VALIDATION AND REFINEMENT OF CHEMICAL STABILIZATION PROCEDURES FOR PAVEMENT SUBGRADE SOILS IN OKLAHOMA – VOLUME II

FINAL REPORT - FHWA-OK-11-02(2)

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16. ABSTRACT

For projects involving a chemically stabilized layer as a part of the structural design of the pavement, it is typical to conduct a mix design to assess the additive content needed to achieve a certain unconfined compressive strength (UCS) and to determine the resilient modulus (M_R) of the stabilized soil. However, there is considerable uncertainty regarding whether the strength and resilient modulus of the field stabilized soil are consistent with design values determined in the laboratory. A purpose of this study was to compare results of field tests and laboratory tests on chemically stabilized soil at different curing times to assess whether a relationship exists between field and laboratory measurements. The goal was to determine if a field testing method could be used to assess whether the strength and stiffness in the field are consistent with laboratory measurements used for design. In addition, numerous other physical and chemical tests were conducted on the soils with an aim to enhance interpretation of UCS and M_R and comparisons to field tests.

Field testing included three devices that are portable, quick, and easy to use. These devices include: the Dynamic Cone Penetrometer (DCP), the PANDA penetrometer, and the Portable Falling Weight Deflectometer (PFWD). Laboratory testing was conducted to determine the unconfined compressive strength (UCS) and resilient modulus (M_R) of laboratory specimens prepared using additive contents that were similar to samples taken from field test locations. To estimate the additive contents in the field samples, a mineralogical test method known as “whole rock analysis” using x-ray fluorescence (XRF) was investigated. Samples mixed in the laboratory were tested to determine the UCS and M_R after curing times of 1, 3, 7, 14, & 28 days. Field tests were conducted at each of the five test sites after curing times that fell within the 1 to 28 day time frame; however, because of construction logistics and weather conditions it was not always possible to match the curing times of laboratory tests or conduct field tests over the full 28 days at every site. Nevertheless, sufficient field data was collected to make meaningful comparisons with laboratory test data.

Mineralogical, electrical, chemical, physical and index property testing (Atterberg Limits, linear shrinkage, Total Specific Surface Area (SSA), etc.) was conducted on the natural soils and the stabilized cured samples to observe the relationship of these properties to stabilized soil strength and stiffness. The effect of curing temperature on stabilized strength gain of soils was also examined. The UCS samples were cured at both room temperature (68°F) and at 40°F, which is the minimum ambient temperature specified for chemical stabilization of subgrades.

Correlations were examined and involved basic soil measurements (mineralogical, electrical, chemical and index properties) and mechanical properties (UCS and M_R), and field test results (DCP, PANDA, and PFWD). Some of the various correlations developed show promise as methods for predicting UCS and M_R based on more simply measured soil properties. Relationships between field and laboratory tests also show promise as a means to evaluate strength and stiffness gains in field stabilized soils. Additionally, lower curing temperatures were observed to have an adverse affect on more reactive clayey soils.

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APPROXIMATE CONVERSIONS TO SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
LENGTH				
in	Inches	25.4	millimeters	mm
ft	Feet	0.305	meters	m
yd	Yards	0.914	meters	m
mi	Miles	1.61	kilometers	km
AREA				
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yard	0.836	square meters	m^2
ac	Acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
VOLUME				
fl oz	fluid ounces	29.57	milliliters	mL
gal	Gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: volumes greater than 1000 L shall be shown in m^3				
MASS				
oz	Ounces	28.35	grams	g
lb	Pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
TEMPERATURE (exact degrees)				
^oF	Fahrenheit	5 (F-32)/9 or (F-32)/1.8	Celsius	^oC
ILLUMINATION				
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
FORCE and PRESSURE or STRESS				
lbf	pound force	4.45	newtons	N
lbf/in^2	pound force per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS				
SYMBOL	**WHEN YOU KNOW**	**MULTIPLY BY**	**TO FIND**	**SYMBOL**
LENGTH				
mm	millimeters	0.039	inches	in
m	Meters	3.28	feet	ft
m	Meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
AREA				
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
VOLUME				
mL	milliliters	0.034	fluid ounces	fl oz
L	Liters	0.264	gallons	gal
m^3	cubic meters	35.314	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
MASS				
g	Grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
TEMPERATURE (exact degrees)				
^{o}C	Celsius	1.8C+32	Fahrenheit	^{o}F
ILLUMINATION				
lx	Lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
FORCE and PRESSURE or STRESS				
N	newtons	0.225	pound force	lbf
kPa	kilopascals	0.145	pound force per square inch	lbf/in^2

*SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380. (Revised March 2003)

Disclaimer

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Summary

For projects involving a chemically stabilized layer as a part of the structural design of the pavement, it is typical to conduct a mix design to assess the additive content needed to achieve a certain unconfined compressive strength (UCS) and to determine the resilient modulus (M_R) of the stabilized soil. However, there is considerable uncertainty regarding whether the strength and resilient modulus of the field stabilized soil are consistent with design values determined in the laboratory. A purpose of this study was to compare results of field tests and laboratory tests on chemically stabilized soil at different curing times to assess whether a relationship exists between field and laboratory measurements. The goal was to determine if a field testing method could be used to assess whether the strength and stiffness in the field are consistent with laboratory measurements used for design. In addition, numerous other physical and chemical tests were conducted on the soils with an aim to enhance interpretation of UCS and M_R and comparisons to field tests.

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Correlations were examined and involved basic soil measurements (mineralogical, electrical, chemical and index properties) and mechanical properties (UCS and M_R), and field test results (DCP, PANDA, and PFWD). Some of the various correlations developed show promise as methods for predicting UCS and M_R based on more simply measured soil properties. Relationships between field and laboratory tests also show promise as a means to evaluate strength and stiffness gains in field stabilized soils. Additionally, lower curing temperatures were observed to have an adverse effect on more reactive clayey soils.

1.0 Introduction

1.1 General

It is common in roadway construction to use chemical additives to modify and stabilize fine grained subgrade soils. Typical additives include Portland Cement, Lime, Class C Fly Ash (CFA), and Cement Kiln Dust (CKD). For projects involving a chemically stabilized layer as a part of the structural design of the pavement, it is common to conduct a mix design to assess the additive content needed to achieve a certain unconfined compressive strength (UCS), and to determine the resilient modulus (M_R) of the stabilized soil. However, there is considerable uncertainty regarding whether the strength and resilient modulus of the field compacted soil are consistent with the laboratory determined soil properties used in design.

A primary purpose of this study was to compare results of field tests and laboratory tests on chemically stabilized soil at different curing times to assess whether a reasonable correlation could be established between field and laboratory measurements. The goal was to determine if a field testing method could be used to assess whether the strength and stiffness in the field are consistent with design values determined in the laboratory. In addition, numerous other physical, chemical and index property tests were conducted on the soils to enhance the interpretation of test results.

Three manually operated in situ tests were used in this study including two hammer driven portable penetrometers and a portable falling weight deflectometer (PFWD). The penetrometers included the dynamic cone penetrometer (DCP) and PANDA penetrometer. The latter is a lesser known device manufactured in France and used extensively for earthwork quality control in Europe. Verification of stabilization in the field with these testing devices could provide engineers with a means to verify the quality of the subgrades upon which our roadways are constructed.

The laboratory and field testing program utilized five roadway construction stabilization sites across Oklahoma. The five sites included various soil types according to AASHTO and USCS classification systems and various chemical additives as follows : Site #1 A-4 (ML) soil stabilized with CFA, Site #2 A-6 (CL) soil stabilized with CFA, Site #3 A-7-6 (CH) soil modified with quicklime, Site #4 A-6 (CL) soil modified with quicklime and then stabilized with CFA, and Site #5 A-4 (ML) soil stabilized with CFA. Soil samples and additives were collected from the field sites to prepare laboratory specimens for strength testing to compare to the results of the field testing.

For this study, a chemical analysis method called “whole rock analysis” using X-ray Fluorescence (XRF) was used to estimate actual chemical additive content of the treated soil samples collected in the field. This testing method has been used extensively in environmental applications, but has not been used to quantitatively measure changes in elemental composition as a result of chemical soil stabilization. The method was found to provide reasonable approximations of additive contents in laboratory prepared soil samples using commercial soil minerals; it was therefore used to estimate the additive amounts obtained in stabilized soil from the five test sites. The chemical additive amounts found using this method were used in preparing the laboratory UCS and M_R samples so that reasonable comparisons of field and laboratory measurements could be made.

In addition to conducting field and laboratory testing to examine strength and stiffness, testing was conducted to determine mineralogical, electrical, chemical, physical and index properties (Atterberg Limits, linear shrinkage, Total Specific Surface Area, etc.) of natural soils and chemically stabilized samples. The effect of curing temperature on stabilized strength gain of soils was also examined. The UCS samples were cured at both room temperature (68°F) and at 40°F, which is the minimum ambient temperature specified for chemical stabilization of subgrades.

Correlations were examined involving basic soil measurements (mineralogical, electrical, chemical and index properties), mechanical properties (UCS and M_R), and field test results (DCP, PANDA, and PFWD). Some of the various correlations developed show promise as methods for predicting UCS and M_R based on more simply measured soil properties. Relationships between field and laboratory tests also show promise as a means to evaluate strength and stiffness gains in field stabilized soils. Additionally, lower curing temperatures were observed to have an adverse effect on more reactive clayey soils.

1.2 Objectives and Tasks

This is Volume II of a two-volume final research report. The first volume titled “Validation and Refinement of Chemical Stabilization Procedures for Pavement Subgrade Soils in Oklahoma – Volume I” (Cerato et al. 2011) together with Volume II represents an extension of a previous research project that resulted in a report titled “Evaluation and Field Verification of Strength and Structural Improvement of Chemically Stabilized Subgrade Soil” by Snethen et al. (2008). The purpose of research described in this volume is captured broadly by the following two objectives:

1. Produce field and laboratory data to understand the evolution of strength and stiffness with curing time, and the relationship between these two soil characteristics in chemically stabilized soil.
2. To better understand the similarities and/or differences between strength and stiffness measurements made in the laboratory and corresponding measurements made in the field after construction.

To achieve these broad objectives, the following tasks were completed:

a. Selected five roadway projects that represent different subgrade soil types, chemical additive types, and climatic conditions across Oklahoma,

b. Collected representative soil and additive samples from project locations for classification, mineralogical, physical, chemical, and mechanical property (UCS, M_R) testing of untreated and treated soils.

c. Conducted laboratory testing to estimate additive contents in stabilized soil obtained from field test sites.

d. Prepared UCS and M_R samples using additive contents determined from measurements on field samples and conducted UCS and M_R tests on samples at different curing times between 1 and 28 days.
e. Following compaction and acceptance of the chemically treated subgrade at field sites, conducted a sequence of strength and stiffness testing using the Dynamic Cone Penetrometer (DCP), PANDA Penetrometer, and Portable Falling Weight Deflectometer (PFWD). Tests were conducted at curing times falling between 1 and 28 days.
f. Established graphical and/or mathematical relationships between curing time, stiffness and strength using field and laboratory measurements. Where appropriate, other soil properties (e.g. Atterberg limits, Specific Surface Area, Cation Exchange Capacity, etc.) were used in developing and enhancing correlations between various engineering soil properties.

In addition to the tasks above that were developed in the original scope of work, the effect of curing time and temperature on the strength gains of stabilized subgrade soils was examined in the laboratory using unconfined testing on samples cured at room temperature and 40°F under similar humidity. The purpose was to gain a better appreciation for the sensitivity of different additive and soil combinations to curing temperature. Possibly this would help to explain differences observed between field and laboratory measurements of strength and stiffness.

1.3 Layout of Report

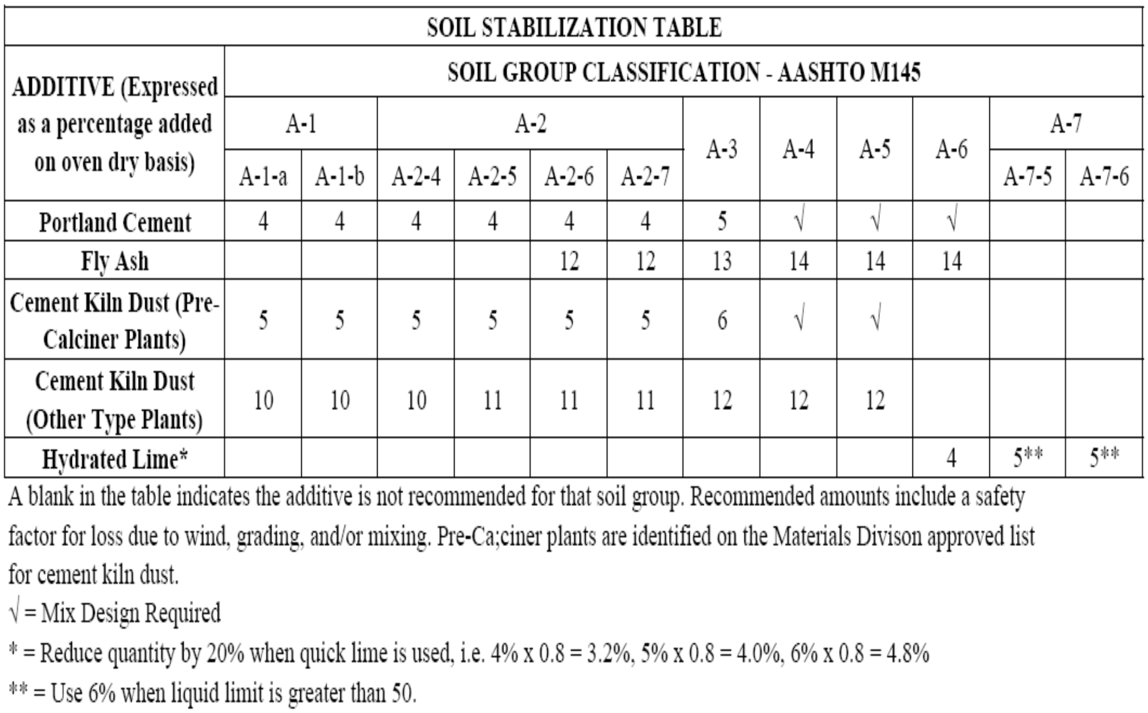
There are four chapters following the introductory chapter. Chapter 2 provides a review of some pertinent literature related to the procedures and tests utilized in this research. Chapter 3 describes the research testing program followed by the results in Chapter 4. Finally, Chapter 5 provides some conclusions and recommendations for practice and further research.

2 Literature Review

2.1 Chemical Treatment of Subgrade Soils

When pavements are underlain by fine-grained soils, it is often cost effective to improve the subgrade soil using chemical additives. Commonly used chemical additives include: cement kiln dust (CKD), class C fly ash (CFA), lime (hydrated and quicklime), and Portland cement (PC). Lime is typically used with high plasticity clays for its ability to reduce plasticity index (PI) and improve workability. On the other hand, CKD and CFA are less effective at reducing PI and typically used with lower plasticity clays and silts. However, CKD and CFA are intrinsically cementitious and generally more effective than lime at improving strength and stiffness, particularly in the short term. (e.g. Solanki et al. 2009a). There are two different types of improvement commonly referred to, modification and stabilization. Modification is used to improve workability of the natural soil through PI reduction and is most often done with lime. The goal of stabilization is typically to achieve a specified increase in subgrade soil strength with a chemical additive. Generally, stabilization is further distinguished from modification by virtue of the fact that improved engineering properties of the stabilized layer are used in the structural design of the pavement. Pavement designs involving modification utilize the unimproved subgrade properties. Many departments of transportation in the United States have developed their own procedures for chemical stabilization of subgrades to streamline the design process. The Oklahoma Department of Transportation (ODOT) has developed OHD L-50 “Soil Stabilization Mix Design Procedure” (ODOT 2009). A primary feature of OHD L-50 is a soil stabilization table that provides recommended percentages for chemical additives based on AASHTO M145 Soil Group Classification (AASHTO 2009). The OHD L-50 recommendations can be seen in Table 1.

Table1: ODOT Soil Stabilization Table (ODOT 2009)



SOIL STABILIZATION TABLE

ADDITIVE (Expressed as a percentage added on oven dry basis)	SOIL GROUP CLASSIFICATION - AASHTO M145											
	A-1		A-2				A-3	A-4	A-5	A-6	A-7	
	A-1-a	A-1-b	A-2-4	A-2-5	A-2-6	A-2-7					A-7-5	A-7-6
Portland Cement	4	4	4	4	4	4	5	√	√	√		
Fly Ash					12	12	13	14	14	14		
Cement Kiln Dust (Pre-Calciner Plants)	5	5	5	5	5	5	6	√	√			
Cement Kiln Dust (Other Type Plants)	10	10	10	11	11	11	12	12	12			
Hydrated Lime*										4	5**	5**

A blank in the table indicates the additive is not recommended for that soil group. Recommended amounts include a safety factor for loss due to wind, grading, and/or mixing. Pre-Ca;ciner plants are identified on the Materials Divison approved list for cement kiln dust.

√ = Mix Design Required

* = Reduce quantity by 20% when quick lime is used, i.e. 4% x 0.8 = 3.2%, 5% x 0.8 = 4.0%, 6% x 0.8 = 4.8%

** = Use 6% when liquid limit is greater than 50.

2.2 Lime Stabilization

One of the chemical stabilizers used in this study was lime. Since the 1950s when lime stabilization began in earnest, extensive research on the subject has been conducted; yet most of the concepts behind stabilization with lime in use today were published by the 1960s (Petry and Little 2002).

The primary chemical reactions that occur when lime is mixed with soil and water are cation exchange and pozzolanic reactions. As hydrated lime [$Ca(OH)_2$] dissolves in water, the calcium (Ca^{2+}) and hydroxyls (OH^-)

disassociate making the divalent calcium ions available for exchange with cations in the diffuse double layer of clay particles. The cation exchange and increased ionic concentration of the pore water results in a contraction of the diffuse double layer, flocculation and agglomeration of particles, and nearly instantaneous reduction in PI with improved workability. Another consequence of dissolution of lime is an increase in pH of the soil-water-lime mixture due to the increased concentration of hydroxyl ions. The increased pH causes silica and alumina associated with clay particles to become soluble. Thus, the lime provides the calcium and a proper chemical environment, while the soil, which acts as the pozzolan, provides the silica and alumina ions necessary to form cementitious compounds (calcium-alumina silicates). These pozzonlanic reactions begin shortly after mixing and continue for a month or longer, but generally slow down after 14 to 28 days. It is the pozzolanic reactions that are responsible for improvements in strength and stiffness needed for stabilization to be effective. A common design process for lime stabilization was published by the National Lime Association (NLA) and involves the determination of the optimum lime additive content using ASTM D 6274 or the Eades and Grim (1966) pH test, preparing samples mixed at the optimum lime content, curing of the compacted samples and then, UCS testing according ASTM D 5102. Most parameters needed for design can be estimated based on the UCS testing; however, rigorous pavement design may require the addition of resilient modulus (MR) testing (NLA 2006).

2.3 Class C Fly Ash

Class C fly ash (CFA) is a chemical stabilizer that is a byproduct of burning coal at power plants. It is the material that is collected from the flue gases in a coal fired furnace (Cokca 2001). The use of CFA in poor subgrade soils has proven to increase strength and stiffness parameters in previous studies and can be a cost-effective alternative to other additives such as lime and Portland cement. One mechanism for added strength in soils from CFA stabilization is cementation of the soil particles. CFA contains all the necessary elements to form a cement mixture, including calcium, alumina, and silica. The cementation reactions begin once the CFA and water are mixed into the soil. Another mechanism at work is cementation due to pozzolanic reactions. When oxides in the CFA dissolve, there is an increase in pH and therefore the clay particles in the soil act as pozzolans, similar to lime, contributing additional alumina and silica to the cementitious reactions.

Generally, CFA stabilization requires more additive material than lime stabilization due to lower amounts of soluble calcium oxide. Studies have shown that to achieve similar results as soil stabilized with 8% lime (by dry weight) for example, as much as 20% CFA is needed (Cokca 2001). The CFA, however, may still prove to be a more cost effective option considering that it is a byproduct, which typically makes it much cheaper than lime on a material cost basis. Also, there is still a large amount of CFA that is placed in landfills as waste, resulting in both a monetary and environmental cost for disposal (Edil et al. 2006). With designers taking a more sustainable approach in recent years, the appeal of CFA may continue to increase, possibly resulting in even lower costs while providing environmental benefits. In a study involving four different Fly Ashes (FA) from Wisconsin it was found that the California Bearing Ratio (CBR) of soft clays increased 4 to 8 times over the raw soil strength by adding 10% to 18% FA, respectively (Edil et al. 2006).

2.4 Whole Rock Analysis using X-Ray Fluorescence

X-Ray Fluorescence (XRF) Spectrometry can be used to determine the presence of various elements within soils, sediments, water sources, and even food items. In soils, the presence of atomic elements such as calcium, can be measured accurately at concentrations representing 0.01–100% of total content (ALS Laboratory Group 2010). This analysis is performed by converting the material to glass and exposing the solid glass sample to an x-ray source. The elements in the solid sample emit a specific x-ray signature, unique to that element, and by measuring the intensity of corresponding wavelengths it is possible to determine quantities of various element in the sample (Hazardous Waste Consultant 2007). With XRF technology, it is possible to quickly perform a multi-element analysis of a sample.

In addition, the analysis captures elements in different forms, including ions, carbonates, oxides, and organic matter within the soil (Baranowski et al. 2002). The XRF analysis has been used extensively in various environmental applications because of its ease of use and economical benefits compared to other elemental analysis techniques. In environmental applications, XRF has been used to detect trace elements in drinking and surface waters, with the advantages of low cost, low detection limits, and specimen preparation simplicity (Zawisza and Sitko 2006). The simplicity and accuracy of XRF has even been utilized for field testing in environmental applications. The ability to measure trace elements is important in many different fields, and XRF has been used to measure changes in chemical composition in soils for monitoring of pollutants and also for agricultural needs (Baranowski et al. 2002).

The ability of XRF analysis to measure trace elements in multiple applications is extensive in the literature; however, it has not been utilized to measure chemical additive content in stabilized soil subgrades. The existing fields in which XRF analysis are utilized suggests that it should prove useful as a soil additive content determination tool for pavement applications. It is for these reasons that the XRF technology was used to determine the elemental constituents of soils (untreated and stabilized) and additives in this study. The objective was to determine additive contents in field samples in order to make laboratory samples with similar amounts of additive for strength (UCS) and stiffness (M_R) testing. This allowed for a more reasonable comparison of strength and stiffness determined from laboratory and field measurements.

2.5 Dynamic Cone Penetration Testing

The DCP, shown in Figure 1, is a portable penetration apparatus that is used to test in situ soil strength. The device consists of a 0.785-inch diameter 60^o conical tip connected to rods that are driven into the soil by blows from a 17.6-lb. (8-kg) hammer dropped 23 inches onto an anvil (Burnham and Johnson 1993). The conical tip is advanced into the ground vertically by blows of the hammer, and after each blow a penetration depth measurement is recorded. The test results are typically reported as penetration per blow (mm or in./blow), or Dynamic Cone Index (DCI), and can be presented versus depth from ground surface (e.g. Miller 2000). A lower value for DCI indicates stronger soils. The DCP has been used to effectively test natural and chemically stabilized subgrade soils for roadway construction (e.g. Burnham and Johnson 1993, Miller 2000).

It has been shown that the DCI values obtained from the DCP testing correlate well with California Bearing Ratio (CBR) values (Livneh 1989). While the DCP involves dynamic loading and the CBR involves static loading, the resistance to penetration in both cases depends on the soil strength and hence it would be expected that such a correlation would be good (Miller 2000). The correlation between DCI and CBR values has also been investigated for stabilized soils with similar results. Misra et al. (2006) showed that the correlation of DCP results to CBR values for Class C Fly Ash stabilized soils was reliable and could be used in the field to quickly determine CBR values of clayey soils. CBR values obtained from DCP testing have also been used to determine M_R values of stabilized soils for pavement designs (e.g. Snethen et al. 2008). Also, it has been shown that DCP testing and correlated M_R values from this testing may be a good indicator of the long term performance of stabilized soil layers (Snethen et al. 2008).

Results of DCP testing in a laboratory have been compared to laboratory UCS testing. Enayatpour et al. (2006) calibrated DCI values (mm/blow) of cement and lime stabilized soils measured in a laboratory setting to the results of laboratory UCS testing. They found that strong correlations existed but recommended field verification.

Figure 1: Dynamic Cone Penetrometer

2.6 PANDA Penetrometer Testing

The PANDA penetrometer is a dynamic cone penetration apparatus that is advanced into the soil layer by blows from a 4.4-lb. (2-kg) dead blow hammer. This penetration device is unique compared to other penetration devices in that it does not utilize a standard drop height. Both the height of drop and applied energy can vary during testing. This is possible due to two different measuring instruments utilized in the device; the speed of impact of the hammer on the head of penetrating rod is measured by an accelerometer located in the head of the device and the depth of penetration into the soil is measured by a retractable tape similar to a strain gauge (Langton 1999). The advantage is that the force of the blow can be adjusted to match the strength of the material being penetrated.

The PANDA penetrometer is shown in Figure 2. The result obtained is called dynamic cone resistance (q_d) calculated using the Dutch formula (Cassan 1988) in Equation 1:

$$q_d = \frac{1}{A} \cdot \frac{0.5MV^2}{1+\frac{P}{M}} \cdot \frac{1}{x_{90^\circ}} \qquad \text{(Equation 1)}$$

Where:

x_{90° is the penetration due to one blow off the hammer (90° cone) (cm)

A is the area of the cone (cm^2)

M is the mass of the striking hammer (kg)

P is the weight of the struck mass (kg)

V is the speed of impact (of the hammer) (cm/s)

q_d is the dynamic cone resistance (MPa)

Most published correlations (e.g. Langton 1999) between PANDA "q_d" and other soil parameters have been developed in France and England, as the device was developed in France. In Oklahoma, Miller and Snethen (2006) reported on a series of DCP and PANDA tests in a granular backfill and found that DCI and q_d were strongly correlated, a somewhat expected result. Also, Snethen et al. (2008) reported that the PANDA penetration tip resistance is a good indicator of the strength of stabilized soil layers and may be useful as a quality control testing device to ensure that strength gains intended by a soil stabilization design are adequately met in the field (Snethen et al. 2008).

