.2 Conclusions

This thesis summarized the properties of the native and treated soil, which were tested during the design and construction phases of the NTE Segment 1 project, evaluated the strength and stiffness testing procedures of the LSS, and confirmed the assumed UCS to resilient modulus LSS correlation from Thompson (1966). The following conclusions can be devised from the results of the geotechnical investigation and quality assurance testing:

- UCS testing was an appropriate method to estimate the resilient modulus of the clay found in the NTE Segment 1.
- The use of LSS as a subbase for flexible pavement reduces both the PVR of the soil and decreases the required thicknesses of other pavement layers.
- The strength characteristics of the LSS, determined during construction, confirmed the design assumptions, which were based on the test results from the 8 samples tested during the initial GI.
- Extensive UCS testing conducted as part of the quality assurance program during construction confirmed the lime/clay reaction.
- In regards to the pavement design, a resilient modulus of 25,000 psi and structural coefficient of 0.12 were appropriate structural characteristic assumptions for the LSS.

7.3 Recommendations

In the early beginnings of a project, the engineer must extensively investigate the natural soil within the project area. They should identify the different geological formations and review the typical soils found in those formations. The GI must be properly documented and all required testing should be clear and concise. The preparation of samples for testing should be documented along with the locations from where the samples were taken. The tests to be included in the lime series testing program should be clearly defined: pH, PI series, Modified or Standard Proctor, UCS, and resilient modulus tests. Organization and an understanding of the GI goals are fundamental to developing a good LSS design. The geotechnical investigation process must be methodical and collect the correct amount of information required.

It may be prudent to follow a mechanistic empirical pavement design approach in future projects. The AASHTO 1993 methodology has a few shortcomings; some have been touched upon in this study. Currently, many materials used in pavement design, such as LSS, were not part of the AASHO Road Test. Thus, these materials were not directly assigned structural coefficients. A mechanistic approach would directly consider the resilient modulus of the materials when assessing stresses and strains within a pavement system. The mechanistic approach would eliminate the need to estimate structural coefficients of materials, which were not directly measured during the AASHO Road Test.

During seasonal changes, the native soil can undergo a natural change in moisture content. Since the entire soil active zone cannot be treated with lime, some expansive soil remains in the active zone. The change in moisture content may lead to swelling beneath the LSS, causing changes in applied pressures. Certain precautions were undertaken during the construction of Segment 1 to prevent this situation from occurring. The lime treated area was extended beyond the pavement edge 15 feet or to the future right of way, creating a barrier to prevent moisture fluctuation. However, durability tests for future projects may be beneficial in evaluating the LSS integrity under applied loads and variable moisture contents.

Based on the NTE Segment 1 project’s lime testing procedure and published research, the following improved lime testing procedure is recommended for future highway projects of considerable length.

INITIAL GEOTECHNICAL INVESTIGATION

1. An extensive review and evaluation of existing soil data, such as boreholes and laboratory data from past projects, geologic formations, and soil classification reports.
2. Borehole layout, depth, and sampling depth should be determined considering the following requirements:
   - Locations for boreholes should be determined based on common agency practice but no space between pavement boreholes should be greater than 300 feet along the highway alignment.
   - Borehole locations should be staggered along the width of the highway alignment.
   - Boreholes for pavement should extend to a depth of at least the bottom of the assumed active zone (10 to 20 feet).
   - Samples should be taken continuously or at least at every 3 feet.
3. Locations for lime series testing should be defined based on historical data and documented.
4. Standard Testing for gathered samples should be defined:
   - Visual classification;
   - Atterberg limit tests conducted on every sample;
   - At least one soluble sulfate test and partial sieve analysis conducted per each borehole location;
   - Organic testing should also be considered in areas of concern.
5. Information from Atterberg limit tests can be used in coordination with geological data to group similar soils into individual sections along the project alignment.

6. Based on soil grouping, additional samples for lime series testing can be collected, if need be.

LIME SERIES TESTING

1. Soluble sulfate and organic content tests should be conducted.
2. Sieve analysis should be performed.
3. PI series tests to show the soil is reactive with lime.
4. pH series testing to provide a minimum percent of lime.
5. Standard or modified compaction to provide optimum moisture content.
6. UCS testing on lime treated clay:
    a. Two samples from each location should be mixed with the minimum lime content, 2 percent above the minimum and 4 percent about the minimum using a rotary mixture.
    b. The samples should be allowed to mellow in sealed bags for at least 24 hours.
    c. The lime treated soil should be compacted at optimum moisture content using standard or modified compaction.
    d. Samples should be molded to a 4” diameter and 6” height.
    e. Samples should be cured for 28 days at 73°F or 5 days at 100°F to 106°F.
    f. Samples should be subjected to a 24-hour capillary soaking period after curing.
    g. Each sample should be subjected to unconfined vertical compression until failure.
    h. The average of the two samples should achieve the required compression strength.

i. Optimum lime content should be selected based on compression strength of the samples.

7. Resilient Modulus testing on lime treated clay:
   a. One sample at each location should be mixed with the optimum lime content using a rotary mixture.
   b. The samples should be mellowed in a bag for at least 24 hours.
   c. The samples should be compacted at optimum water content using the method used to compact the samples for UCS testing.
   d. Samples should be molded to the same dimensions as the samples in the UCS tests.
   e. Samples should be cured and soaked in the same manner as the UCS test samples.
   f. Samples should be subjected to resilient modulus testing.
8. The results from the resilient modulus test should be compared to the results of the UCS tests to confirm the strength to stiffness relationship.
9. Optimum lime content for each soil group should be defined and reported to construction.

This testing procedure will help identify the optimum lime content for a soil. The optimum lime content may change along the alignment of a highway project. The value must be determined by the design engineer based on test results and engineering experience.

Since accelerated curing time is unique for each soil, further testing could be conducted to estimate the appropriate accelerated curing time for 28-day strength. Lime treated soil samples can be cured for 1, 2, 3, 4, 5, 6, and 7 days at 104°F. These samples would be subjected to UCS testing and results could be compared to a sample cured for 28 days at 73°F; soaking would not be required. This process could save the quality assurance program time, if the required accelerated curing was less than 5 days.

During construction, every step of the LSS construction must be supervised by an expert who understands the process in detail. The agreed upon process must be followed exactly to insure success. The area of new LSS must be protected from environmental hazards such as pooling water during the curing process. This protection should continue even after the highway and construction is completed. This can be done with a proper drainage system and an extension of LSS to the project right of way or a significant distance determined by the engineer in charge. This extension can protect the structural LSS from being saturated and weakened by environmental causes.

The QC and QA programs should be efficient and well organized. The staff should be well-trained and use state of the art filing programs. The lab used for testing should be well-versed in lime testing, have an experienced staff, and use state of the art equipment. Organization and a strong knowledge of the lime stabilization process will lead to a strong QA/QC program.

The number of UCS tests conducted on the lime treated soil for the NTE Segment 1 was extensive. Many projects will not have the resources to conduct such a thorough quality assurance program. There are alternatives to UCS testing that could prove less expensive and time consuming. UCS strength testing serves as a means to estimate the stiffness of the LSS through correlations. This stiffness could be measured directly in the field through FWD. Other correlations have been established based on results from the California Bearing Ratio. These are two alternatives that could replace UCS testing.

To further substantiate the correlation between UCS data and resilient modulus, additional FWD tests could be conducted 28 days after the 1st LSS lift is constructed. If the FWD is run after the 2nd lift, it is difficult to compare these results with the lab UCS. The stiffness values from these tests could be compared to the UCS laboratory tests, which could serve as another method to confirm strength and stiffness assumptions.

The process for design and implementation of the NTE Segment 1 LSS construction can serve as an example for future projects that want to utilize the strength and stiffness characteristics of the LSS. Understanding the geology, the reactions between lime and the soil, the testing procedures, and the construction process will help the engineer in charge develop a first-rate design and QC/QA program.

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Andrew Domke was born in Bloomington, Indiana in August of 1983. He received a Bachelor of Science degree in Civil Engineering from Purdue University, West Lafayette in December of 2007. After graduating, he worked in Madrid, Spain with Ferrovial Agroman as a geotechnical design assistant until February of 2009. While in Spain, he received additional engineering education at the University of Europe, Madrid. After two years in Spain, he was selected to work on the North Tarrant Expressway Segment 1 and 2, located north of Fort Worth, Texas, as part of a two person geotechnical design team with Bluebonnet Contractors, LLC.

During Andrew's time working on the NTE project, he was admitted to the University of Texas at Arlington in the fall of 2012. Andrew obtained his professional engineering license in April of 2013. He completed his program of work at UTA in the spring of 2014 and earned his Master of Science in Civil Engineering. He is currently working on the NTE Segment 3A highway expansion project as a geotechnical design engineer with North Tarrant Infrastructure, LLC.