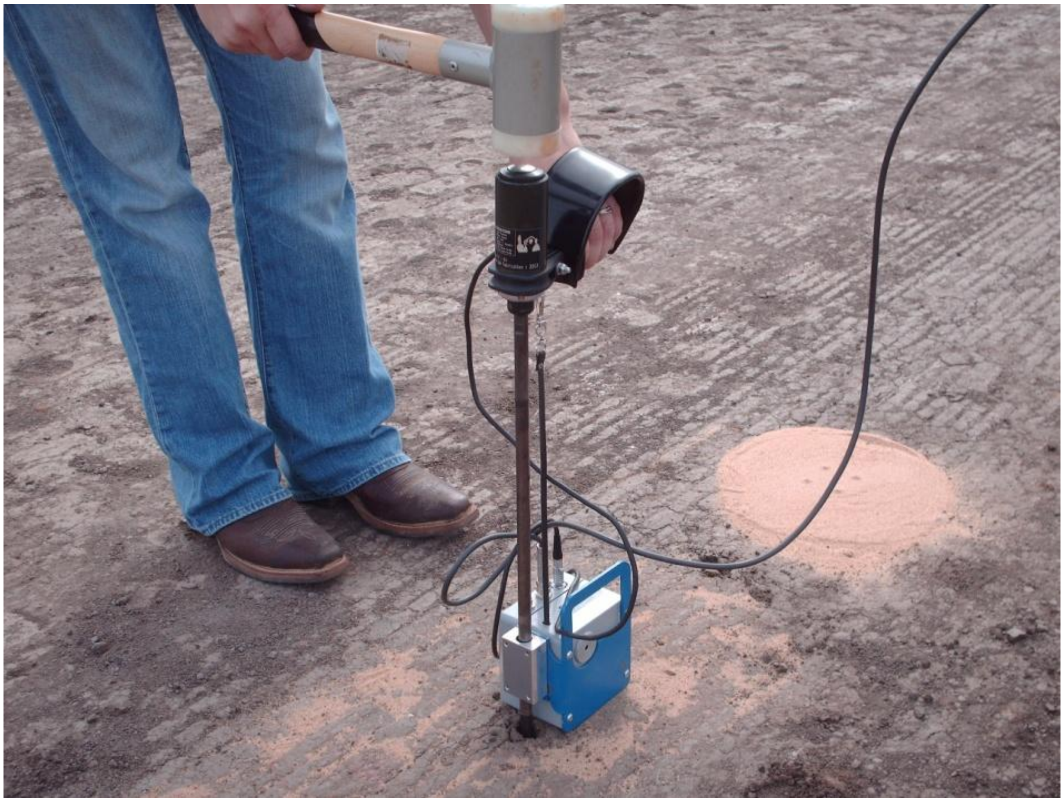


Figure 2: PANDA Penetrometer

2.7 Portable Falling Weight Deflectometer Testing

The Portable Falling Weight Deflectometer (PFWD) measures the displacement of a 12-inch diameter steel plate subjected to a dynamic load from dropping a 22-lb. weight a height of 28 inches. Similar to full-scale FWD, the calculated result of PFWD measurement is a dynamic elastic modulus (E_{vd}) of a subgrade layer.

A number of researchers have studied various factors that influence the PFWD test results (Van Gurp et al. 2000, George 2006, George et al. 2006, Lin et al. 2006, Kim et al. 2007, Nazzal et al. 2007, Mooney and Miller 2009). These studies revealed that the results of PFWD tests depend on several factors including the type (manufacturer) of device, plate size, loading type (weight and load duration), sensor configuration, soil type, soil density and moisture content, and method of interpretation. Further results of PFWD tests from a given device were found to correlate reasonably well with results of DCP tests and laboratory test parameters including California Bearing Ration (CBR) and resilient modulus.

According to the study by Nazzal et al. (2004), the influence depth of the LFWD (or PFWD) ranged from 10.5 to 11 in. (270 to 280 mm), depending on the stiffness of the tested materials. Also based on field testing results, they concluded that the repeatability of the LFWD depends on the stiffness of the tested material. They observed poor repeatability for weak subgrade layers and better repeatability for stiff and well-compacted layers.

In the study conducted by Mooney and Miller (2009), the PFWD was used to investigate the stress–strain response within the soil during PFWD loading. Also they examined the appropriateness of using homogeneous, isotropic, linear elastic half-space theory for soil modulus estimation. They concluded that the soil type governs the contact stress distribution between the soil surface and loading plate. However, displacements predicted using homogeneous, isotropic, linear elastic assumptions do not match well the strains measured in-situ. An exponential modulus function was found to produce a better match between experimental and theoretical elastic strains.

The PFWD test is a simple and quick in-situ testing method and may provide a good measure of performance of stabilized pavement layers; however, a number of factors affect the modulus (Snethen et al. 2008). Snethen et al. (2008) observed that E_{vd} increased with curing time for stabilized soil in the field; however, the rate of increase of E_{vd} was 50 to 90% lower compared to increases in laboratory measured modulus. Figure 3 shows the PFWD used in the current study.

Figure 3: Portable Falling Weight Deflectometer

2.8 Effects of Curing Temperature on Stabilized Soil Strength

Specifications for soil stabilization using chemical additives typically require that construction occur at or above certain ambient temperatures in order to assure proper curing of the stabilized soil. The ODOT 2009 specifications require that application of chemical additives for subgrade stabilization occur when the air temperature is at 40°F and rising and application of these additives for subgrade modification occur at 33°F and rising. In the Unified Facilities Criteria for Soil Stabilization for Pavements (U.S. Department of Defense 2004) it is stated that chemical reactions that occur in lime-stabilized soils will occur slowly unless temperatures are at 60°F (16°C) and that temperature, time, and moisture content are the most important factors for lime stabilization of soils. Similarly, for cementitious soil stabilizers the temperature needs to be at or above 40°F.

Little et al. (2000) states that the reactions associated with lime stabilization are dependent on curing conditions such as temperature, since the sustained pozzolanic reactions occur over a long period of time. Strength gains due to lime stabilization at low temperatures (20-35°F) over short periods of time, a few days to a couple of weeks, have been shown to be minimal compared to the strength gains over the same period of time at higher temperatures (50°F) (George et al. 1992). Limited by these temperature guidelines, most stabilization work in the U.S. where marginal soils require stabilization cannot be conducted in the months of November through mid March and stabilized soils typically need to be covered by a paved layer by the beginning of December or the start of winter (Daniels and Janardhanam 2007).

While the effects of temperature on chemical stabilization of soils are well established, it was desired for the current study to conduct some limited testing to investigate the sensitivity of the soil-additive combinations to curing temperature. The goal was to possibly provide additional insight into comparisons of field and laboratory measurements of strength and stiffness.

2.9 Resilient Modulus of Stabilized Soil

The structural design of pavement layers is based on the properties of the subgrade soil extending to about two feet beneath the structure (Croney and Croney 1997). The resilient modulus, M_R, is an important parameter that characterizes the elastic behavior of the supporting soils used in the design of pavements (AASHTO 1993, 2007). The M_R is obtained from a repeated load triaxial test on a cylindrical specimen of soil, and is mathematically defined by dividing deviator stress by the recoverable strain. Paving materials are not fully elastic, but experience some permanent deformation after each load application. However, if the load is small compared to the strength of the material and is repeated for a number of times, the deformation under each load repetition becomes nearly completely recoverable, proportional to the load, and can be considered as elastic (Huang 1993).

Sample preparation and M_R testing procedures for subgrade, subbase and base materials are discussed in the AASHTO T 307-99 standard test method. The test procedure simulates the stress state experienced by material in a pavement structure subjected to oscillating traffic loads. This test method covers the testing of undisturbed and compacted subgrade soils and untreated base and subbase materials. According to AASHTO T 307, the M_R value calculated from the above mentioned testing procedures is an indicator of the elastic modulus of the material, recognizing certain nonlinearities.

There has been considerable research on the influence of factors that affect the laboratory determination of M_R. For example, Mohammad et al. (1994) addressed the effects of testing procedures and type of measurement system on results of M_R tests. It was found that the testing procedure and measurement system, stress history, soil fabric, moisture content, and possible pore pressure development can substantially influence the test results. Generally, the M_R of unconsolidated soil is strongly dependent on the confining pressure and deviator stress (e.g. Hicks and Monismith 1971, Maher et al. 2000, Titi et al. 2006, Mokwa and Akin 2009) and matric suction (e.g. Khoury and Zaman 2004, Gupta et al. 2007, Khoury et al. 2010).

A number of researchers have studied the resilient modulus of stabilized soil (Tuncer and Basma 1991, Parsons and Milburn 2003, Aydilek and Arora 2005, Edil et al. 2006, Solanki 2009, Snethen et al. 2008). Tuncer and Basma (1991) carried out research on lime stabilization of cohesive soils. The testing results showed a general trend of increase in M_R values with increasing curing periods for all additive fractions.

Parsons and Milburn (2003) conducted a study to evaluate long-term performance of chemically treated soil representing different soil classifications, namely CH, CL, ML, and SM mixed with lime, cement, and CFA. It was observed that stiffness increases with time for selected soil/additive combinations, particularly for CL soils mixed with lime and cement. In addition, stiffness values (moduli) for fly ash treated soils were generally lower than those for lime and cement, and showed limited or no increases with time.

Arora and Aydilek (2005) performed a set of tests including unconfined compression, CBR, and M_R to investigate the effect of fines content, curing period, molding water content, compactive effort, cohesion, and cement or lime addition on geotechnical parameters of fly ash amended highway bases. Results of the research show that the strength of a mixture is highly dependent on the curing period, the compactive energy, cement content, and water content at compaction.

Edil et al. (2006) performed research to evaluate the effectiveness of different fly ash contents ranging between 10-30% on California Bearing Ratio (CBR) and M_R. They observed higher M_R when the amount of the fly ash was increased up to 18%. In this study, they also investigated the effect of curing time on M_R on one soil and two fly ashes. They concluded that between 7 and 14 days the M_R increased modestly. However, between 14 and 56 days, the M_R increased by 20–50%. Thus, fly ash stabilized subgrades should stiffen over time, resulting in an increase of pavement support.

According to the study by Solanki et al. (2009b) on engineering properties of silty clay stabilized with different percentages of lime and CFA, both stabilizers improve M_R and modulus of elasticity. Using the stepwise linear regression method, they developed an equation relating M_R and engineering index properties. They concluded that measured values were well correlated with predicted values.

Pinilla et al. (2011) conducted a research study on the effect of soil properties, additive type and curing time on the M_R of chemically stabilized soils;

this work was part of a larger project involving strength and stiffness testing of stabilized soil subgrades at five sites in Oklahoma (Snethen et al. 2008). They used CKD and CFA as stabilizers for five selected construction sites. They developed a power model for M_R evolution with time; M_R improvement for 28 days cured samples was 7 to 46 times higher than that for untreated samples. They concluded that the power regression curves exhibit a good fit to the M_R improved values.

3 Methods and Materials

3.1 Introduction

This chapter describes the test site soils and additives and the testing methods used in this research to determine the various laboratory and field soil parameters investigated

3.2 Test Sites, Soils, and Chemical Additives

Untreated and treated soil samples were obtained from five chemical stabilization roadway construction sites. A list of the test sites and corresponding soil series is shown in Table 2. The approximate locations of the field testing sites are shown in Figure 4.

At each field test site the chemical additive(s) used were sampled at the beginning of the stabilization process when the material arrived on site. The two additives in this study were Class C Fly Ash (CFA) and Quicklime.

Table 2: Field Test Site Locations and Corresponding Soil Series

Site Number	Soil Name	County	Soil Series Name
#1	US 281	Canadian	Norge Silt Loam
#2	Penn Ave.	Logan	Renfrow Silty Clay Loam
#3	US 177	Kay	Lela Clay
#4	SH 7	Johnston	Chigley-Rock Outcrop Complex
#5	US 81	Canadian	Gracemore Loamy Fine Sand

Figure 4: Test Site Locations

3.3 Field Testing

3.3.1 Introduction

Field testing using the DCP, PFWD and PANDA Penetrometer was performed at the roadway construction sites. Prior to addition of the chemical stabilizer, a location along the subgrade alignment was selected as a representative area. This location was marked to ensure that subsequent testing was performed in the same general locations. At each location, three test zones approximately 50 feet apart were identified as shown in Figure 5.

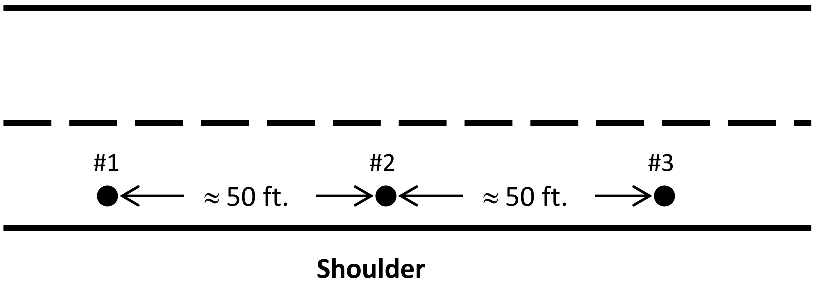


Figure 5: Field Testing Layout

3.3.2 Dynamic Cone Penetration (DCP) Testing

To monitor the effects of chemical stabilization on the strength and stiffness gains of subgrade soils during curing, the DCP test was conducted at each site on both the untreated and treated soil at specific curing times after compaction. The DCP was performed at each testing point according to the procedure given in the American Society for Testing and Materials (ASTM) D6951 “Standard Test Method for Use of the Dynamic Cone Penetrometer in Shallow Pavement Applications” (ASTM 2009).

From each DCP test the depth of penetration into the soil following each blow was recorded, typically for depths of penetration ranging from 20 to 25 in. An average Dynamic Cone Index (DCI) in units of mm/blow was determined for treated subgrade layers and untreated subgrade layers of similar thickness. The average DCI for each curing time for a given site was computed from results obtained at the three testing locations.

3.3.3 PANDA Penetration Testing

The PANDA penetration tests were conducted to monitor the effects of chemical stabilization of subgrade soils. After the DCP testing was finished, the

PANDA penetration testing was performed at the same three DCP test points, but outside of the DCP influence zone. Due to the small diameter of the DCP penetration rod this distance was estimated at 10 inches. After testing, the data collection unit was connected to a computer, test data was uploaded, and using the software provided by the manufacturer a value for Average Tip Resistance, q_d, was computed.

Similar to the DCI calculation, the average tip resistance was calculated for the stabilized layer at each testing point and for corresponding untreated layers. Then an average for each curing time for a given site was computed using values obtained from the three testing points.

3.3.4 Portable Falling Weight Deflectometer (PFWD)

The PFWD test was conducted at each site on both the untreated and treated soil at various times after compaction during the 28-day curing period. The PFWD testing was performed in accordance with ASTM E2583–07 “Standard Test Method for Measuring Deflections with a Light Weight Deflectometer (LWD).” The PFWD was used to determine the elastic modulus of the subgrade. In this project, a Zorn 2000 device with a 12-inch diameter rigid plate was positioned on the ground and a 22-lb. sliding hammer dropped 27 inches produced the impact forces. The measured deflection of the ground is combined with the applied load to calculate the elastic modulus (E_{vd}) using conventional Boussinesq static analysis.

3.4 Whole Rock Analysis using XRF

The Whole Rock Analysis using XRF was performed by ALS Laboratory Group in Reno, NV. Samples of the untreated soil as well as the chemically stabilized soil from each field test site were prepared by processing over a U.S. #80 sieve and approximately 50 grams of soil was sent to ALS. Thirteen different elemental contents were determined, as well as Loss on Ignition (LOI). The resulting percentages (by dry weight) were provided by ALS, and CaO was used as the elemental compound to calculate the percentage of chemical additive in the field samples. This was done simply by taking the difference (increase) in CaO (% of dry weight) between the treated and untreated soil and dividing by the amount of CaO in the additive.

3.5 Classification and Physical Property Testing

3.5.1 Particle Analysis

A particle size analysis was performed on each soil collected from the field test sites. Testing was performed in general accordance with the American Society for Testing and Materials (ASTM) D 422-00 “Standard Test Method for Particle-Size Analysis of Soils” (ASTM 2009).

3.5.2 Harvard Miniature Compaction Tests

Compaction tests were performed in general compliance with the ASTM D 4609-01 “Evaluating Effectiveness of Chemicals for Soil Stabilization” (ASTM 2010), with a major modification. Instead of the spring-loaded kneading compaction method, a miniature drop hammer was used. This compaction hammer was developed to produce constant compaction energy for the Harvard Miniature (HM) mold that could be calibrated to the compaction characteristics of the Standard Proctor compaction method (Khoury and Khoury 2008). The diameter of the rammer was chosen so that the ratio of the diameter of the rammer to the diameter of the mold is approximately the same as the ratio in the standard Proctor test. The guide sleeve has vent holes in both ends to prevent any pressure build up. The free fall distance of the rammer (0.863 lbs) was kept constant at 12 inches (30.48 cm), similar to the distance in the ASTM D 698-91 (or AAHSTO T-99) test method. Khoury and Khoury (2008) performed tests on 4 different soils and determined that to achieve the best match to the Standard Proctor density, 10 drops per five compacted layers with the small drop hammer should be used. A similar result was obtained by Cerato et al. (2011, Volume I of this report) with exception that for two A-7-6 soils, 5-6 blows per layer was optimal for matching the Standard Proctor results.

Figure 6 shows the Harvard Miniature compaction mold and drop hammer. While the total theoretical energy applied to the HM sample using 10 blows per layer is different than the theoretical energy applied to a sample in a standard Proctor test (ASTM D 698), experience has shown that consistent compaction curves can be produced with the small HM drop hammer method that reasonably match the standard Proctor curves.

The compaction tests performed in this study were used to create moisture-density curves for each treated soil at the additive content obtained from testing field samples using the XRF Whole Rock Analysis Method. To prepare compaction test specimens, approximately 140 g of air-dried soil was dry mixed with the appropriate amount of additive and then the required amount of deionized water was thoroughly mixed with the dry components. The mixture was then compacted in five layers in the HM device using 10 blows per layer. Once the fifth layer of soil was compacted, extra soil was trimmed from the top and bottom of the mold and used to determine the moisture content of the sample. The soil specimen was then removed from the mold with a mechanical extractor.

The compacted sample was then removed from the mold and this process was repeated for multiple targeted moisture contents to define a moisture-density curve and determine optimum moisture content (OMC) and maximum dry density for each soil. This was done for both the chemically stabilized and untreated soil samples.

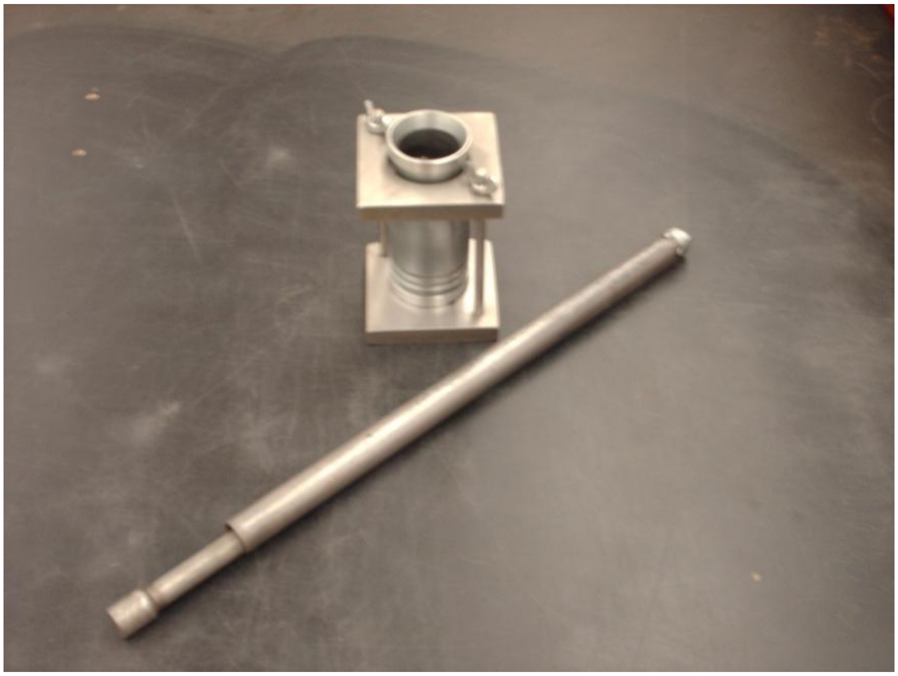


Figure 6: Harvard Miniature Compaction Mold and 12" Drop Hammer

3.5.3 Atterberg Limits Testing

Atterberg limit tests were performed on the untreated soils as well as, chemically treated soils from UCS specimens cured for 14 days. The Atterberg Limits tests were performed in accordance with ASTM D 4318-00 “Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils” (ASTM 2009).

3.5.4 Shrinkage

3.5.4.1 Linear Shrinkage

This test method was first introduced by the Texas Highway Department in 1932 (Heidema 1957) and currently appears as a British Standard, BS 1377 (1990) and a TxDOT Standard, TEX-107-E (1999). The difference between the two standards is the shape of the linear shrinkage mold; the British Standard uses a half of a brass pipe with boxed edges and the TxDOT standard uses a rectangular prism box mold. Approximately 150 grams of soil passing a #40 sieve were used to perform the test procedure. First, the soil sample was mixed with deionized water to approximately the Liquid Limit. A portion of the soil was placed in either a semi-circular linear bar mold approximately 6 inches long and 1 inch in diameter (BS 1377) or 5 inches long by 0.75 inches in width and height (TEX-107-E). The soil was placed in three layers and tapped against a flat surface in between the layering to remove air bubbles. The mold was allowed to air dry. Typically, no length and mass readings are taken until the sample has been oven-dried; however, if the shrinkage limit value is required, it was necessary to take intermediate readings in order to determine where volume change ceases, while water content is still decreasing. Therefore, mass and length measurements were taken several times a day until the length did not change measurably. At that point, the mold was oven-dried for 24 hours at 110 ± 5°C. After drying, the mass and length measurements were taken once more. The length of the soil sample was measured using a digital caliper. The average length was used to calculate the linear shrinkage. The linear shrinkage was calculated by the following equation:

$$LS=100^{*}\left(1-\frac{L_{avg}}{L_o}\right) \qquad \textbf{(Equation 2)}$$

Where:

LS = Linear shrinkage (%),

L_{avg} = Average final length of the soil inside the linear bar mold (in),

L_o = Original length of the linear bar mold (in).

3.5.4.2 Shrinkage Limit

The linear shrinkage measurement of soil was used to determine the shrinkage limit. The test method was performed in general accordance with the British Standard (BS 1377: 1990, Test 5), which is an alternative to the Mercury Method (ASTM D 427-00) “Standard Test Method for Shrinkage Factors of Soils by the Mercury Method.” This test was also performed in conjunction with the Linear Shrinkage test detailed previously. The changes in length measured during the air-drying period were plotted versus the water content, where the shrinkage limit was described as the first water content at which no variation in the length of the soil sample was observed. The determination of the shrinkage limit from the linear shrinkage is presented in Figure 7.

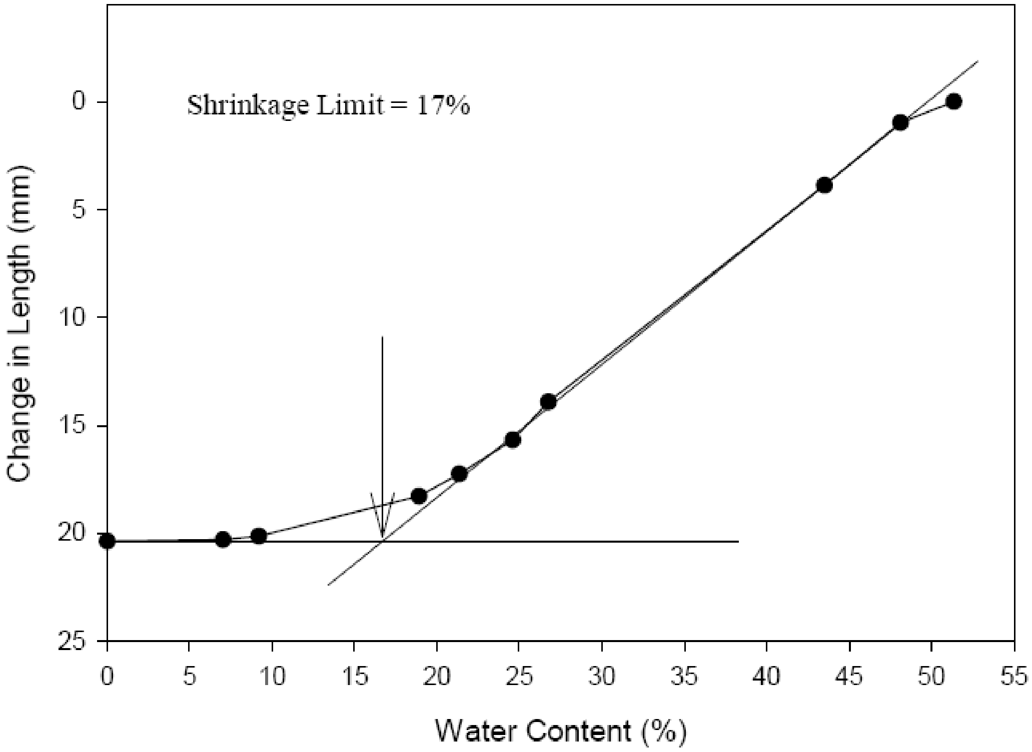


Figure 7: Determination of Shrinkage Limit from Bar Linear Shrinkage Test

3.6 Mechanical Property Testing

3.6.1 Unconfined Compression Strength Testing

To estimate the strength of soils stabilized with chemical additives, unconfined compression strength (UCS) testing was performed. This testing was done in general accordance with ASTM D 2166-00 “Standard Test Method for Unconfined Compressive Strength of Cohesive Soils” (ASTM 2009). Samples were prepared by mixing soil collected at the field test with the appropriate amount of chemical additive estimated using the XRF analysis as described previously. Samples were compacted using the HM procedure described in Section 3.5.2 at the OMC. After the specimens were compacted and trimmings had been collected for moisture content determination, the samples were extracted from the HM mold, wrapped in plastic wrap, sealed with masking tape, labeled with an identification number and sealed in a plastic Ziploc bag. The prepared samples were then placed in either the humid room to cure at 100% relative humidity for room temperature curing conditions, or they were placed in a refrigerator containing a humidity device to cure at 90%+ humidity and 40°F ± 2°F. To assess repeatability, three UCS samples were made prepared for each curing time, soil, and curing temperature condition. The three samples had to be within 0.5% of the targeted OMC and the range between the three samples was not to exceed 0.75% moisture content.

After the specimens had cured for the designated curing time, it was removed from the plastic wrap and prepared for UCS testing. The specimens cured at the 40°F temperature condition were removed from the refrigerator and were allowed to equilibrate back to ambient room temperature while remaining sealed. The specimens were tested in compression using strain-controlled loading at a strain rate of 2% per minute. During testing, the strain was determined using a dial gage with a resolution of 0.001 inches and the axial load applied to the specimens was measured using a digital load cell with a resolution of 1 lb. The values of the axial displacement and load recorded during testing were used to create stress-strain curves. After a sample had been tested and processed, it was pulverized to pass a #40 sieve then saved for Atterberg Limits and Bar Linear Shrinkage testing.

3.6.2 Resilient Modulus Testing

Laboratory resilient modulus determination procedures are presented in this section, including sample preparation and testing method. Raw soil samples and additives were collected from the five test sites. After being air-dried, mortar and pestle were used to break up large particles. Soil samples were processed over a #4 sieve. Duplicate stabilized samples were prepared with field additives at the estimated field additive content and optimum moisture content (OMC). Samples were compacted to 95% of maximum dry density based on moisture density curves.

A standard mold, 4 inches in diameter and 8 inches in height, was used for compacting soil mixtures in five volume-controlled layers at the specified moisture content and dry density in accordance with the kneading compaction procedure presented in AASHTO T-307-99. These procedures were the same for both treated and untreated specimens. Specimens were wrapped with cellophane, placed in Ziploc bags, and cured in a humid room for periods of 1, 3, 7, 14, and 28 days. Untreated specimens were similarly prepared, placed in the humidity room and tested the following day.

Resilient modulus testing was conducted in accordance with AASHTO T 307-99. A computer controlled servo-hydraulic testing machine was utilized and during testing, deformation was recorded using two Linear Variable Differential Transformers (LVDTs). For raw specimens, a triaxial chamber with external LVDTs was used, and for stabilized specimens, a chamber with internal LVDTs was utilized. The internal LVDTs have more limited stroke, but greater accuracy to detect the very small deformations that can occur in stabilized soils. Vertical displacement and applied load readings were digitally recorded from which the deviator stresses and resilient strains were calculated.

The M_R was calculated by measuring the total resilient (recoverable) axial deformation of the specimen in response to the loading in the form of 15 stress sequences, each having 100 loading cycles using a cyclic haversine shaped stress pulse with duration of 0.1 seconds and resting period of 0.9 seconds. Loads and displacements recorded for the last five cycles of each sequence were used to determine fifteen corresponding values of the M_R.

Tests began with conditioning using a confining stress of 6 psi and deviatoric stress of 4 psi for 500 cycles. Conditioning reduces the imperfect contacts between end platens and specimens (Puppala et al. 1999). After conditioning, the specimen was subjected to several combinations of confining stress (nominally 2, 4, 6 psi) and deviatoric stress (nominally 2, 4, 6, 8, 10 psi) amounting to 15 different test sequences from which M_R was determined.

3.7 Mineralogical Property Testing

3.7.1 Total Specific Surface Area (SSA) using the Ethylene Glycol Monoethyl Ether (EGME) Method

Testing for the Total SSA follows the method presented by Cerato and Lutenegger (2002) in their study “Surface Area and Engineering Properties of Fine-Grained Soils.” The testing was performed on oven-dried samples of soil that passed the #40 sieve size.

3.7.2 Carbonate Content Testing

Carbonate content testing was performed according to the method presented by Dreimanis (1962) using the Chittick Apparatus. The Chittick Apparatus measures the amount of carbonates present in the soil by measuring the amount of carbon dioxide that evolves from carbonates reacting with hydrochloric acid. In order to perform this test a sample of soil passing the #40 sieve size was oven-dried for at least 24 hours at 110°C ± 5°C.

3.7.3 Sulfate Content

The sulfate content of the untreated soils was determined using a colorimetry sulfate analysis. The testing was performed according to the methods set forth by the Oklahoma Department of Transportation (ODOT) in their standard OHD L-49 “Method of Test for Determining Soluble Sulfate Content of Soil” (ODOT 2005).

3.7.4 Soil pH and Direct Current Electrical Conductivity

Soil pH was determined for the untreated soils using the method given in ASTM D 4972-01 “Standard Test Method for pH of Soils” (ASTM 2009). Conductivity values for untreated soils were measured at the same time the pH testing was conducted. A calibrated digital conductivity meter was used to measure the values of electrical conductivity and the readings were taken after the pH testing of the mixture. Three readings were taken for each mixture and the conductivity value was taken as the average of these readings.

4 Results and Discussion

4.1 Introduction

Five roadway construction sites were chemically stabilized to improve subgrade soil strength and stiffness. Field tests were conducted at these sites after various curing times and included DCP, PANDA, and PFWD testing. Samples of untreated and treated soil and additives were collected from these sites for laboratory testing. Numerous tests were conducted to characterize the soils and additives, to estimate the additive contents of field samples, and to determine the strength (UCS) and stiffness (M_R) of samples prepared in the laboratory and tested at various curing times. Results of field and laboratory measurements of strength and stiffness were analyzed and compared with the goal of evaluating correlations for assessing strength gains of stabilized subgrades with time and for evaluating field strength and stiffness using in situ testing devices.

4.2 Soil Descriptions

The field test sites and results of tests to characterize the untreated soil are listed in Table 3. Soils at the sites classified as A-4 (ML), two as A-6 (CL), and one as A-7-6 (CH) according to the AASHTO (and USCS) systems.

Table 3: Classification and Physical Properties of Untreated Soils

Site #	Site Name	AASHTO Group Class.	UCS Group Class.	Percent Fines (%)	Percent Clay size (%)	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)	Linear Shrinkage (%)	Shrinkage Limit (%)
#1	US 281	A-4(0)	ML	50.1	10.3	NP	NP*	NP	2.2	2.7
#2	Penn Ave.	A-6(15)	CL	74.1	29.6	40	18	22	14.3	7.5
#3	US 177	A-7-6(35)	CH	93.5	35.5	54	20	34	16.8	12.0
#4	SH 7	A-6(11)	CL	66.0	20.8	35	14	21	11.7	11.0
#5	US 81	A-4(0)	ML	57.5	13.8	NP	NP	NP	1.4	8.5

*NP – Non Plastic

4.2.1 Chemical Additives

The source and type of a chemical additive used at each field test site is shown in Table 4. Samples of specific chemical additives were collected in the field and utilized in corresponding laboratory tests.

Table 4: Chemical Stabilizers and Sources

Site Number	Soil Name	Type of Stabilizer	Source of Chemical Additive
#1	US 281	CFA	Red Rock Plant
#2	Penn Ave.	CFA	Red Rock Plant
#3	US 177	Granulated Quicklime	Marble City, TX
#4	SH 7	Granulated Quicklime & CFA	LaFarge, Muskogee Plant US Lime, TX
#5	US 81	CFA	LaFarge, Muskogee Plant

4.2.2 Physical, Chemical and Mineralogical Properties of Soils

The soils investigated in this study were subjected to a series of tests to determine physical, chemical and mineralogical properties. For untreated soils, the tests performed were: grain size distribution (ASTM D 422-00), Atterberg

Limits (ASTM D 4318-00), linear shrinkage (BS 1377:1975, Test 5), Total Specific Surface Area (SSA) (Cerato and Lutenegger 2002), sulfate content (ODOT 2005), carbonate content (Dreimanis 1962), pH (ASTM D 4972-01), and direct current electrical conductivity. The results of these tests for untreated soils are presented in and Table 5. The results of Atterberg Limits and linear shrinkage on 14 day cured samples are shown in Table 6. Total SSA testing was conducted on the stabilized soils as a function of curing time; however, the results were highly erratic and are not reported here. Possibly, the presence of the chemical additive produces adverse reactions with the chemicals used during testing.

Results of the bar linear shrinkage tests for untreated soil and stabilized soil cured at room temperature for 14 days are compared in Figure 8. The linear shrinkage (LS) for the non-plastic soils from Sites 1 and 5 showed little change, while clayey soils associated with Sites 2, 3, and 4 showed substantial reductions in the linear shrinkage with the addition of chemical additive. This is consistent with expectations of a chemical stabilizer mitigating adverse volume change behavior in high PI soils. However, the shrinkage limit (SL) values were not consistent in their behavior relative to soil type. This may be due in part to the fact that shrinkage curves for the stabilized soils were not linear and it was somewhat difficult to interpret the point of zero volume change shown in Figure 7. On the other hand, the linear shrinkage is a straightforward measurement and leaves little room for interpretation. Thus, it is appears to be a reliable indicator of the effectiveness of a chemical stabilizer for a given soil.

For the stabilized soil samples that exhibited plasticity in the untreated state, the plasticity was reduced significantly once stabilized. The plasticity index of the soil from Site #2 went from 22 in the untreated state to 14 after treatment and 14 days of room temperature curing; similarly, the PI of soil from Site #3 went from 34 to 18 and from Site #2 the PI went from 21 to non-plastic (NP). Results from Sites #2 and #3 indicate the decrease in PI was the result of a decrease in liquid limit and increase in plastic limit. Corresponding liquid and plastic limit changes for each of these sites were: Site #2 LL went from 40 to 34 and PL went from 18 to 20; and Site # 3 LL went from 54 to 44 and PL went from 20 to 26. Results in Table 6 indicate that curing temperature had little effect on the Atterberg limits or bar linear shrinkage test results.

Table 5: Mineralogical Properties of Natural (Untreated) Soils

Site #	Soil Name	Total SSA (g/m^2)	Sulfate Content (ppm)	Cation Exchange Capacity (meq/100g)	Calcite Content (%)	Dolomite Content (%)	Carbonate Content (%)	pH	Conductivity (mS/cm)
#1	US 281	44.5	240	11.5	1.9	0.6	2.5	8.35	145.2
#2	Penn Ave.	116	262	18.2	3.0	3.9	6.9	7.88	476.0
#3	US 177	161	2086	26.9	3.2	1.0	4.2	7.29	1199.0
#4	SH 7	88.5	259	22.6	1.2	3.9	5.1	8.50	282.7
#5	US 81	30	262	17.2	3.6	1.9	5.5	9.00	159.7

Table 6: Stabilized Soils Physical Properties (14-day Curing)

Site Number	Soil Name	Curing Condition	LL (%)	PL (%)	PI (%)	LS (%)	SL (%)
#1	US 281	Room Temperature	NP*	NP	NP	1.68	2.33
		40°F	NP	NP	NP	2.09	2.94
#2	Penn Ave.	Room Temperature	34	20	14	9.12	5.45
		40°F	35	20	15	8.93	6.57
#3	US 177	Room Temperature	44	26	18	10.64	7.52
		40°F	43	28	15	10.34	7.66
#4	SH 7	Room Temperature	NP	NP	NP	1.91	5.54
		40°F	NP	NP	NP	1.82	6.37
#5	US 81	Room Temperature	NP	NP	NP	0.82	2.26
		40°F	NP	NP	NP	1.67	1.80

*NP - Non Plastic

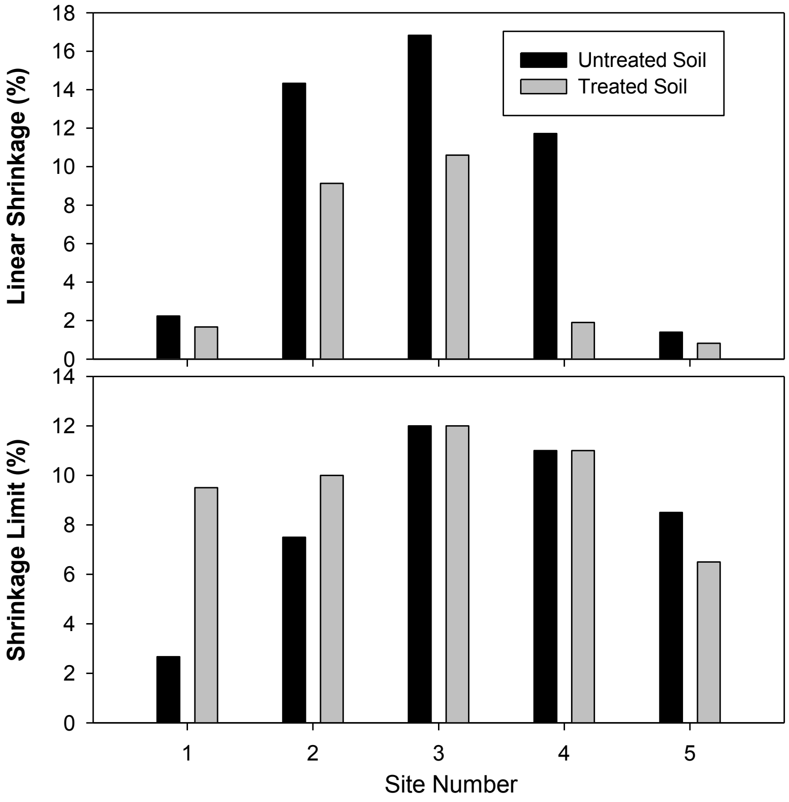


Figure 8: Comparison of Results from Bar Lineage Shrinkage Tests on Untreated and Treated Soil Cured 14 days at Room Temperature

4.3 XRF Analysis to Determine Chemical Additive Content

The XRF testing was conducted by ALS Laboratory Group on untreated soil, raw additive, and chemically treated soils obtained from the field to determine the chemical stabilizer content as described in Section 3.4. The calculated amount of additive determined from XRF on treated soil samples and the target amount sought during construction are compared in Table 7 for each field test site.

To assess the accuracy of this method, several control samples were sent along with the actual field test samples. These control samples were prepared in the laboratory using a commercial Kaolinite called “Old Hickory Clay” (OHC) and three chemical stabilizers including hydrated lime, CKD, and CFA. All of the samples were labeled only with a letter/number, and no information about the soils or additives was listed in order to ensure unbiased testing. Table 8 shows the results of the control sample testing compared to the actual additive contents. As shown in Figure 9 and Table 8, for lime the additive content is overestimated by about 0.5% to 1% (by weight) for additive percentages between 1 and 5%. For CKD and CFA the additive content is generally underestimated using XRF by about 0.2% at low additive percentages and about 3.0% at the higher additive contents. As shown, generally for a given additive the XRF results appear better at lower additive percentages.

While there are differences between actual and predicted additive contents shown in Figure 9, the variations appear consistent and predictable, which suggests the method holds promise as a field verification tool. Additional work is needed in this regard. Comparing the XRF-determined additive contents with field target values in Table 7, it is seen that for Sites #1 and #2, the XRF-determined values exceed the design values; however, for Sites #3 and #5 the opposite is seen. The amount of stabilizer for Site #4 could not be obtained using XRF because lime and CFA were added at different times and testing was conducted only on the sample containing both additives. If samples were also tested after the lime was added and mixed, prior to the CFA, then in theory the XRF method could have been used.

While it is recognized that the XRF method is not perfect, it was decided that the treated soil prepared in the laboratory for various tests would use the XRF-determined percentages from Table 7. This decision was made because the measured values appeared reasonable relative to the field target values, the control testing was limited (only one soil type, kaolinite), and because correcting the XRF-determined values based on the trends in Figure 9 would move them farther away from the design values for three of the soils.

Table 7: XRF Determined Additve Content from Field Mixed Samples

Site Number	Soil Name	Type of Stabilizer	Design Specified Content (%)	XRF Determined Content (%)
#1	US 281	CFA	14	15.4
#2	Penn Ave.	CFA	12	13.4
#3	US 177	Quicklime	2.7	2.3
#4	SH 7	Quicklime & CFA	4 , 12	N/A
#5	US 81	CFA	14	12.1

Table 8: XRF Control Samples Results

Sample	Additive Type	Additive Content (%)	XRF Determined Additive Content (%)
OHC 1	Lime	1	1.49
OHC 2	Lime	2	1.69
OHC 3	Lime	2	1.69
OHC 4	Lime	3	3.36
OHC 5	Lime	4	4.92
OHC 6	Lime	4	4.50
OHC 7	Lime	5	6.09
OHC 8	CKD	5	4.80
OHC 9	CKD	5	4.41
OHC 10	CKD	10	10.15
OHC 11	CKD	10	8.51
OHC 12	CKD	15	12.21
OHC 13	CKD	15	12.56
OHC 14	CFA	5	4.75
OHC 15	CFA	5	4.49
OHC 16	CFA	10	8.62
OHC 17	CFA	10	9.19
OHC 18	CFA	15	13.07
OHC 19	CFA	15	12.01

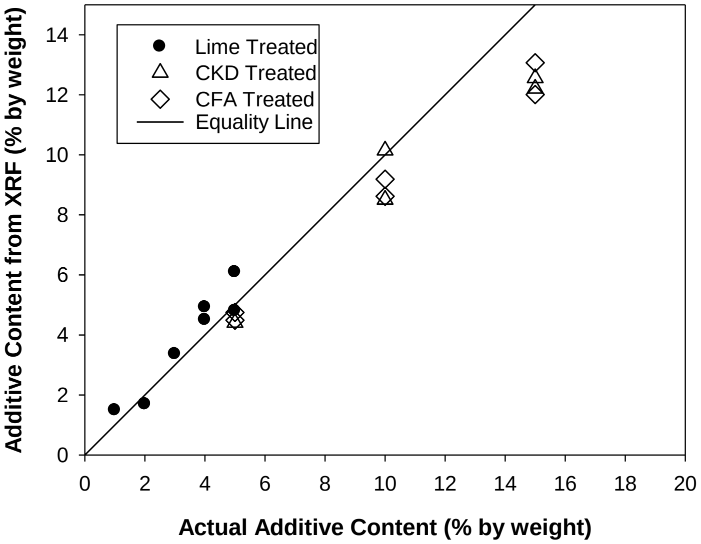


Figure 9: Comparison of Additive Content Determined by XRF and Actual Additive Content in Prepared Sample

4.4 Harvard Miniature Compaction

The optimum moisture content (OMC) and maximum dry density (MDD) for untreated and stabilized soils are summarized in Table 9. The OMC and MDD were used to prepare UCS test specimens. The compaction curves for each field test site are in Appendix A.

Table 9: Results of Harvard Miniature Compaction

Site Number	Soil Name	Untreated OMC (%)	Untreated Maximum Dry Unit Weight, γ_d (pcf)	Additive	Additive Content (%)	Stabilized OMC (%)	Stabilized Maximum Dry Unit Weight, γ_d (pcf)
#1	US 281	12.00	116.5	CFA	15.38	10.70	119.9
#2	Penn Ave.	17.50	108.8	CFA	13.39	15.70	109.8
#3	US 177	21.46	98.5	Lime	2.34	22.40	98.5
#4	SH 7	13.50	112.7	Lime, CFA	4, 12	12.75	112.5
#5	US 81	10.80	119.0	CFA	12.15	9.00	123.7

4.5 Unconfined Compression Test Results

Unconfined compression tests were conducted on Harvard Miniature specimens to determine an average unconfined compression strength (UCS) for each soil at curing times corresponding to field tests, as well as additional standard curing times of 1, 3, 7, 14, and 28 days. Average UCS values were determined using three samples for room temperature curing conditions, as well as three samples for 40°F curing. The UCS values for all tests are tabulated in Appendix B. The results of UCS testing for both curing temperatures are shown in Figures 10 through 14. Averages and standard deviations for dry density and water content data obtained for each site during preparation of stabilized soil specimens are shown in Table 10.

4.5.1 Site #1, US 281

In Figure 10 the UCS strength of the stabilized soil is much higher than that of the natural soil. After one day of curing, the strength increases, reaching the required strength of 50 psi above that of the untreated soil. This rapid increase in strength is a benefit of the intrinsic cementitious properties of CFA. Following the initial rapid strength gain there is a gradual and continuous increase in strength due to pozzolanic reactions. As shown in Figure 10, the curing temperature appears to have a negligible effect on strength gain for this low plasticity soil.

4.5.2 Site #2, Penn Ave.

UCS results for Site #2 are shown in Figure 11. The results also show a rapid increase in strength due to CFA stabilization and the 50 psi increase in UCS was achieved after one day of curing. A gradual and substantial increase in strength occurred with increasing curing time. The influence of temperature is more pronounced as curing time increases with lower temperature curing resulting in lower UCS. This appears to be associated with the higher plasticity of this soil as compared to Site #1.

Table 10: UCS Samples Preparation Statistics

Site Number	Soil Name	No. of Samples	Average Dry Unit Weight (pcf)	Standard Deviation of Dry Unit Weight (pcf)	Average Water Content (%)	Standard Deviation of Water Content (%)
#1	US 281	48	121.2	1.3	10.46	0.15
#2	Penn Ave.	42	110.2	2.0	15.56	0.29
#3	US 177	54	100.0	1.0	22.07	0.39
#4	SH 7	30	111.2	1.8	12.89	0.30
#5	US 81	54	122.6	1.3	9.15	0.17

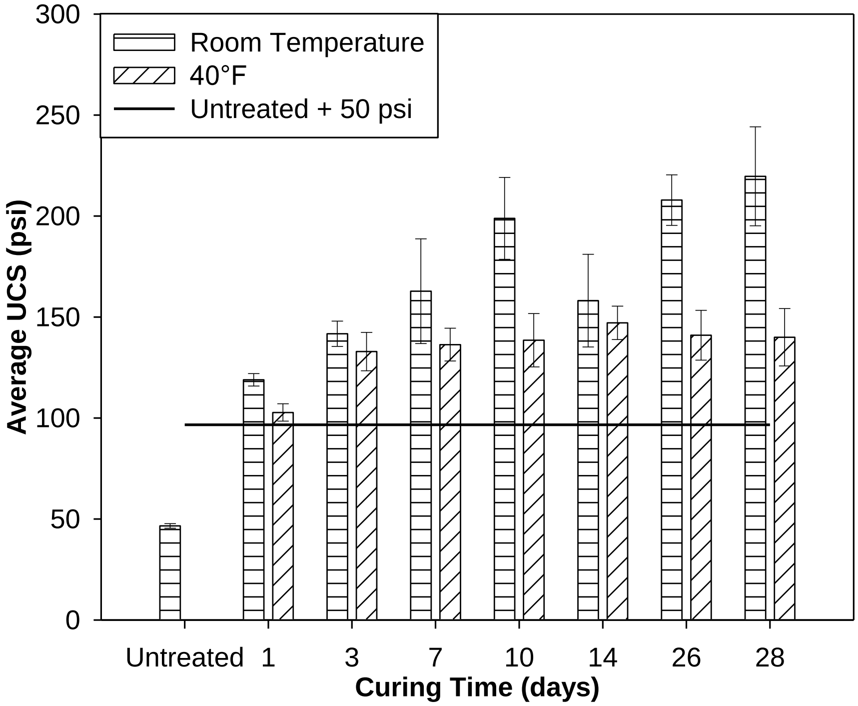


Figure 10: Site #1 US 281 UCS Results

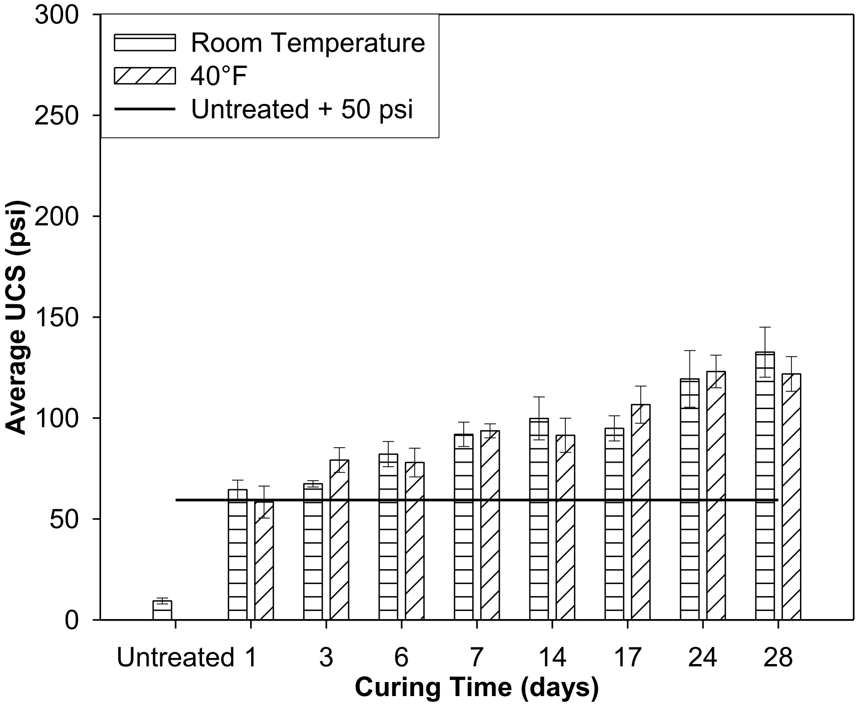


Figure 11: Site #2, Penn Ave. UCS Results

4.5.3 Site #3, US 177

The results of UCS tests for lime treated soil from Site #3 are shown in Figure 12. The initial average strength increase for the one day cured samples did not meet the 50 psi increase; however, with additional curing time, this strength gain was achieved. This is an indication of the substantial pozzolanic reactions that can occur with lime treatment. As with Site #2, results in Figure 12 suggest that lower temperature curing slows the strength gain in soils with significant clay content.

As shown in Figure 12 the average UCS values at 7 and 15 days do not match the general trend exhibited at other curing times. Possibly the larger granules of crushed and process quicklime contributed to greater sample variability compared to powdered additives.

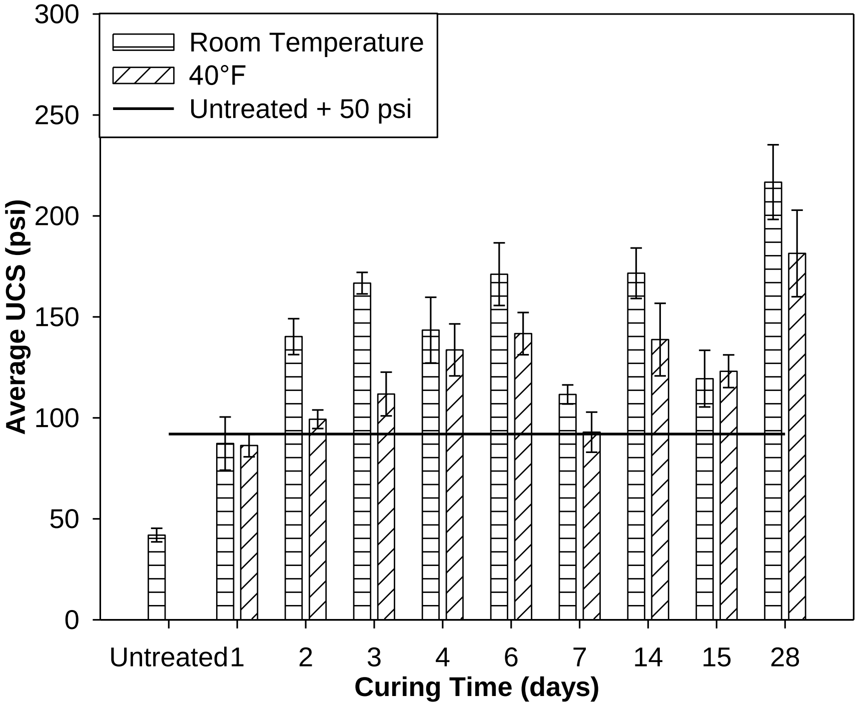


Figure 12: Site #3, US 177 UCS Results

4.5.4 Site #4, SH 7

Figure 13 shows the trend for strength increase over time. This site was modified with quicklime to reduce the PI, and then stabilized with CFA to increase strength. The behavior seen in the figure reflects both of these treatments. As with previous sites, a substantial gain in strength is achieved after one day of curing followed by a significant gradual increase in strength at later curing times. There was a slight decrease in strength from 14 to 28 days that may be associated with sample variability. This variability is reflected in the range bars associated with the average values of UCS.

While the soil at Site #4 was cohesive, the influence of low temperature curing was not as pronounced as with Sites #2 and #3. This may be a result of Site #4 soils having less fines and less clay size fraction (Table 3) as compared to Sites #2 and #3. Possibly the combination of lime and fly ash reduces the influence of temperature as well.

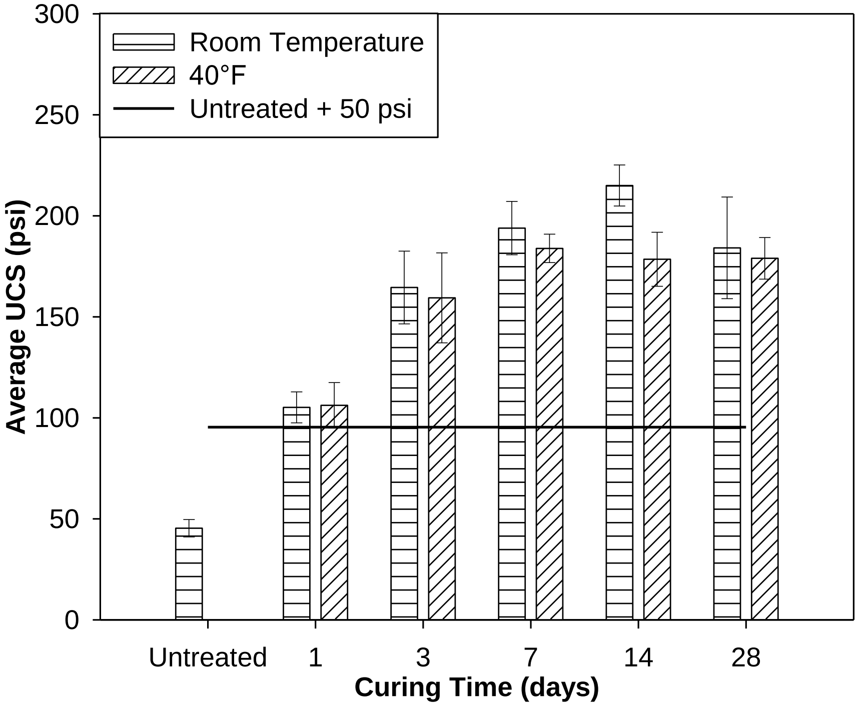


Figure 13: Site #4, SH 7 UCS Results

4.5.5 Site #5, US 81

Results of UCS testing for Site #5 are shown in Figure 14. The soil associated with this site was a low plasticity silty soil, similar to Site 1. Hence, the behavior exhibited with addition of CFA is similar to that observed in Figure 10 for Site #1. There is an initial and significant increase in UCS after one day of curing, followed by a slight increase with curing time, and the influence of curing temperature appears to be insignificant.

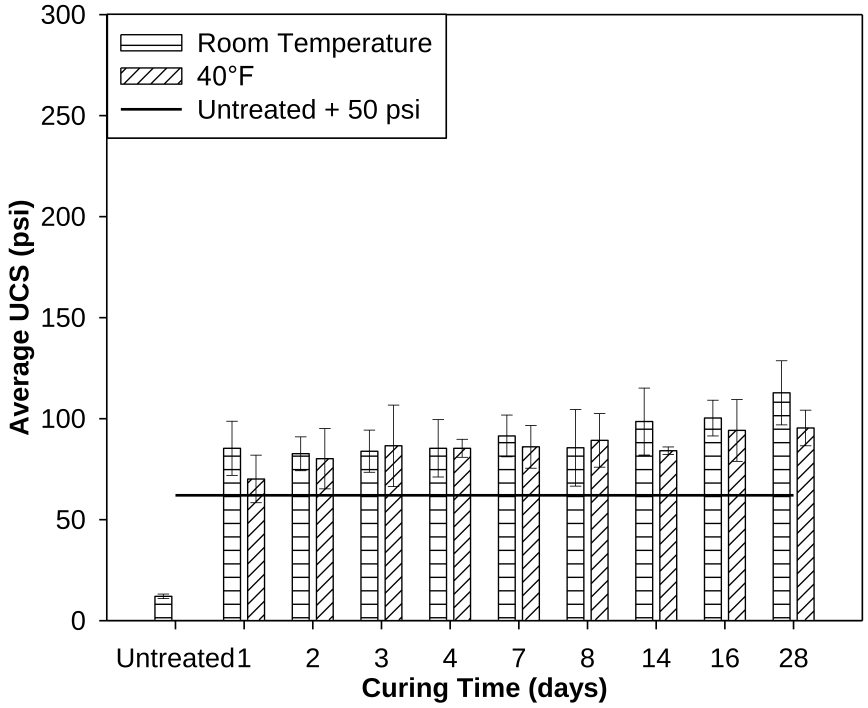


Figure 14: Site #5, US 81 UCS Results

4.5.6 Modeling Strength Gain with Curing Time

A power model was found to reasonably represent the increase in resilient modulus with curing time, as discussed in Section 4.6. In Figure 15, the power model provides a reasonable fit to the strength-curing time relationship for the five sites involved in the current study. Figure 16 shows that the model also provides a reasonable fit to the data obtained in the previous study for five other sites in Oklahoma that involved stabilization with CFA and cement kiln dust (CKD). The data from the previous study represent samples mixed and compacted in the laboratory using a Harvard miniature compaction device (spring loaded tamper). The power model parameters are summarized in Table 11 along with the coefficients of determination (r^2).

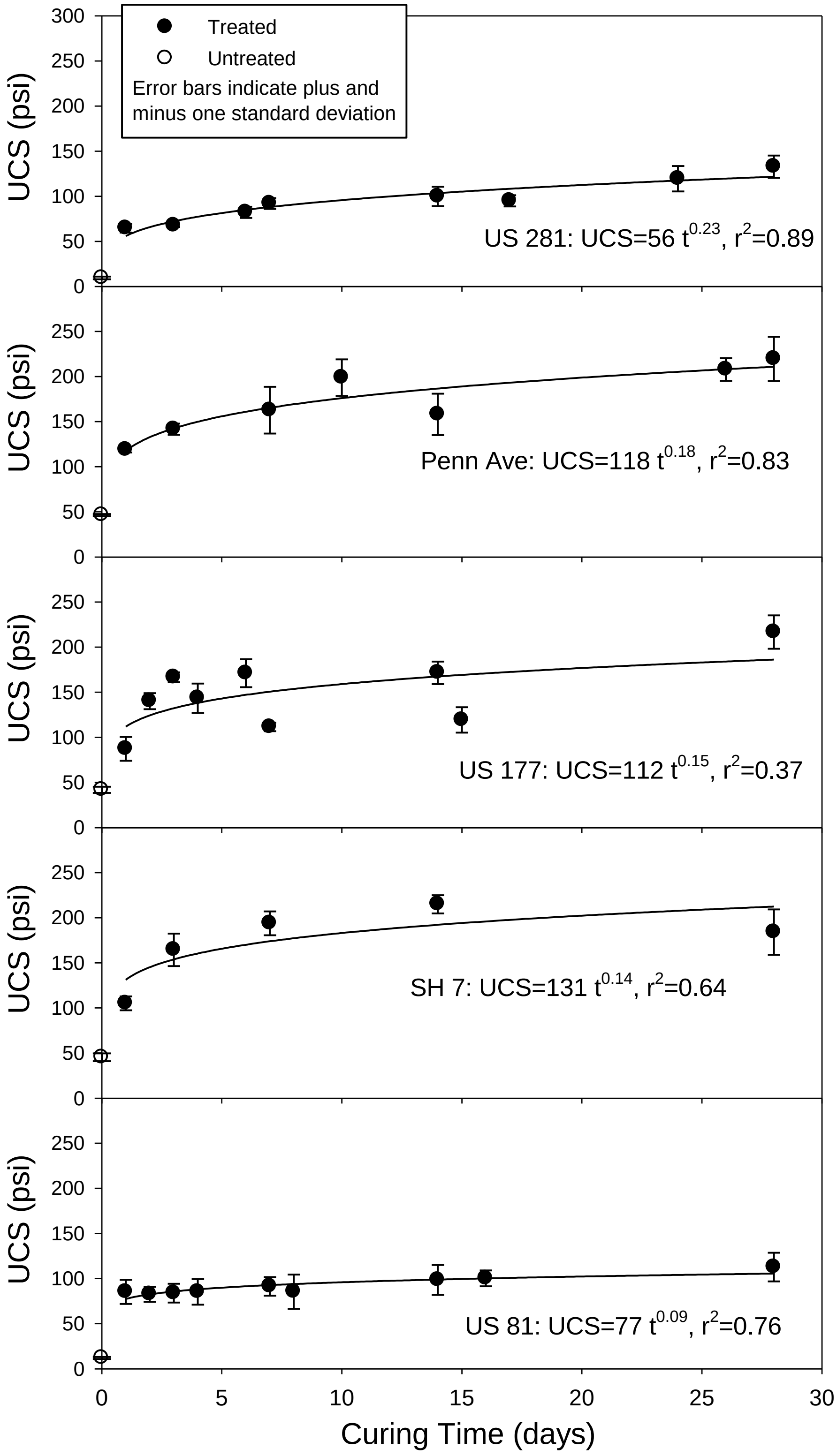


Figure 15: Power Model Curves Relating UCS to Curing Time

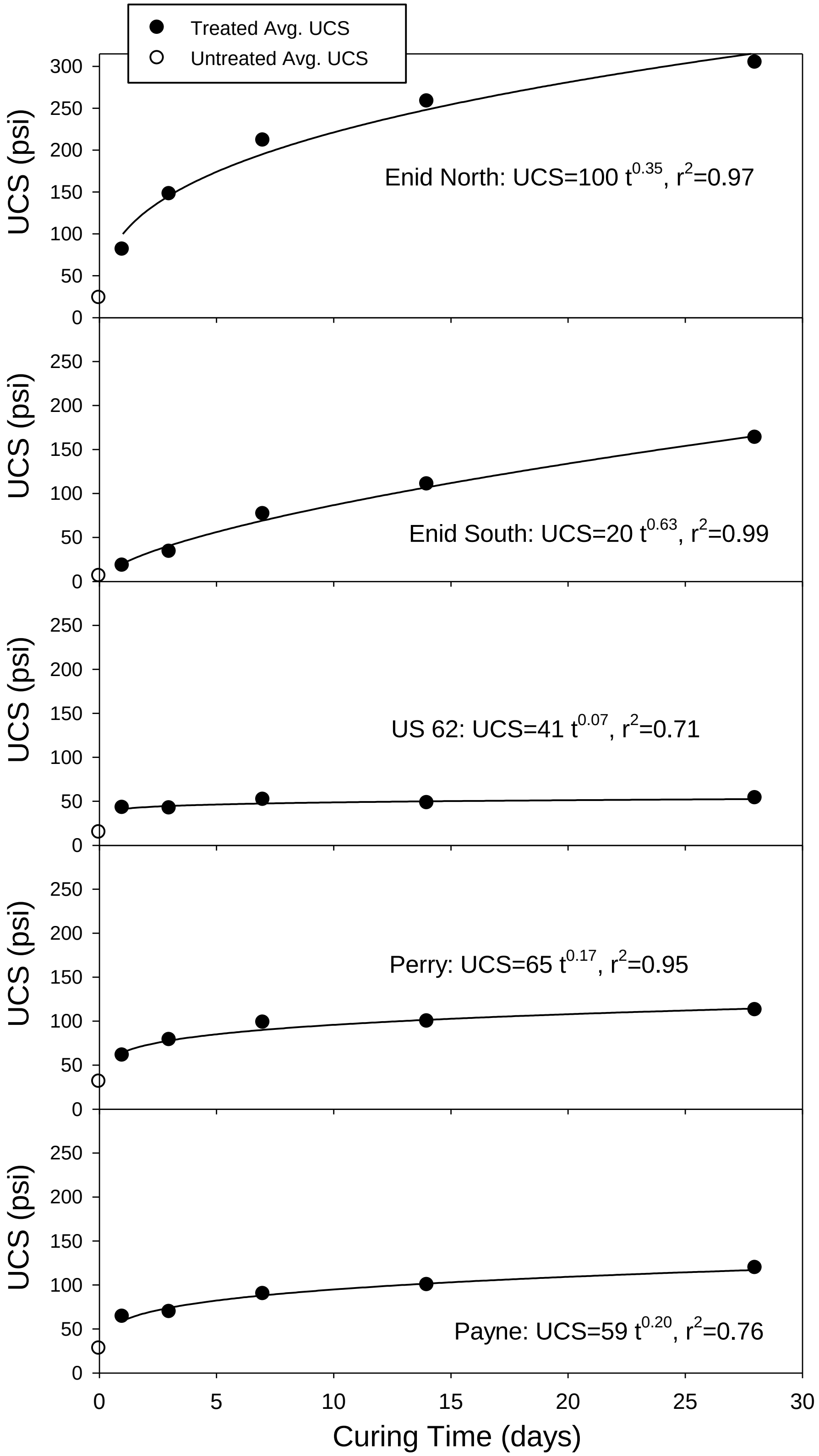


Figure 16: Power Model Curves Relating UCS to Curing Time from Previous Study (Snethen et al. 2008)

Table 11: Summary of Exponential Regression Model Parameters for Describing UCS Improvement with Curing Time

Site	Soil Type	%fines, F	SF*	Additive Type	Additive Content, AC (%)	UCS_1 (psi)	R_{tu}	r^2
US 281	ML	50.1	1.9	CFA	15.4	56	0.233	0.89
Penn Ave	CL	74.1	3.1	CFA	13.4	118	0.175	0.83
US 177	CH	93.5	2.6	LIME	2.3	112	0.153	0.37
SH 7	CL	66	2.9	CFA/LIME	12/4**	131	0.144	0.64
US 81	ML	57.5	2.0	CFA	12.2	77	0.093	0.76
Enid North	SC	43	2.2	CKD	14	100	0.346	0.97
Enid South	SM	22	1.0	CKD	12	20	0.627	0.99
Anadarko	ML	56	1.7	CFA	15	41	0.072	0.71
Perry	CL	77	2.9	CFA	15	65	0.171	0.95
Payne	CL	53	2.4	CFA	16	59	0.204	0.76

*Stabilization Factor, SF = a*F+b*LS+c*AC+d*UCS_{1d}

LS = Untreated Linear Shrinkage, UCS_{1d} = UCS measured at 1 day of curing

**AC assumed equal to 16%.

The power model is ideal for describing strength increase with time because it is simple and convenient, and it nicely captures large increases in strength at early curing times and the gradual increases at later curing times. As shown in Equation 3, power model parameters include UCS_1 and R_{tu}, which represent the UCS at one day of curing and the exponential rate of improvement with time, respectively.

$$UCS = UCS_1 t^{R_{tu}} \qquad \textbf{(Equation 3)}$$

where: UCS = the unconfined compression strength at curing time, t,

UCS_1 = functional value of UCS at curing time of one day in units of psi,

t = curing time in days

R_{tu} = the power function exponent.

The rate parameter, R_{tu}, is an indicator of how quickly strength increases with time, larger values indicating more significant strength gains. The model is convenient in that if R_{tu} can be reasonably estimated then one only needs to obtain a 1-day UCS (i.e. UCS_1) to predict strengths at other times. This can be useful when estimates of strength improvement are needed very quickly, for example if soil type changes are unexpectedly encountered during construction of stabilized subgrades. In actuality, UCS at any curing time can be used to define the model if R_{tu} is known (i.e. estimated).

To use the model as a means of estimating strength increases with curing time requires a reasonable estimate of the rate parameter R_{tu}. Towards this end, relationships between untreated and treated soil properties and the model parameter, R_{tu}, were examined to see if useful correlations could be established. Figure 17 shows R_{tu} for all 10 sites (current study and previous study) plotted against percent fines for untreated soil. This figure reveals that there is a trend in the relationship observed although there is also some scatter. The trend for R_{tu} versus fines is reasonably good, but this trend is somewhat an artifact of the fact that the CKD was used with soils containing less fines while the CFA and Lime were used with soils containing larger percentages of fines. It is possible that CKD used with soils containing larger amounts of fines or CFA used with soils containing fewer fines may not fall on this same trend line. Thus, some caution is necessary in using this correlation.

It was expected that in addition to the amount of fines, the rate of improvement as reflected by R_{tu} would depend on the nature of the fines (e.g. plasticity), additive type, and additive content. To capture the influence of all of these factors, four parameters that characterize the nature of the soil and additive were lumped together in a stabilization factor (SF) and compared with the model parameters. The four parameters include the percent of fines (F), the linear shrinkage of the untreated soil (LS), the additive content (AC) and measured 1-day unconfined compression strength of the treated soil (UCS_{1d}). These parameters were chosen because they can be obtained easily and relatively quickly. They were used to define the stabilization factor as follows in Equation 4,

$$SF = a*F + b*LS + c*AC + d*UCS_{1d} \qquad \textbf{(Equation 4)}$$

where: a, b, c, and d are equal to one divided by a number approximately equal to the maximum value (of the ten soils) of F, LS, AC, and UCS_{1d}, respectively. Essentially, these leading coefficients normalize the primary variables to numbers between zero and one, so that parameters with large magnitudes do not have a dominant influence on SF. The values assigned to each parameter were a=1/100=0.01, b=1/20=0.05, c=1/20=0.05, and d=1/120=0.008. These coefficients nominally represent the inverse of the largest value of a given parameter in the ten-soil data set.

In developing Equation 4, other soil parameters were tried as well as other mathematical forms of SF; however, the form given in Equation 4 worked best and involved parameters that are easily determined yet reflect the nature and quantity of soil and additive involved. This was important since a method to estimate a reasonable value of R_{tu} that could be used with the one-day UCS to estimate strengths at other curing times was the desired outcome.

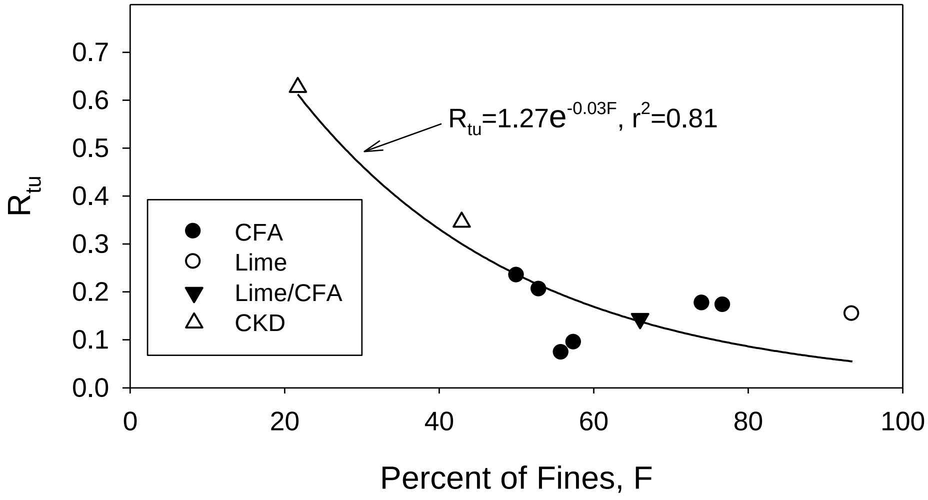


Figure 17: Power Model Parameter R_{tu} for UCS-Time Relationship versus Percent of Fines

Model parameter, R_{tu} is plotted against SF in Figure 18. The coefficient of determination for the exponential decay equation relating SF to R_{tu} in Figure 18a was not as good compared to the percent fines relationships; however, the SF parameter was a more logical choice to capture the influence of important factors related to stabilization effectiveness as opposed to just the fines content. Thus, the SF correlation represents a more general model. If the two data points (US 81, Anadarko sites in Table 11) identified as outliers are removed, then the r^2 value improves significantly as shown in Figure 18b.

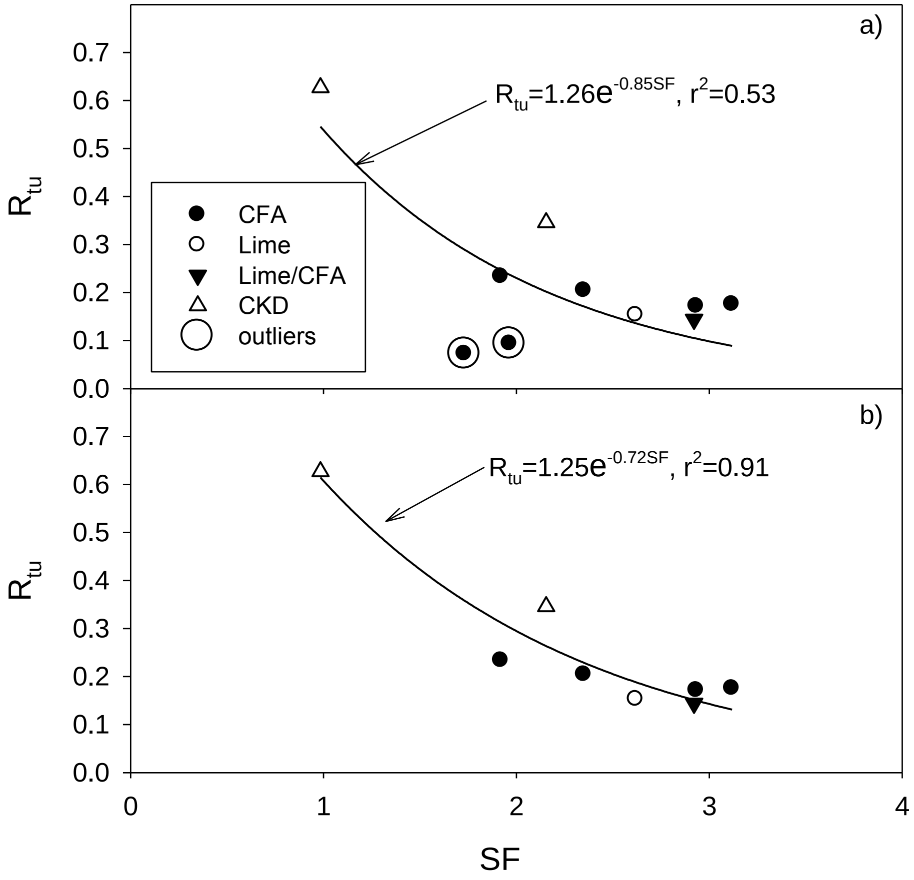


Figure 18: Power Model Parameters for UCS-Time Relationship versus Stabilization Factor

4.6 Resilient Modulus Test Results

Results of resilient modulus testing results are summarized in Figures 19 to 23. Data shown in these figures are tabulated in Appendix C. The individual test and average M_R values are plotted versus curing time for treated samples while untreated values are plotted at zero curing time. The M_R values shown in the figures were obtained at nominal confining stress of 4 psi and nominal deviator stress of 10 psi. Corresponding values of moisture content and dry density are plotted above M_R values for individual tests. While there is some scatter in the data, generally the results at all sites reflect significant early gains in stiffness after 1 to 3 days of curing followed by rather gradual changes from 3 to 28 days.

Following the approach described by Pinilla et al. 2011, based on the study reported in Snethen et al. 2008, a power model ($M_R=M_{R1}t^{Rt}$) was adopted to model the relationship between curing time (t) and resilient modulus for stabilized soil mixed in the laboratory. The resulting curves are shown in Figures 19 to 23 along with the model parameters (M_{R1}, R_t) and coefficients of determination (r^2). The coefficients of determination varied between 0.38 to 0.86. The lowest values were obtained for the Penn Ave. and SH 7 sites. For the Penn Ave. site the low r^2 value is the result of a slight decrease in M_R from 1 to 3 days and a more significant decrease from 15 to 28 days. Resilient modulus measurements are very sensitive to a number of factors due to the very small displacement measurements associated with the testing, especially for stabilized soils. Thus, any slight variations in sample heterogeneity, moisture content, density and fabric may contribute to variations in the results. Possibly, the slightly lower moisture contents associated with the 28 day tests are partly responsible for the variations noted. For the SH 7 site, the lower value of r^2 (=0.54) was primarily attributed to the variation in M_R seen in the 7 day results. Even with these lower r^2 values at the two sites in question, the resulting power models seem to provide a reasonable description of M_R with curing time.

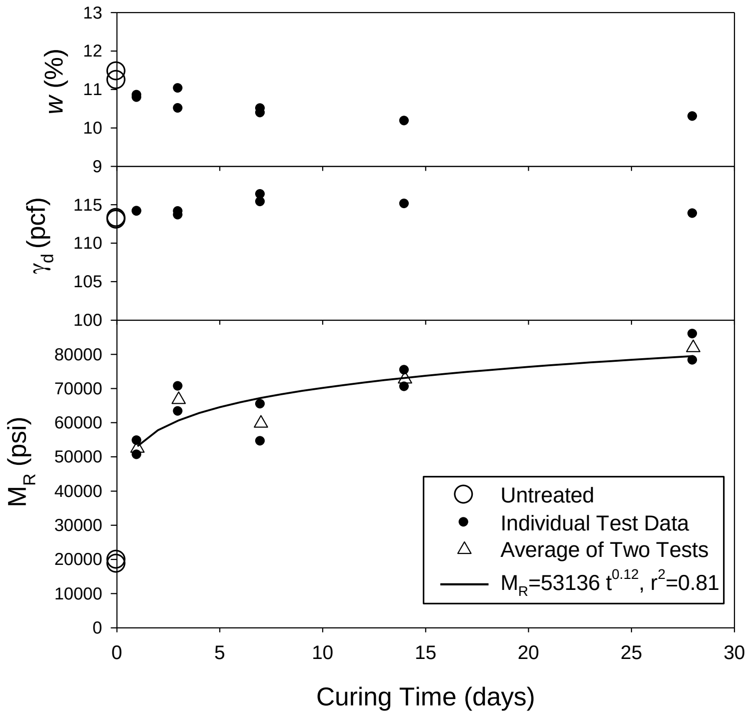


Figure 19: Site #1, US 281 Resilient Modulus Test Results

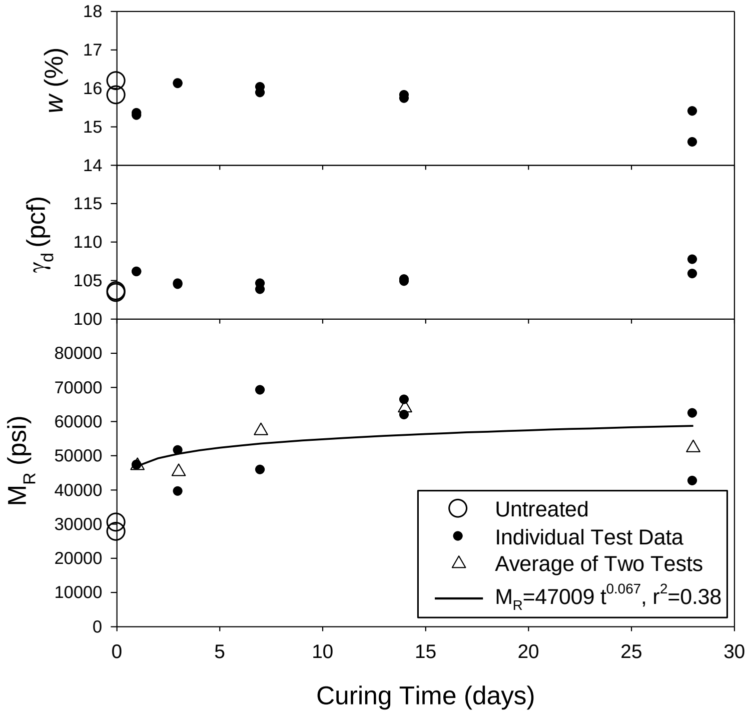


Figure 20: Site #2, Penn Ave. Resilient Modulus Test Results

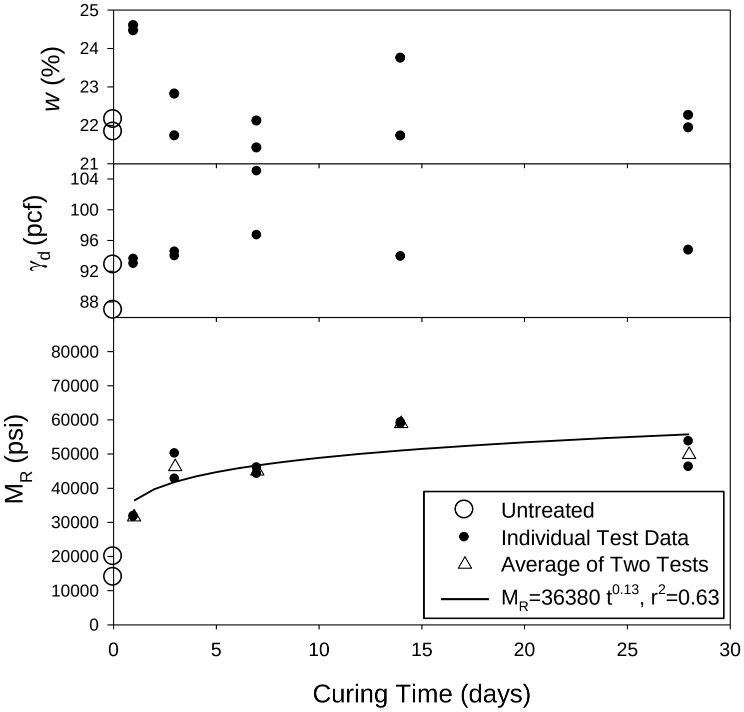


Figure 21: Site #3, US 177 Resilient Modulus Test Results

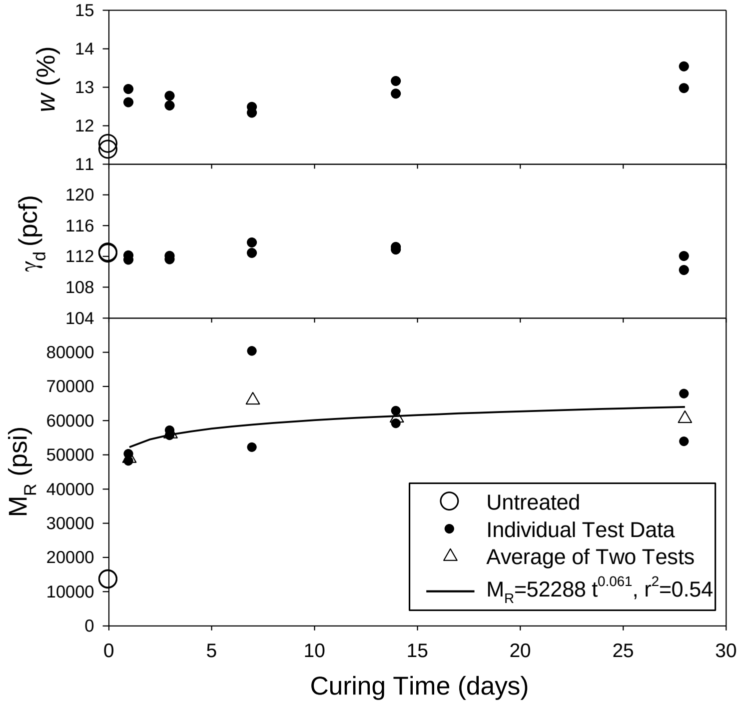
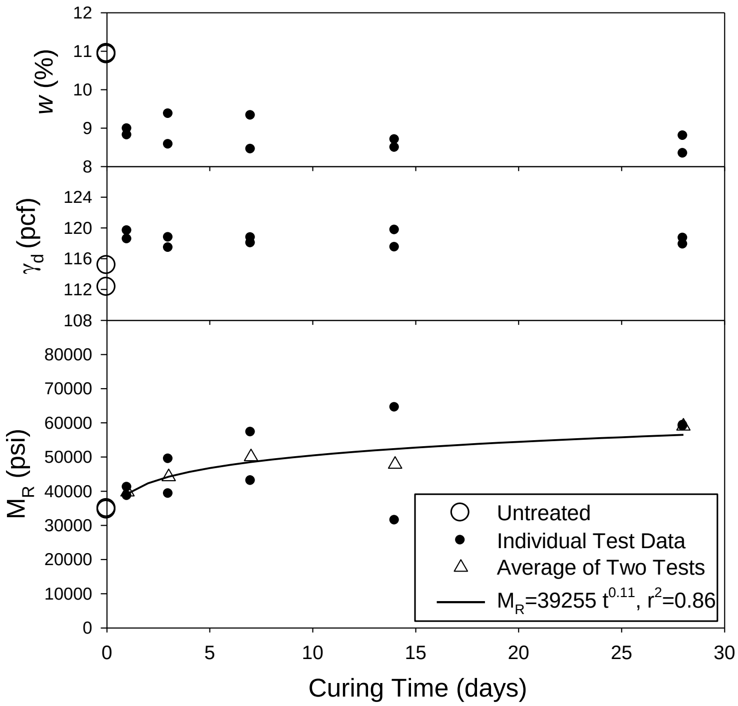


Figure 22: Site #4, SH 7 Resilient Modulus Test Results



Test Figure 23: Site #5, US 81 Resilient Modulus Test Results

The rate of change (%/day) in treated M_R based on values calculated using the power model is summarized in Table 12 for all sites. In Table 12, the percent change from 1 day to 3 and 3 to 28 days of curing is shown. The most significant early gains (percent change) in M_R form 1 to 3 days were achieved for the non-plastic silty soil from US 281 and US 81 mixed with CFA and for the CH soil mixed with Lime from US 177. For the silty soils with CFA the cementitious reactions are unimpeded by clay particles compared to the CL type soils at Penn Ave. and SH 7, and this may partly explain the significant early improvement in M_R. The early improvement (at 3 days) in the CH soil mixed with lime is attributed to the significant early reactions that occur with lime. The percent change in improved M_R from 3 to 28 days was again most significant for the two silty soil sites with CFA (US 281, US 81), followed by SH 7 (lean clay with lime and CFA), Penn. Ave. (lean clay with CFA), and then US 177 (fat clay with lime).

Table 12: Summary of Early and Late Rates of Improvement in Resilient Modulus

			Raw	**Treated, from Power Model**			**Rate of Improvement**		**Ultimate Improve-ment**
Site	**Soil Type**	**Additive/ Content (%)**	**M_R (psi)**	**M_{R-1} (psi)**	**M_{R-3} (psi)**	**M_{R-28} (psi)**	**Early (1-3) days (%/day)**	**Late (3-28) days (%/day)**	**Raw-28 days (%)**
US281	ML	CFA/ 15.4	19148	53136	60684	79497	20	1.2	415
Penn Ave	CL	CFA/ 13.4	28893*	47009	50600	58768	6	0.6	203
US177	CH	Lime/ 2.3	13395	52288	55894	64010	13	0.6	478
SH7	CL	Lime/4+ CFA/12	34664**	39255	44258	56484	7	1.1	163
US81	ML	CFA/ 12.2	16888	36380	41886	55786	16	1.3	330

M_R : Average value of resilient modulus for untreated soil
M_{R-i} : Value of M_R for i days of curing time computed using power model
*M_R determined on samples compacted 1.5% dry of OMC
**M_R determined on samples compacted 2 % dry of OMC

Generally, the ultimate improved M_R values were significantly larger than the untreated values as shown in Table 12. The untreated M_R values for the two lean clayey sites (Penn. Ave. and SH7) were quite high because compacted water contents were 1.5 to 2 percentage points below optimum (Table C3). In clayey soils, increases in M_R of 4 to 6 times larger than M_R values obtained wet of optimum have been observed (e.g. Maher et al. 2000, Gupta et al. 2007). Large increases in stiffness at water contents dry of optimum are attributed to the significant matric suction that develops at lower water contents. Comparing results of treated samples to untreated samples compacted at optimum or wet of optimum, the ultimate improvements due to stabilization would be considerably greater for the two CL soils (Penn. Ave., SH 7).

A summary of the power models for the five tests sites is presented in Table 13 along with results from the five other sites studied previously (Snethen et al. 2008) and reported by Pinilla et al. (2011). In the previous study it was found that the exponential growth rate expressed in the parameter R_t was strongly correlated to the percent of fines in the untreated soil for soils treated with CKD and CFA. The current study involves three sites where CFA was used, one site where lime was used and one site involving pretreatment with lime followed by CFA. Model parameter, R_t, from the previous and current studies are plotted together in Figure 24 against percent of fines. For the plot of R_t versus percent fines an exponential decay equation provides a good fit to the data with an r^2 value of 0.95. As discussed in relation to the R_{tu}-fines relationship for UCS results, examination of Figure 24 reveals that the early part of the best fit curve passes through two CKD data points while the latter part of the curve passes through the seven CFA, one lime and one lime/CFA data points. In other words, the early part of the correlation curve is controlled by CKD mixed with soils at lower percentage of fines compared to the latter part that is dominated by soils containing more fines mixed with CFA. Thus, similar to the correlation involving fines and the rate exponent for UCS discussed in the previous section, one should be careful in using this curve.

Table 13: Summary of Exponential Regression Model Parameters for Describing M_R Improvement with Curing Time

Site	Soil Type	%fines, F	SF*	Additive Type	Additive Content, AC (%)	M_{R1} (ksi)	R_t	r^2
US 281	ML	50.1	1.9	CFA	15.4	53	0.12	0.81
Penn Ave	CL	74.1	3.1	CFA	13.4	47	0.067	0.38
US 177	CH	93.5	2.6	LIME	2.3	36	0.13	0.63
SH 7	CL	66	2.9	CFA/LIME	12/4**	52	0.061	0.54
US 81	ML	57.5	2.0	CFA	12.2	39	0.11	0.86
Enid North	SC	43	2.2	CKD	14	110	0.36	0.86
Enid South	SM	22	1.0	CKD	12	29	0.98	0.98
Anadarko	ML	56	1.7	CFA	15	43	0.21	0.92
Perry	CL	77	2.9	CFA	15	121	0.11	0.42
Payne	CL	53	2.4	CFA	16	29	0.24	0.99

*Stabilization Factor, SF = a*F+b*LS+c*AC+d*UCS_{1d}

LS = Untreated Linear Shrinkage, UCS_{1d} = UCS measured at 1 day of curing

**AC assumed equal to 16%.

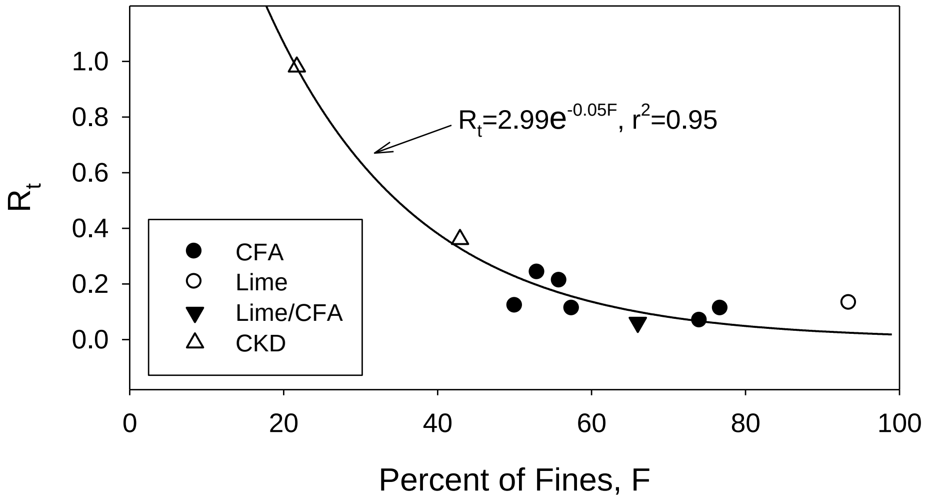


Figure 24: Power Model Parameter R_t for M_R-Time Relationship versus Percent of Fines

As discussed with regard to UCS, it is desirable to correlate the M_R rate parameter R_t with a parameter that captures the amount and nature of the fines and additive. Thus, in Figure 25, R_t is plotted against the stabilization factor, SF, defined previously. As shown, the correlation is still reasonably good, and again is more desirable in that it considers factors related to the amount and nature of the fines as well as the additive and not just solely fines content.

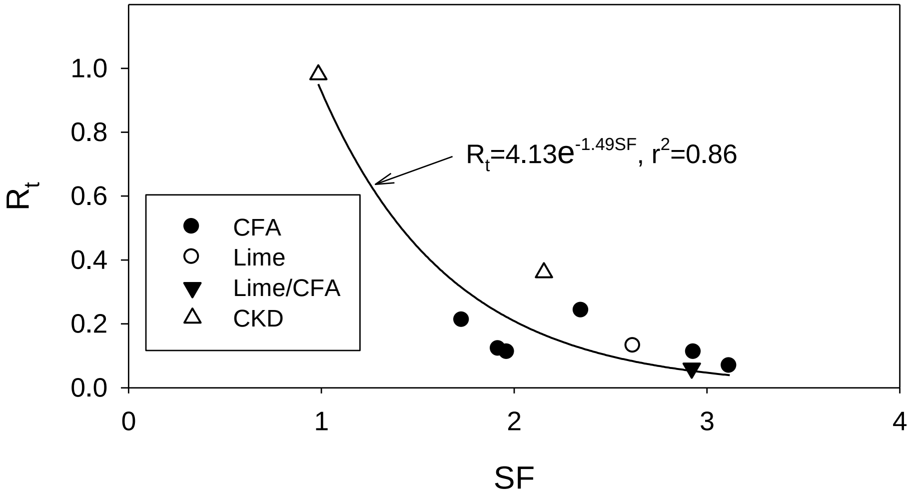


Figure 25: Power Model Parameter R_t for M_R-Time Relationship versus Stabilization Factor

4.7 Comparison of Unconfined Compression and Resilient Modulus Test Results

Comparing the results of Figures 10-14 and 19-23, it was generally noted that both the UCS and M_R values increased with increasing curing time. It stands to reason that a correlation between the UCS and M_R will exist for the same soils and additive types at similar curing times. In Figure 26 the untreated UCS and M_R values are compared for the current five sites along with the previously studied five sites (Snethen et al. 2008). Interestingly, there is very little correlation between the untreated values of strength and stiffness.

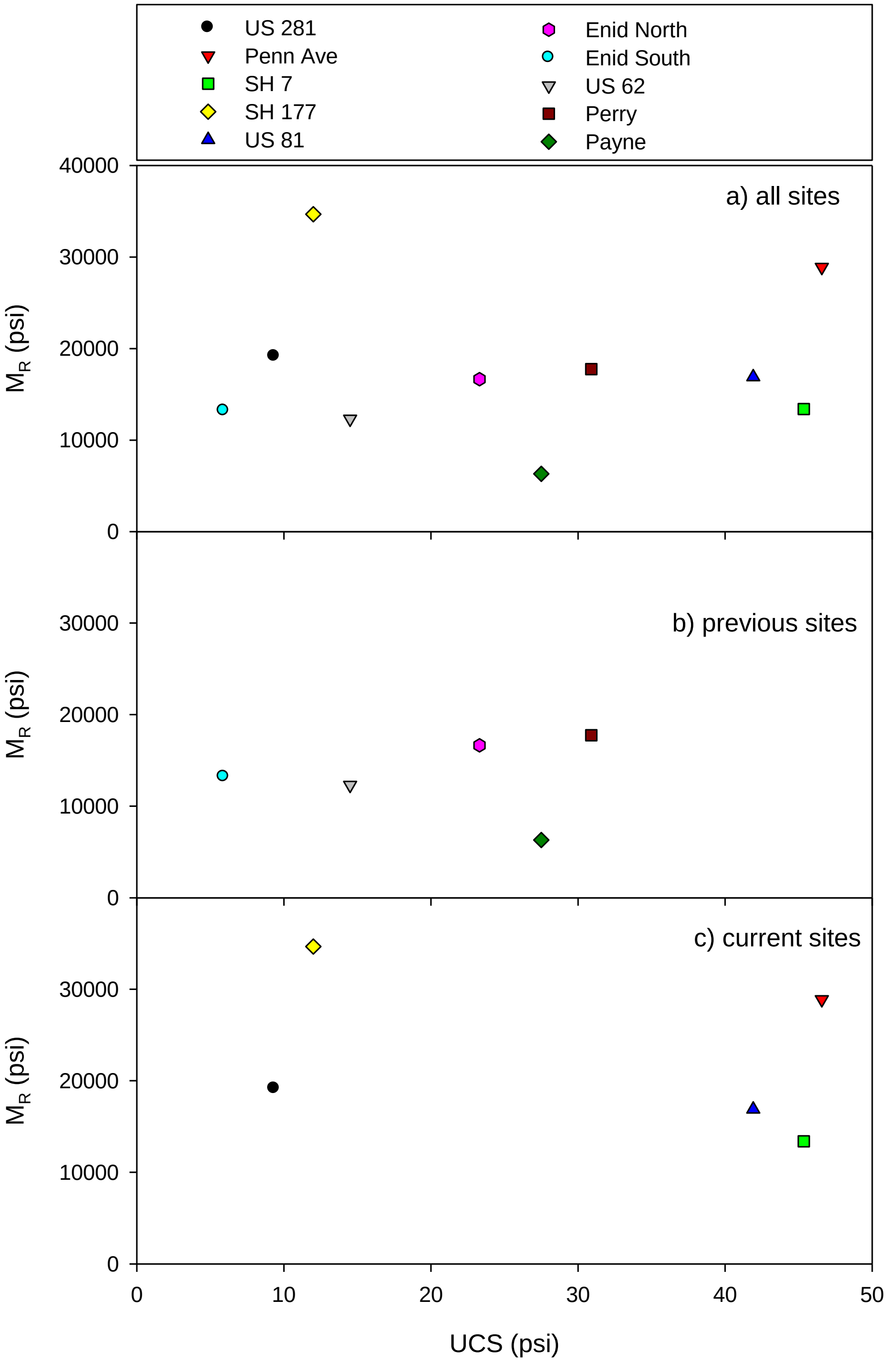


Figure 26: UCS versus M_R for Untreated Soils – a) all test sites, b) test sites from previous study and c) test sites from current study

That the untreated UCS does not correlate to untreated M_R is largely attributed to the fact that UCS depends nearly entirely on cohesion due to the lack of confining stress. On the other hand, M_R is obtained under confinement and so depends on both cohesion and stress dependent frictional behavior of the soil. For soils that derive a significant component of both their strength and stiffness from cohesion, a correlation would be expected to exist; however, for soils that lack significant cohesion, the low UCS values would not be expected to correlate as well with M_R. The soils in the current and previous study covered a broad spectrum including those that have little or no cohesion (i.e. predominantly silt and sand) and those that have significant cohesion (high plastic clays). Hence, the degree to which UCS and MR depend on cohesion is expected to be variable, which explains the lack of correlation. Further, the unconfined test represented a large strain measurement with significant plastic deformation, whereas M_R was determined under low strains, representing elastic soil behavior. Hence, the relationship between strength and stiffness is expected to vary for different soil types due to differences shearing behavior.

On the other hand, the comparison of treated UCS and M_R values for individual sites generally followed a linear trend, as shown in Figure 27. Unlike the untreated soil, the presence of a chemical additive significantly increased the “cohesion” in the soil and provided a dominant role in strength and stiffness of both unconfined and confined specimens, respectively. Thus, the confining stress was less important when UCS and M_R of treated soils were compared, as the additive played a similar role in the increase of these two parameters at different curing times.

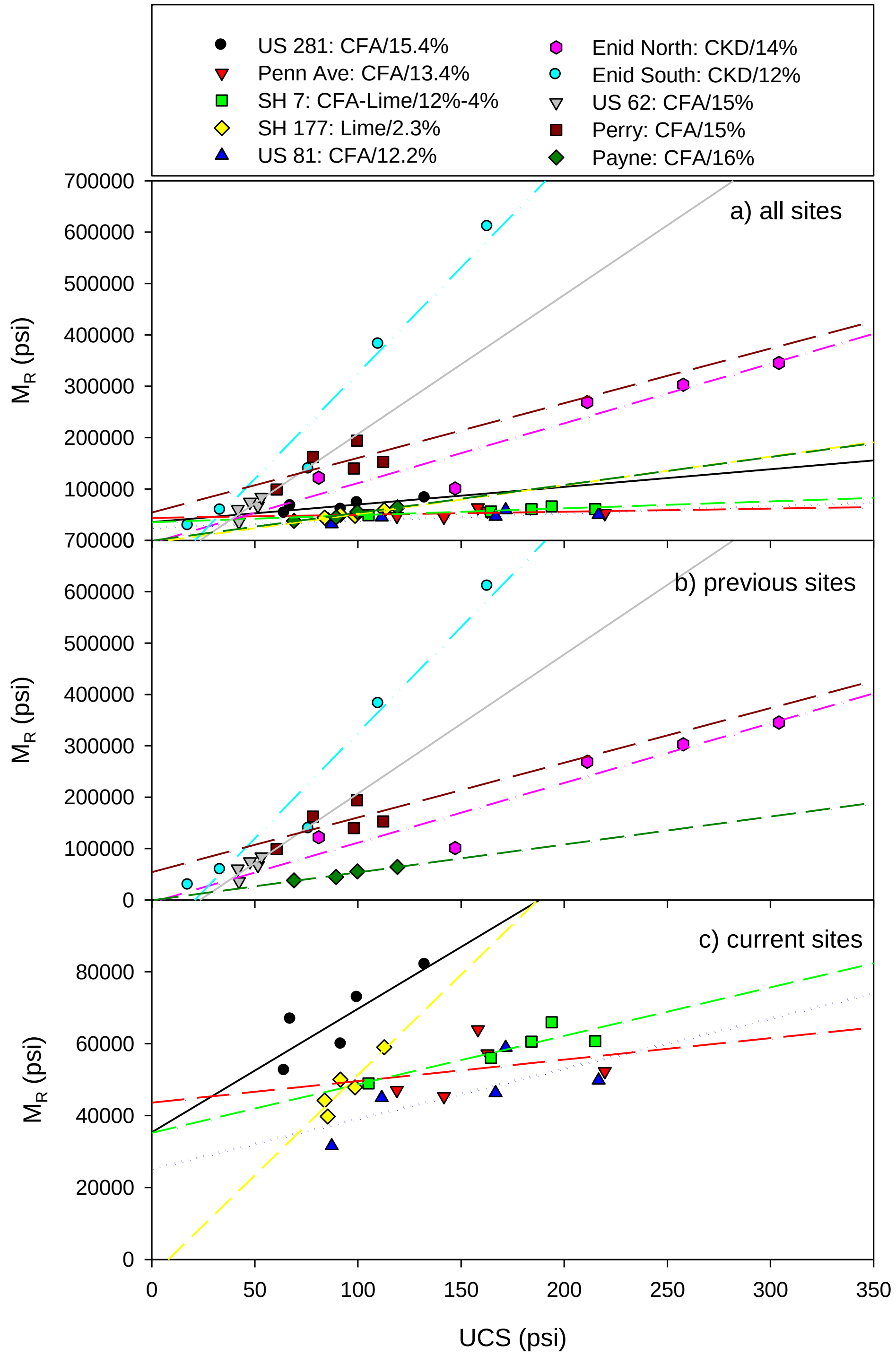


Figure 27: UCS versus M_R for Stabilized Soils at Curing Times Ranging from 1 to 28 Days – a) all test sites, b) test sites from previous study and c) test sites from current study

The slope, intercept and r^2 values for the best fit linear regression line representing the relationship between M_R and UCS for stabilized soils are summarized in Table 14. Six of the sites showed a strong correlation with r^2 values exceeding 0.7 while three of the sites showed weaker correlation with lower r^2 values. Closer inspection of the data for the four questionable sites (Penn Ave., US 177, and Perry) in Figure 27 and Table 14 reveals that while the correlations are weak, there is generally a consistent trend of increasing M_R with increasing UCS. The worst r^2 value was associated with the Penn. Ave. Site and was attributed to the unexpected, and likely erroneous decrease in M_R after 28 days of curing as discussed in Section 4.6.

Table 14: Summary of Slope and Intercept Values from Linear Regression Models Relating M_R to UCS at Different Curing Times for Stabilized Soils (M_R=b+m*UCS)

Site	Soil Type	%fines, F	SF*	Additive Type	Additive Content, AC (%)	b (psi)	M	r^2
US 281	ML	50.1	1.9	CFA	15.4	35397	497	0.70
Penn Ave	CL	74.1	3.1	CFA	13.4	43617	161	0.09
US 177	CH	93.5	2.6	LIME	2.3	25275	203	0.53
SH 7	CL	66	2.9	CFA/LIME	12/4	35239	279	0.78
US 81	ML	57.5	2.0	CFA	12.2	-4456	199	0.83
Enid North	SC	43	2.2	CKD	14	-4711	1177	0.87
Enid South	SM	22	1.0	CKD	12	-85992	3864	0.96
US 62	ML	56	1.7	CFA	15	-63435	1715	0.60
Perry	CL	77	2.9	CFA	15	54192	1813	0.39
Payne	CL	53	2.4	CFA	16	-617	630	0.96

*Stabilization Factor, SF = a*F+b*LS+c*AC+d*UCS_{1d}

LS = Untreated Linear Shrinkage, UCS_{1d} = UCS measured at 1 day of curing

**AC assumed equal to 16%.

It was found, that similar to the power model parameters relating curing time to UCS and M_R, trends exist between percent of fines and the linear regression parameters relating UCS to M_R for stabilized soil. These parameters (slope and intercept from Table 14) are plotted against percent fines in Figure 28. The trends in Figure 28 are substantial; however, interestingly if the slope and intercept data for four sites with the lowest r^2 values (less than 0.7 in Table 14) are removed, the trends become much tighter as shown in Figure 29. By removing the data points with the most uncertainty in the M_R-UCS relationship (with $r^2<0.7$), the validity of the underlying correlation is strengthened. The same precautions for using Figures 17 and 24 apply to Figures 28 and 29, in that the early part of the curves are largely attributed to CKD treated soils while CFA soils dominate at higher fines contents.

In Figure 30, the slope and intercept values relating UCS and M_R at different curing times are plotted against the stabilization factor, SF, for all ten sites. The correlations are slightly better than seen in Figure 28, and again this relationship is considered more valid in that SF better captures the influence of soil and additive properties.

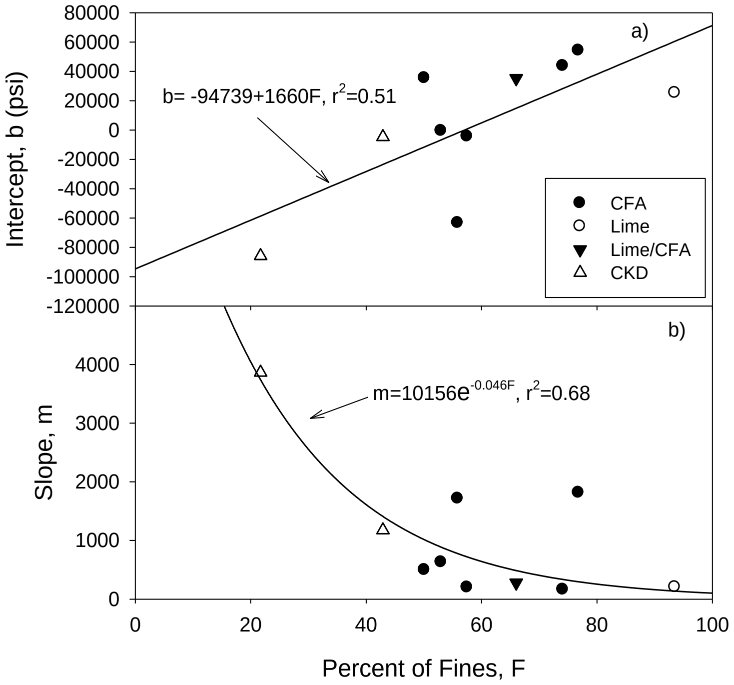


Figure 28: Slope and Intercept Values from Relationship between M_R and UCS Strength versus Fines

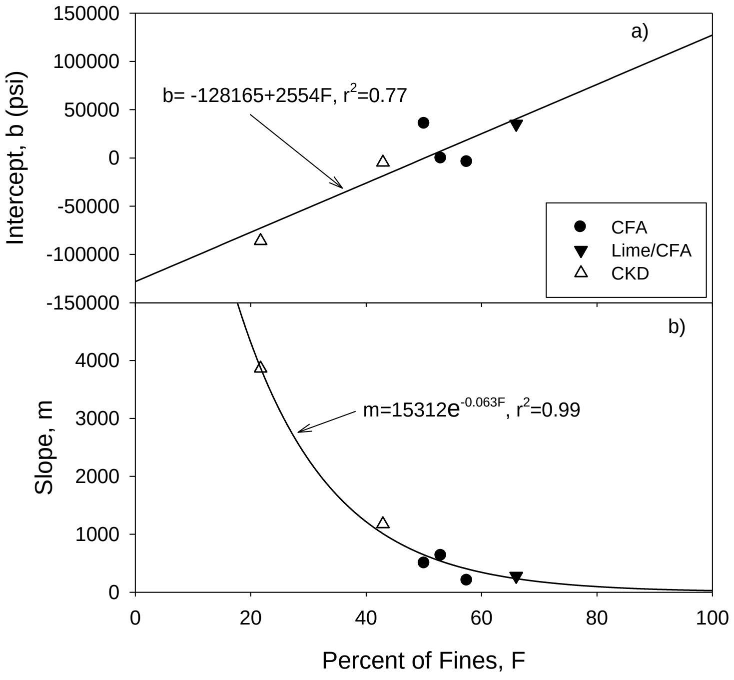


Figure 29: Slope and Intercept Values from Relationship between M_R and UCS for Six Soils versus Percent Fines

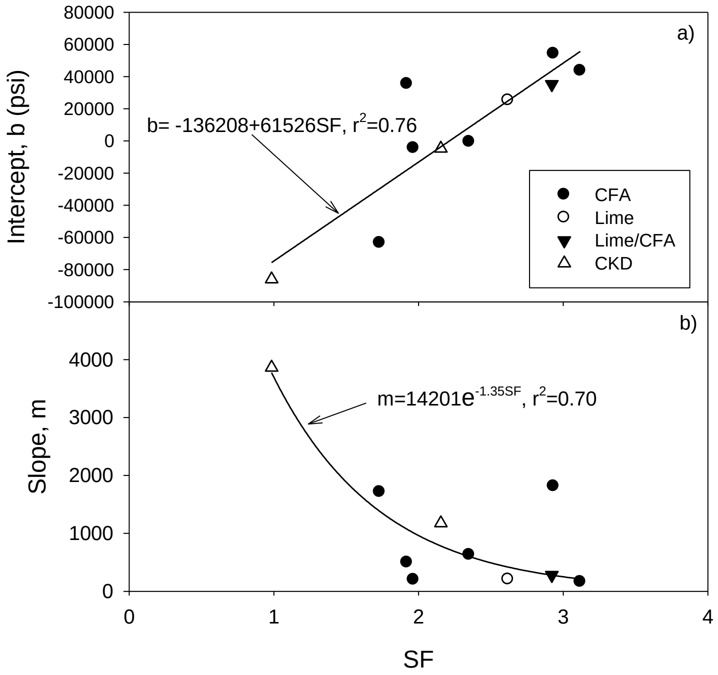


Figure 30: Slope and Intercept Values from Relationship between M_R and UCS Strength versus Stabilization Factor

4.8 Field Testing

Field testing at each site included DCP, PANDA penetration and PFWD. After initial testing on the untreated subgrade, additional tests were conducted at different curing times following compaction of the treated subgrade. Due to the construction schedule conflicts and limited site access, field testing was conducted at varying curing times for each site during a 28 day period. At some sites the testing was more extensive while at others, for example SH 7, only a few tests were conducted early in the curing process. Nevertheless, some valuable observations and insights were gained through the field testing. The goal was to see whether field test results would reflect strength and stiffness gains in the treated subgrade over time and to compare them to corresponding laboratory measurements. The results of field tests at each site are shown in Figures 31 through 35, and are discussed in subsections that follow. Data points corresponding to each of the three testing locations are shown for each curing time along with the average trend (solid line). Tabulated results are provided in Appendix D.

The DCP results are interpreted using dynamic cone index (DCI), which expresses the penetration resistance in units of mm/blow. Thus, a smaller value indicates a stronger soil. However, for the PANDA, tip resistance (q_d) is plotted in units of psi, where q_d increases with increasing strength. Therefore, to facilitate the visual comparison of the DCP and PANDA results, the inverse of DCI (i.e.1/DCI) is plotted with units of blows/mm, so that increasing penetration resistance is similarly reflected by increases in 1/DCI. The PFWD results are expressed as the elastic modulus (E_{vd}) in units of psi.

4.8.1 Site #1, US 281 Field Testing

The results of the DCP, PANDA penetration, and PFWD testing are shown in Figure 31 for Site #1. While there was considerable variation from point to point for results of all three tests, there were strong similarities in the trends for each test. That the results of all three field tests are in agreement suggests that any one of the tests is a reasonably good indicator of changes in field strength and stiffness for this case. Generally, the strength as reflected by the DCP and PANDA penetration resistance and stiffness as reflected by the PFWD modulus increased substantially over the untreated subgrade after six days of curing.

However, after six days a decrease of strength and stiffness occurred that became more significant with curing time.

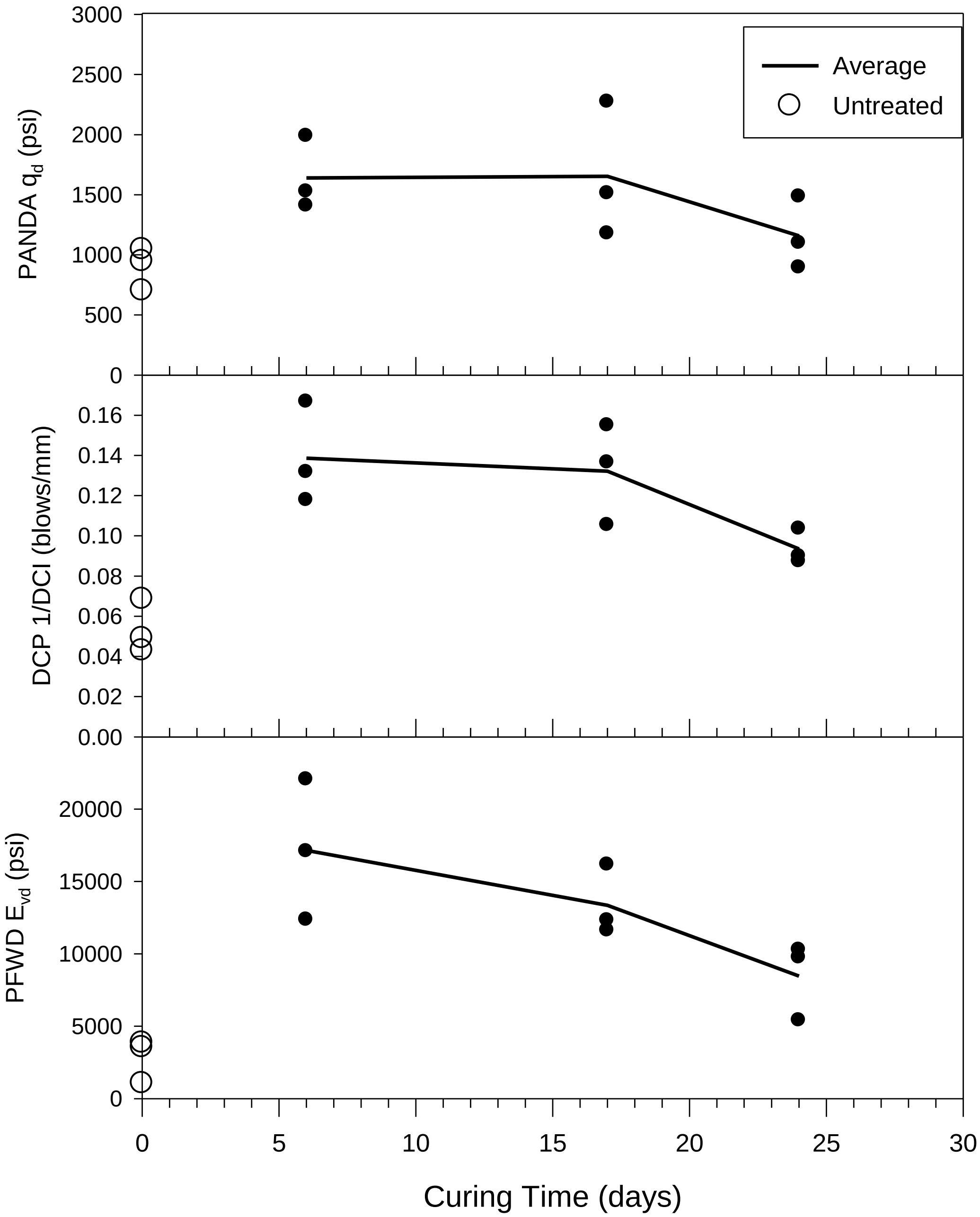


Figure 31: Results of Field Testing at Site #1, US 281

The decrease in strength after six days was most likely due to a substantial rainfall at the roadway construction site. Approximately 2.1 inches fell between six and 24 days of curing. The significant increase in water content in the stabilized subgrade probably caused considerable reduction in matric suction, which reduced the strength and stiffness. It is also possible that significant wetting of the freshly treated subgrade had an adverse affect on the results of chemical stabilization by leaching of chemical components. Whatever the cause, these results demonstrate the unwanted effects that excessive wetting can have on exposed stabilized subgrades.

4.8.2 Site #2, Penn Ave. Field Testing

In Figure 32 results show that DCP penetration resistance (1/DCI) decreased significantly after one day of curing compared to the untreated condition. A further slight decrease occurred after three days followed by a continuous and gradual increase. It is noted, however, that strength after 26 days of curing does not reach the strength of the untreated subgrade. The PFWD modulus (E_{vd}) shows a similar decrease from untreated to 1 day of curing, followed by a gradual increase until 10 days of curing, after which the modulus drops gradually. Like the DCP penetration resistance, the modulus never reached the modulus of the untreated subgrade. That the DCP and PFWD results show similar behavior tends to corroborate these results. However, the PANDA penetration test results show the one day penetration resistance was, on average, about twice as large as the untreated soil and the resistance continued to gradually increase until the final day of testing.

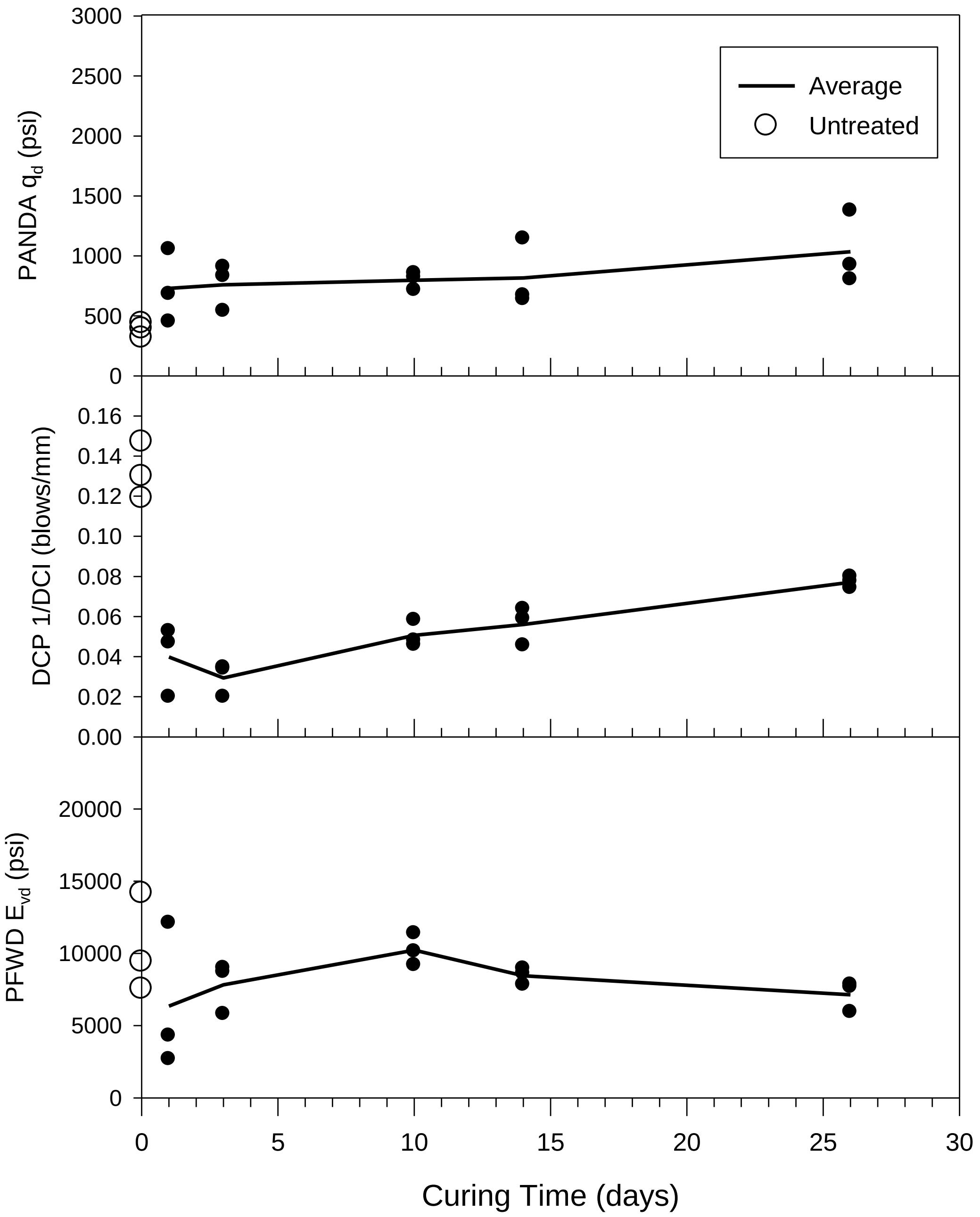


Figure 32: Results of Field Testing at Site #2, Penn Ave.

It was unclear why the behavior of the PANDA penetration resistance was different from the DCP and PFWD; however, one factor that probably influenced the results was the rainfall during this time period. It was noted that on the third day of curing, approximately 1.01 inches fell. For whatever reason, the PANDA tip resistance seemed less affected by the rain event. Another possibility was that the site experienced non-uniform application and mixing of the additive and/or significant variations in water content and density during compaction. This could partially account for the variation in the test results.

4.8.3 Site #3, US 177 Field Testing

In Figure 33 results for Site #3 are shown. Penetration resistance from DCP and PANDA show similar behavior, gradually increasing above the untreated strength as curing time increased to six days then both show a reduction at 15 days. The PFWD modulus increased significantly above the untreated strength after two days of curing and then more or less remained constant and increasing slightly at 15 days. Again, the decrease in penetration resistance at 15 days of curing is thought to be due to rainfall of approximately 0.6 inches that fell between six and 15 days of curing at the construction site.

4.8.4 Site #4, SH 7 Field Testing

Data were obtained at Site #4 for three days of curing as shown in Figure 34. At one day of curing all three tests indicated little change over the untreated soil. However, both DCP and PANDA penetration resistance and PFWD modulus increased significantly from one to three days. Site #4 was unique in that it was modified with quicklime to reduce the PI and increase workability, before stabilization with CFA.

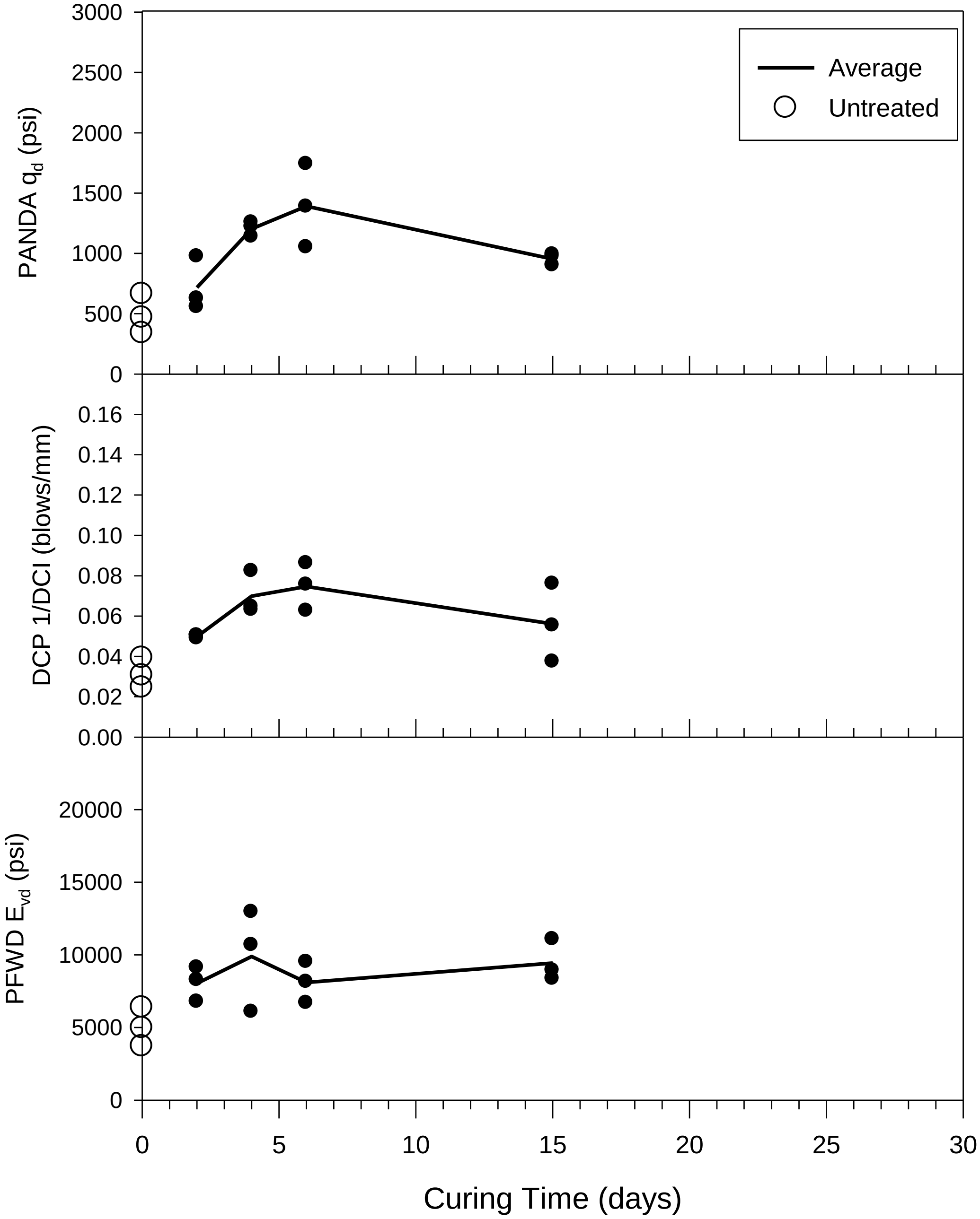


Figure 33: Results of Field Testing at Site #3, US 177

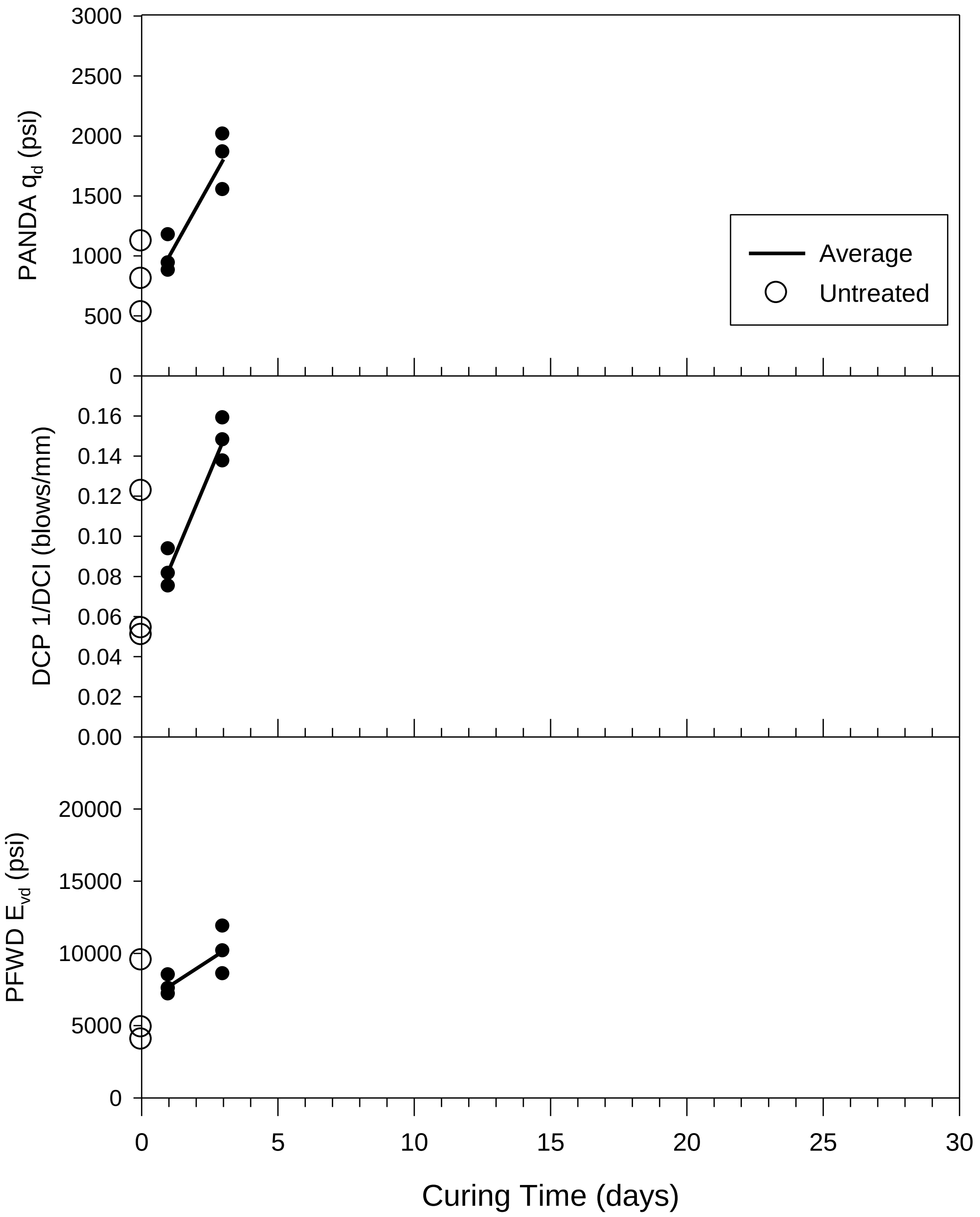


Figure 34: Results of Field Testing at Site #4, SH 7

4.8.5 Site #5, US 81 Field Testing

In Figure 35, results of field testing for Site #5 are presented. While the individual data points for each curing time for each of the field tests showed considerable scatter, results from each field tests showed similarly strong trends with curing time. These trends represent classic expectations of stabilized soil behavior, whereby strength and stiffness gradually increase above the untreated strength as curing time increases. It is noted that there are some anomalously high penetration resistance values for the untreated soil.

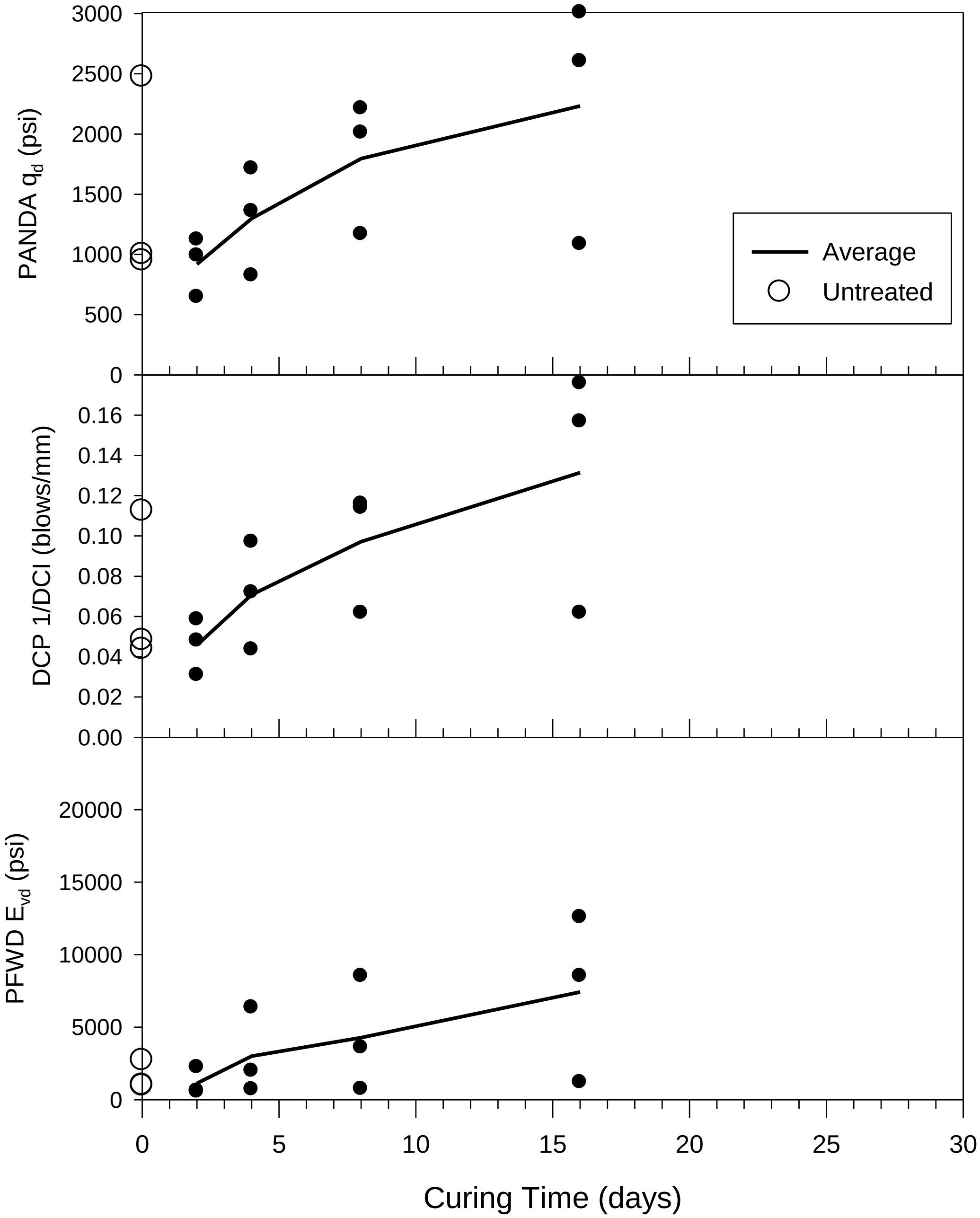


Figure 35: Results of Field Testing at Site #5, US 81

Contrary to Sites #1, #2 and #3, Site #5 received no rainfall during the curing period investigated. The significant differences in strength and stiffness behavior between curing times when rainfall occurs and doesn't occur strongly suggests that significant subgrade wetting during early curing has an adverse effect on stabilized subgrade performance. It remains to be seen to what degree these adverse effects are reversible with continued curing and drying (time permitting) of the stabilized subgrade.

4.9 Comparison of Field and Laboratory Strength and Stiffness

The average results of field testing were compared with the average results from resilient modulus and unconfined compression tests in Figures 36-41. Laboratory test results represent untreated samples and stabilized samples that were mixed in the laboratory. Field and laboratory test results were compared at the same or most similar curing times. A tabulated summary of the results and corresponding curing times used to construct these figures is presented in the Appendix E.

In examining Figures 36, 37 and 38, considerable scatter in the resulting plots relating field and lab test results with no significant trends was observed. Generally, it would be expected that a field penetration test result would correlate strongly to a strength test in the lab on the same material at the same curing time and that this relationship would be somewhat independent of soil and additive type. That is, if the materials being tested in the field, for example with the DCP, are identical to the materials used in the strength test (UCT) in the laboratory, then it is expected that the penetration resistance would be a somewhat unique function of the laboratory strength. The same expectation would apply to field and lab stiffness measurements under similar conditions.

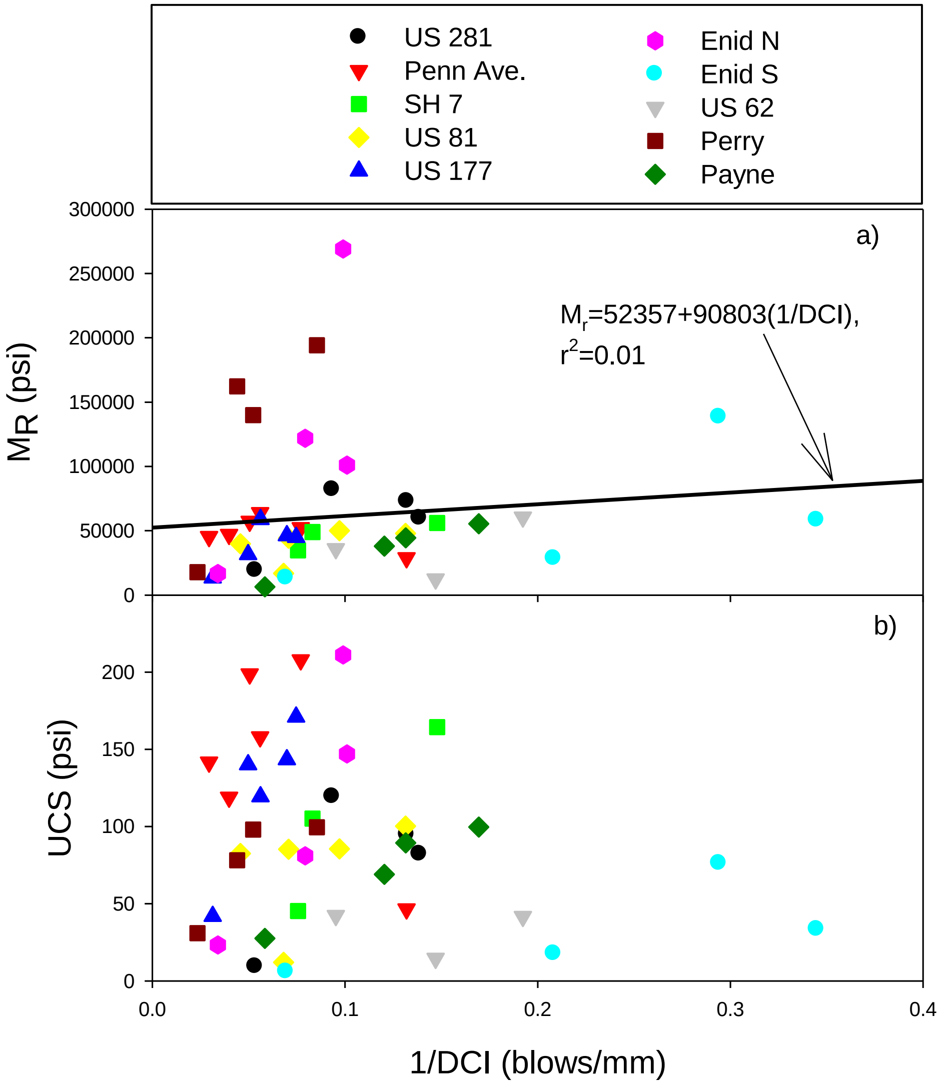


Figure 36: Comparison of Results of DCP Field Tests with Laboratory Measurements of M_R and UCS at Similar Curing Times

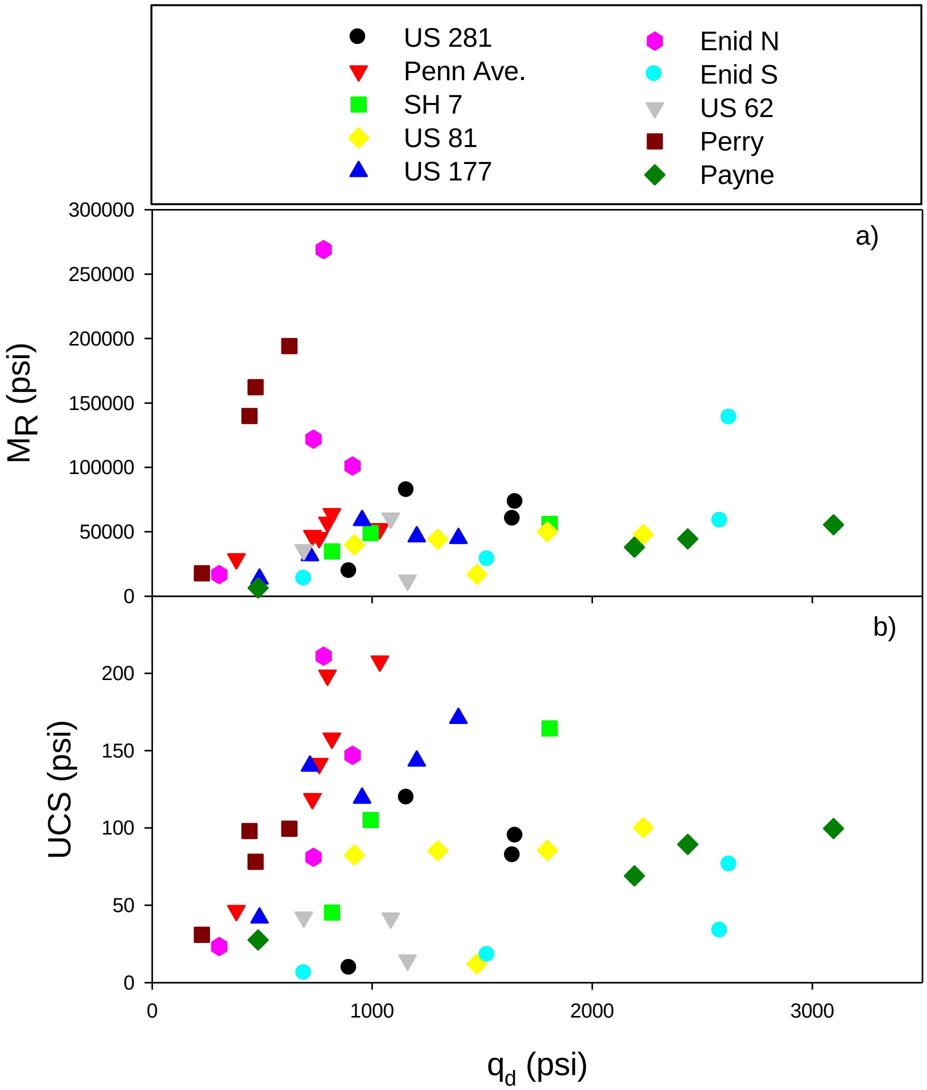


Figure 37: Comparison of Results of PANDA Penetrometer Field Tests with Laboratory Measurements of M_R and UCS at Similar Curing Times

It is assumed that the mechanics governing penetration resistance and field stiffness are a unique function of the shear strength and modulus, respectively, being determined for identical materials in the laboratory. This relationship is expected to be dependent on soil type to some degree, considering variations in particle size and drainage behavior (during shearing), etc. However, for stabilized fine-grained soils these variations may be small given that the soils

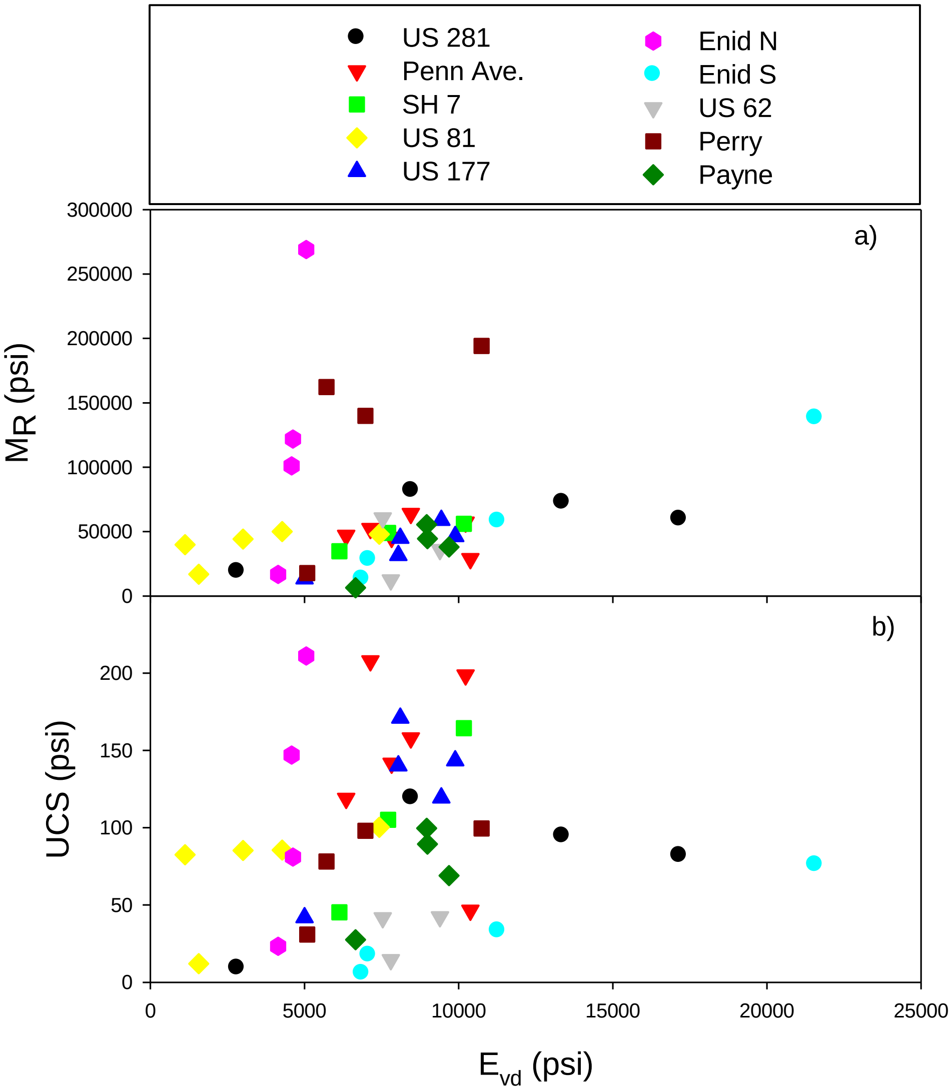


Figure 38: Comparison of Results of PFWD Field Tests with Laboratory Measurements of M_R and UCS at Similar Curing Times

are typically unsaturated and the cementation from the additive plays a dominant role in the strength and stiffness of both laboratory and field specimens. In an earlier study, Miller and Zaman (2000) observed that trends in unconfined compression strengths after curing for 28 days were similar to corresponding DCP and FWD field test results conducted on the subgrade after curing times of 28 to 56 days. The test site in that study involved approximately four 1000 foot long continuous test sections each treated with a different stabilizer including quick lime and CKD from three different sources.

That there was considerable scatter in the results of the comparison of field and laboratory strength and stiffness in the current study suggests the materials being tested were not identical. There are several reasons for this. First, it was known that the curing conditions were much different in the field and laboratory. The field strength and stiffness, because of variations in climate (rainfall and possibly temperature), did not behave the same as the laboratory strength and stiffness, which was likely a significant source of scatter in Figure 36 to 38. As discussed previously, the effects of the rain events had a major influence on the field test results at different curing times.

A closer look at Figures 36a, 37a, and 38a, reveals that if the data from Enid North and Perry were removed, then a relatively weak trend becomes apparent in the relationships between M_R and field test results as shown in Figures 39a, 40a and 41a. While the coefficients of determination for the regression lines were relatively low, there was a noticeable trend consistent with expectation, with the most significant trend observed for the PFWD data. No significant trends were seen in the comparisons of field and UCS test results.

The results of the comparison of field and laboratory tests from this and the previous study indicated that to establish robust correlations, field and laboratory strength and stiffness testing needs to be conducted on soil with identical properties and curing histories. Such an approach could involve compacting and curing soil in a laboratory environment and subjecting the same soil bed to both laboratory and field testing. This would eliminate the influence of environmental conditions during curing and substantially reduce variations in additive content, dry density and water content between soil subjected to laboratory and field tests.

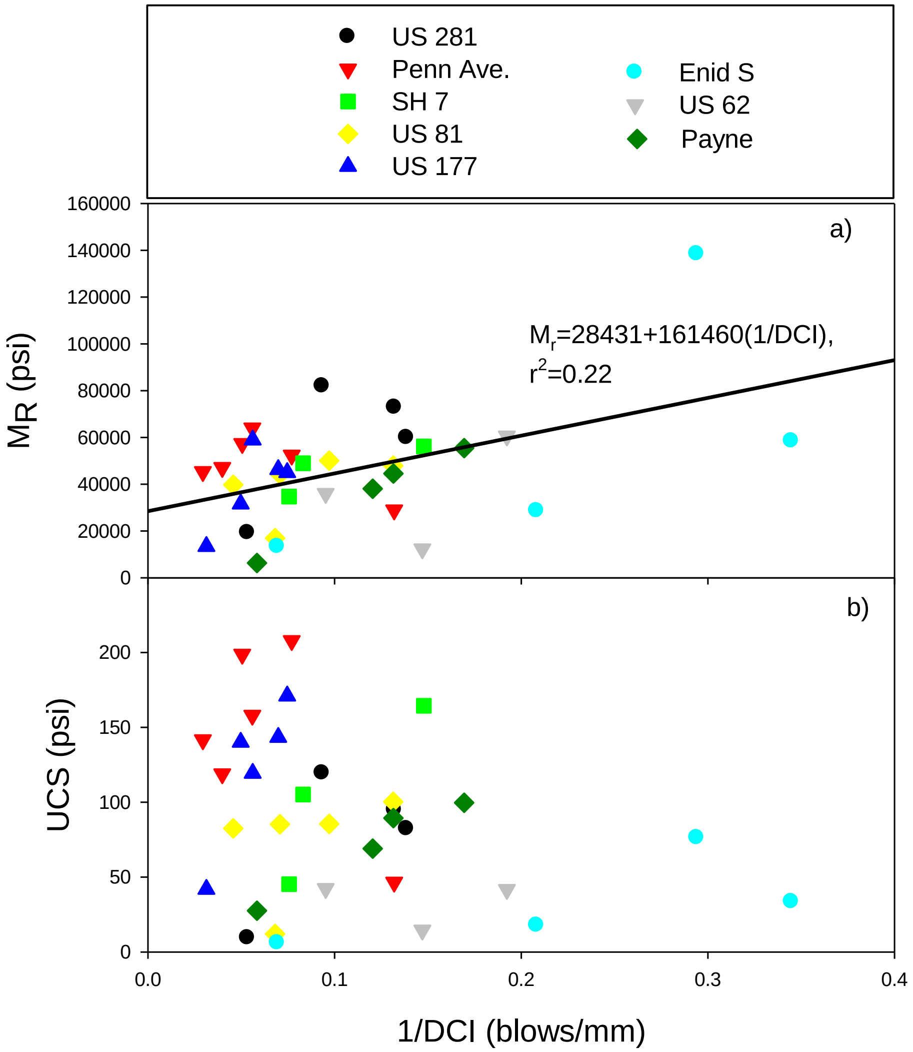


Figure 39: Comparison of Results of DCP Field Tests with Laboratory Measurements of M_R and UCS at Similar Curing Times with Enid North and Perry Site Data Removed

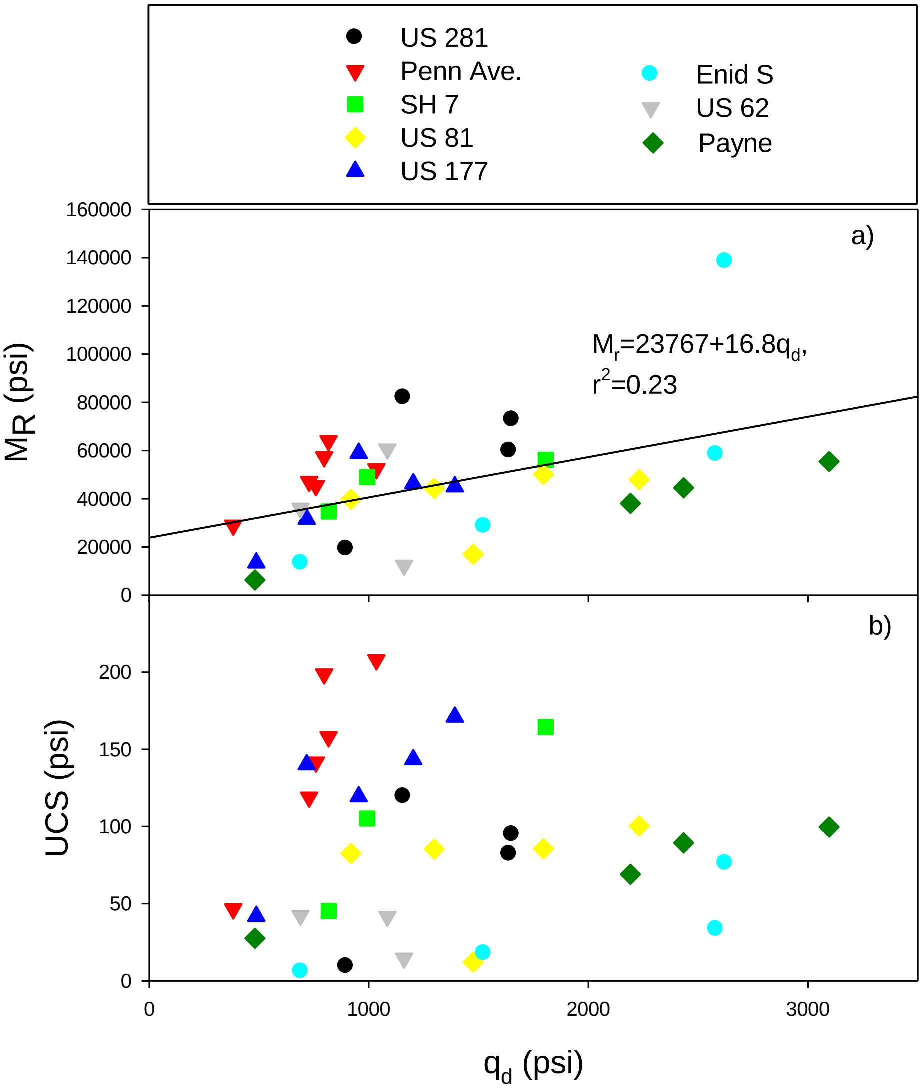


Figure 40: Comparison of Results of PANDA Penetrometer Field Tests with Laboratory Measurements of M_R and UCS at Similar Curing Times with Enid North and Perry Site Data Removed

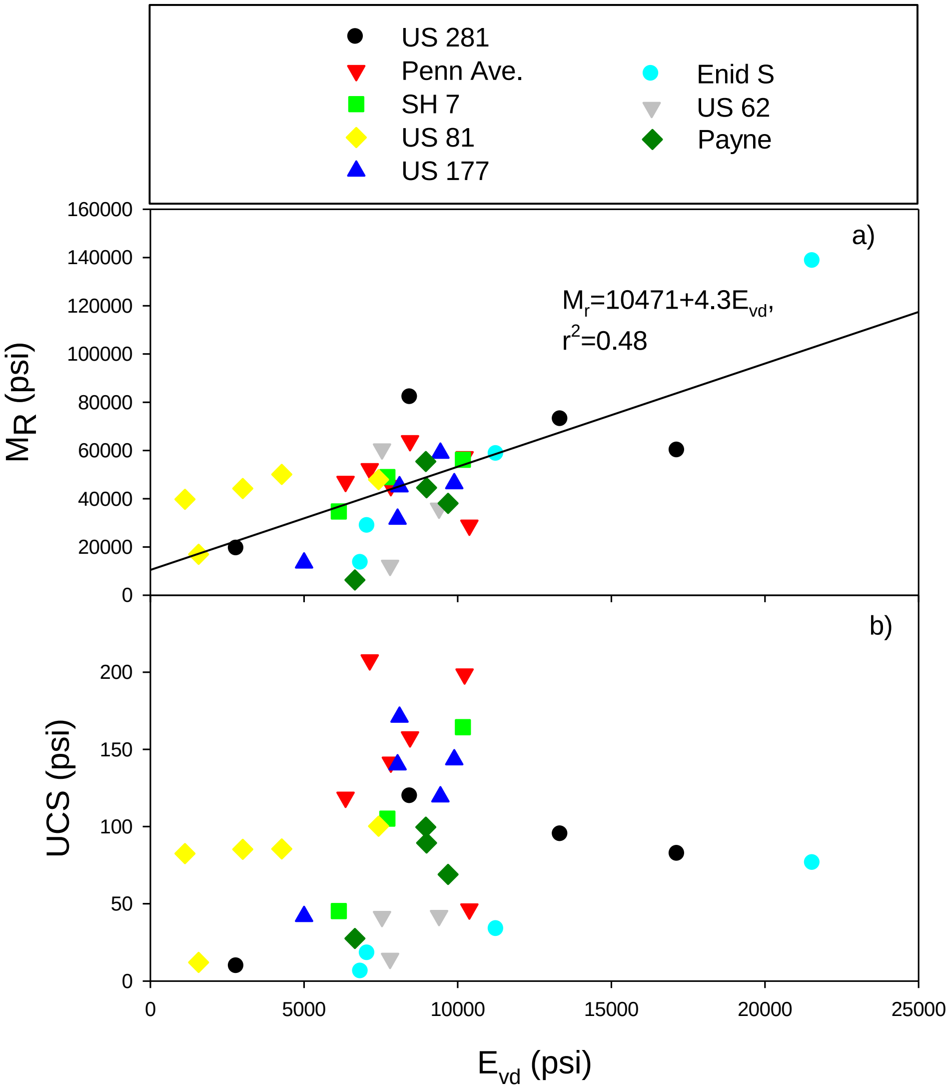


Figure 41: Comparison of Results of PFWD Field Tests with Laboratory Measurements of M_R and UCS at Similar Curing Times with Enid North and Perry Site Data Removed

5 Summary, Conclusions and Recommendations

5.1 Introduction

Unconfined Compressive Strength (UCS) and Resilient Modulus (M_R) were determined for chemically treated soil for curing times up to 28 days using soils and additives obtained from five field test sites in Oklahoma. Untreated soils were also tested for comparison. Samples were prepared at additive contents similar to those determined from testing field samples. Additive contents were estimated using a new approach involving a technique based on x-ray florescence (XRF). Several other tests to characterize the physical and chemical behavior were conducted on treated and untreated soils. The five test sites involved construction of chemically treated highway subgrades using lime and/or Class C fly ash (CFA). In addition to laboratory testing, field tests were conducted on the untreated compacted subgrade at various times following mixing and compaction of the treated subgrade. The analysis and comparison of results led to the following conclusions.

5.2 Summary and Conclusions

1. The Whole Rock Analysis Method using XRF shows promise as a method to estimate additive contents in samples obtained from the field. For a series of samples prepared in the lab with known additive quantities, the XRF testing determined the additive content amount with reasonable accuracy up to about 10 to 12% additive content. For lime, using XRF the additive content was overestimated by about 0.5% to 1% (by weight) at additive contents of 1

to 5%. For CKD and CFA the additive content was generally underestimated using XRF by about 0.5% at low additive percentages and about 3% at the highest additive content.

2. Increases in UCS and M_R of treated soil with increases in curing time were modeled reasonably well using a two parameter power equation ($UCS=UCS_1t^{Rtu}$; $M_R=M_{R1}t^{Rt}$). For UCS data obtained for five sites from this study and five sites from a previous study (Snethen et al. 2008), the coefficient of determination, r^2, for the best fit model for each site, ranged from 0.37 to 0.99, with a median r^2 of 0.80. For the corresponding M_R data the range and median of r^2 values were 0.38 to 0.99 and 0.84, respectively. The power model provides a rapid means of estimating UCS or M_R at any curing time based on UCS or MR at one day of curing (i.e. UCS_1, M_{R1}) and an estimate of the rate exponent (i.e. R_{tu}, R_t).
3. A correlation appears to exist between the rate exponents (i.e. R_{tu}, R_t) and percent of fines for the data from the ten sites mentioned above. These rate exponents were found to decrease with increasing fines content. A simple exponential decay equation was found to provide the best fit to the observed trends. While the r^2 value (0.81) for the best fit model was reasonably good, it was noted that at low fines contents the relationship is defined by data from two sites where cement kiln dust (CKD) was used and at greater fines contents the relationship was controlled by CFA treated soils. This observation suggests the strength of the correlation may be an artifact of this bias. Thus, it was concluded that the model may not be appropriate for

CKD treated soils at high fines contents or CFA treated soils at low fines contents.

4. A stabilization factor (SF) was developed as an attempt to capture the influence of the fines content, nature of fines, additive content, and additive effectiveness in a single parameter. The parameter SF is simply the sum of four normalized properties: percent fines (F), linear shrinkage of the untreated soil (LS), additive content (AC) and measured unconfined compression strength of the treated soil after one day of curing (UCS_{1d}). This parameter was found to provide good correlations to R_{tu} and R_t. While the correlations were not as robust compared to those using percent fines, the parameter SF is a more logical choice given its general applicability.
5. No obvious trend existed between untreated UCS and M_R. This is largely attributed to the fact that UCS depends nearly entirely on cohesion due to the lack of confining stress. On the other hand, M_R is obtained under confinement and thus depends on both cohesion and stress depend frictional behavior of the soil. For soils that derive a significant component of both their strength and stiffness from cohesion, a correlation would be expected to exist; however, for soils that lack significant cohesion, the low UCS values would not be expected to correlate with M_R. The soils in the current study cover a broad spectrum including those that have little or no cohesion (i.e. predominantly silt and sand) and those that have significant cohesion (high plastic clays). Hence, the degree to which UCS and MR

depend on cohesion is expected to be variable, which may explain the lack of correlation in the untreated soils.

6. Unlike the untreated soil, generally, the UCS and M_R corresponding to a given site and curing time should be better correlated because both depend strongly on the added cohesion due to the cementation provided by the chemical additive. A linear relationship (M_R=b+m*UCS) was found to provide a good fit of the data in a plot of M_R versus UCS for most sites. Coefficient of determination, r^2, ranged from 0.09 to 0.96 with a median value of 0.74. If values of the slope (m) and intercept (b) of this relationship can be estimated, then the model provides a means of estimating M_R based on a UCS test.

   It was found that m and b were correlated reasonably well with percent fines; the same cautions apply as discussed previously. When compared to percent fines, the stabilization factor (SF) in this case actually provided a significantly stronger correlation with parameter b, and marginally better with respect to parameter m. Again, SF is more generally applicable to different soil and additive types.

7. Curing temperature was found to have an influence on UCS of some chemically treated soils. Soils with low plasticity showed little effect of curing temperature. However, generally higher plasticity soils tended to gain less strength at lower temperature (40^{o}F) compared to higher temperature (room temperature) at the same curing time. This observation is important in

comparing strength and stiffness obtained under field and laboratory conditions where curing temperatures are likely to be different.

8. The DCP and PANDA penetration tests, and the PFWD test showed similar trends at most field test sites and captured the increase in strength and stiffness of stabilized subgrade soils with increasing curing time. The test results also reflected a decrease in strength and stiffness at some sites due to significant rain events during curing.

   In addition to being sensitive to changes in strength and stiffness of treated subgrades due to curing and adverse weather effects, the field test results also reflected the variability of strength and stiffness at test sites. Thus, the field tests in question show significant promise as tools for monitoring quality and improvement of stabilized subgrades during construction.

9. The UCS and M_R values obtained from the laboratory mixed and cured samples were compared to results of DCP, PANDA, and PFWD field tests for ten sites (five previous, five current). It was found that there was little or no correlation between the field and laboratory strength and stiffness. This was attributed to the significant differences in the curing conditions (i.e. weather) that played a significant role in the results of field tests. However, when data from two sites, that were significantly different, were removed from the data set, a weak correlation was observed between M_R and field test results for the remaining eight sites. The strongest trend was observed for the PFWD – M_R comparison. The trend showed that both the PFWD

modulus and M_R increase with increasing curing time, as expected. These observations show that development of correlations between field and laboratory test results holds promise. However, development of such correlations will require that field and laboratory tests be performed on nearly identical soils and under identical curing conditions.

5.3 Recommendations for Implementation

Several good correlations resulted from comparing laboratory tests from ten different soil stabilization sites in Oklahoma, representing three chemical stabilizers and a broad range of mostly fine-grained soil types. Five of the ten test sites were part of a different research project, approximately three years previous to the current study, and involved different test operators and slightly different test procedures compared to the current study. Nevertheless, the strength of the correlations developed with these data sets was good. This suggests that the test parameters used in development of these correlations were appropriate and based on reliable test data. These correlations would be useful in making preliminary estimates of strength and resilient modulus for use as Level 2 inputs in the pavement design process. Following are four scenarios for predicting M_R, involving progressively more laboratory testing.

<u>Scenario 1: Utilizes Percent of Fines and 1-Day UCS</u>

Required Laboratory Test Parameters: % fines (F), UCS on treated soil after 1 day of curing (UCS_{1d}).

Procedure to Estimate M_R: **1)** estimate R_{tu} using Equation A in Table 15, **2)** estimate UCS at the design curing time using Equation E in Table 15 with

UCS_1=UCS_{1d}, **3)** estimate M_R at same curing time using Equations F, G, L (or H, I, L).

Scenario 2: Utilizes the Stabilization Factor (SF) and 1-Day UCS

Required Laboratory Test Parameters: % fines (F), UCS on treated soil after 1 day of curing (UCS_{1d}), additive content (AC), linear shrinkage for untreated soil (LS).

Procedure to Estimate M_R: **1)** compute stabilization factor using Equation D in Table 15, **2)** estimate R_{tu} using Equation B (or C), **3)** estimate UCS at the design curing time using Equation E in Table 15 with UCS_1=UCS_{1d}, **4)** estimate M_R at same curing time using Equations J, K, L.

Scenario 3: Utilizes Percent of Fines and 1-Day Resilient Modulus

Required Laboratory Test Parameters: % fines (F), M_R on treated soil after 1 day of curing (M_{R1d}).

Procedure to Estimate M_R: **1)** estimate R_t using Equation M in Table 15, **2)** estimate M_R at the design curing time using Equation O in Table 15 with M_{R1}=M_{R1d}.

Scenario 4: Utilizes Stabilization Factor, SF, and 1-Day Resilient Modulus

Required Laboratory Test Parameters: % fines (F), UCS on treated soil after 1 day of curing (UCS_{1d}), additive content (AC), linear shrinkage for untreated soil (LS), M_R on treated soil after 1 day of curing (M_{R1d}).

Procedure to Estimate M_R: **1)** compute stabilization factor using Equation D in Table 15, **2)** estimate R_t using Equation N in Table 15, **3)** estimate M_R at the design curing time using Equation O in Table 15 with M_{R1}=M_{R1d}.

Table 15: Summary of Equations for Level 2 Estimates of M_R

ID	Equation	r^2	N	Required Parameters	Figs. / Tables	Purpose
A	$R_{tu} = 1.27\ e^{-0.03F}$	0.81	10	F	17	Estimate UCS rate exponent
B	$R_{tu} = 1.26\ e^{-0.85SF}$	0.53	10	F, AC, UCS_{1d}, LS	18	Estimate UCS rate exponent
C	$R_{tu} = 1.25\ e^{-0.72SF}$	0.91	8	F, AC, UCS_{1d}, LS	18	Estimate UCS rate exponent
D	$SF = F/100 + LS/20 + AC/20 + UCS_{1d}/120$	---	---	F, AC, UCS_{1d}, LS	---	Stabilization Factor
E	$UCS = UCS_1\ t^{R_{tu}}$	varies	varies	UCS_{1d}	15, 16 / 11	Estimate UCS at curing time, t
F	$b = -94739 + 1660F$	0.51	10	F	28	Compute intercept b for Eq. L
G	$m = 10156\ e^{-0.046F}$	0.68	10	F	28	Compute slope m for Eq. L
H	$b = -128165 + 2554F$	0.77	6	F	29	Compute intercept b for Eq. L
I	$m = 15312\ e^{-0.063F}$	0.99	6	F	29	Compute slope m for Eq. L
J	$b = -136208 + 61526SF$	0.76	10	F, AC, UCS_{1d}, LS	30	Compute intercept b for Eq. L
K	$m = 14201\ e^{-1.35SF}$	0.70	10	F, AC, UCS_{1d}, LS	30	Compute slope m for Eq. L
L	$M_R = b + m*UCS$	varies	varies	UCS	27 / 14	Estimate M_R from UCS
M	$R_t = 2.99\ e^{-0.05F}$	0.95	10	F	24	Estimate M_R rate exponent
N	$R_t = 4.13\ e^{-1.49SF}$	0.86	10	F, AC, UCS_{1d}, LS	25	Estimate M_R rate exponent
O	$M_R = M_{R1}\ t^{Rt}$	varies	varies	M_{R1d}	19-23 / 13	Estimate M_R at curing time, t

Notes: n = number of data points, F = % fines, AC = additive content (%), UCS_{1d} = UCS determined at one day curing (psi), LS = bar linear shrinkage for untreated soil (%), M_{R1d} = M_R determined at one day curing

In addition to being an excellent tool for Level 2 predictions of M_R, equations in Table 15 can be useful where values of UCS or M_R are needed quickly. The laboratory procedures are simple and can be performed within a couple of days. For example, consider a situation where an unexpected soil type change is encountered along a highway alignment during construction involving soil stabilization. Equations in Table 15 allow for a rapid estimate of the M_R-curing time relationship by conducting a one-day curing time UCS and/or M_R test, along with one to three other simple tests. Thus, possible significant delays in construction that could occur if a seven-day or 14-day curing time was used can be avoided. A conservative lower bound estimate of R_t could be utilized at first and then verified by conducting another M_R test at a later curing time while construction proceeds.

In addition to the laboratory tests, three different field tests (DCP, PANDA, PFWD) were conducted at the ten sites for comparison to laboratory M_R and UCS values. While field and laboratory stiffness did not correlate well, some promising trends were observed between laboratory M_R and field test results, particularly the PFWD results (see Fig. 41a). More research as recommended below is needed to develop reliable correlations between M_R and field test results. On the other hand, at individual sites, the field tests were very good indicators of strength and stiffness gain over time and spatial variability in subgrade properties. The tests were especially useful to assess the influence of substantial climatic events early in the curing process. For these reasons, the use of field tests is recommended as indicators of strength and stiffness gains in the field. The tests should be performed on the untreated subgrade and then at regular intervals after treatment starting early in the curing process (e.g. immediately after treating and compacting, 1, 3….days).

Finally, the XRF technique for estimating field additive content was investigated. While the technique was not perfect, it showed promise and could be an extremely useful tool, particularly during forensic investigations where simply the presence or lack of additive in a stabilized layer is in question. It is recommended that for projects involving chemical stabilization, samples of additives, untreated soil, and treated soil (after compaction and final grading) be obtained at three or more representative locations and representing the full design depth of the stabilized layer. The samples are small and can be retained for future testing should disputes arise, or they can be incorporated into the quality control program – recognizing the testing times and limitations involved.

5.4 Recommendations for Research

The following are recommendations for future research based on the findings of this study:

1. Test more natural soils, chemical additives, and treated soils using the Whole Rock Analysis Method with XRF to determine the accuracy of additive content determination for soil types and chemical additives not studied here. The current study suggested that additive contents determined by the XRF method begin to systematically deviate from actual additive contents above 10%. Thus, a correction may be necessary for increasing additive contents. Additional research will help to explore this

possibility. The XRF method could be very useful in quality control applications and forensic investigations, because currently there are no simple and accurate methods for making discrete measurements of field additive contents. A drawback is that the method requires sending the samples to a laboratory for testing and so timely results may not be achieved; this will be generally less important for forensic investigations.

2. To develop reliable correlations between field and laboratory strength and stiffness requires conducting both field and laboratory tests under identical conditions. To this end, it is recommended that a study be conducted whereby laboratory and field soil and curing conditions are the same. One idea would be to produce soil test beds in a laboratory under controlled conditions where field tests could be conducted and lab testing samples could be obtained. A major advantage to this approach is that identical curing conditions could be maintained for laboratory-tested and field-tested soils. Additionally, the mixing of soil, additives and water could be carefully controlled and more uniform density could be achieved through careful compaction in a laboratory setting.

   Another approach would be to obtain thin-walled tube samples from field test site locations for laboratory testing so direct comparisons could be made. In addition, close monitoring of weather via nearby weather stations could be performed to assess climatic effects. This may allow for more soil and additive types to be investigated and avoid the large amount of work required to prepare soil beds in a laboratory. On the other hand, there would be less control over the site access and curing times when testing would be possible. Possibly, a combination of the in-laboratory and field approaches would be the best approach to developing reliable laboratory-field strength and stiffness correlations.

3. It would useful to conduct more UCS and M_R tests on other soil and additive combinations, along with physical and mineralogical tests, to build upon some of the promising empirical relationships established in this and previous studies.

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APPENDIX A: MOISTURE-DENSITY PLOTS

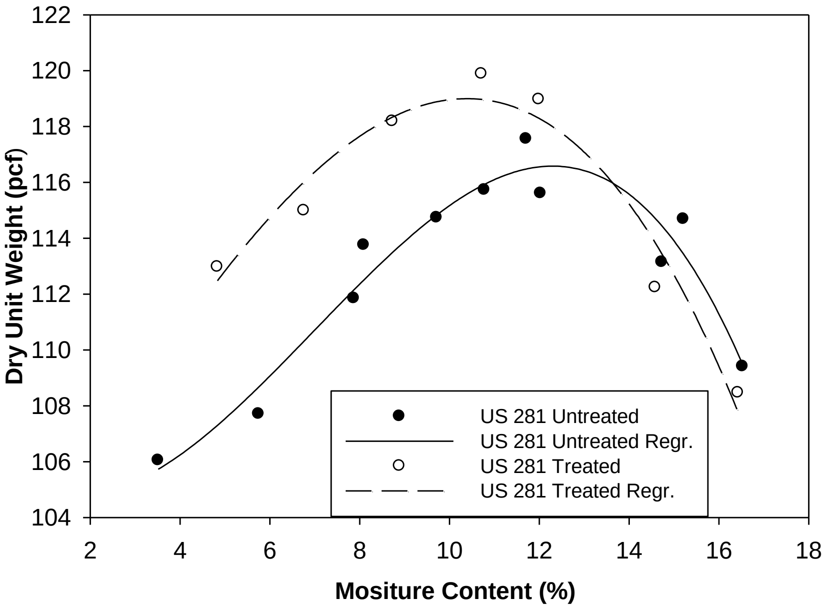

Figure A1: Moisture-Density Curves for Site #1, US 281

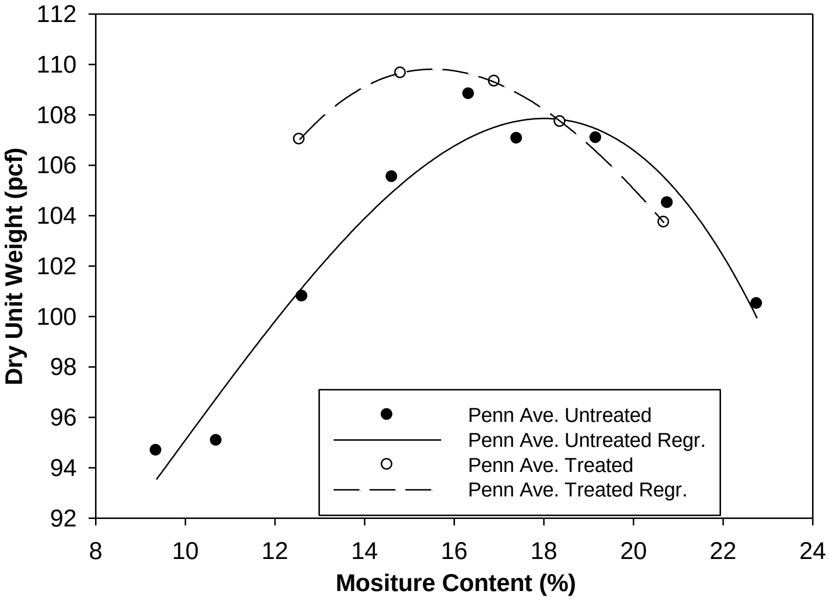

Figure A2: Moisture-Density Curves for Site #2, Penn Ave.

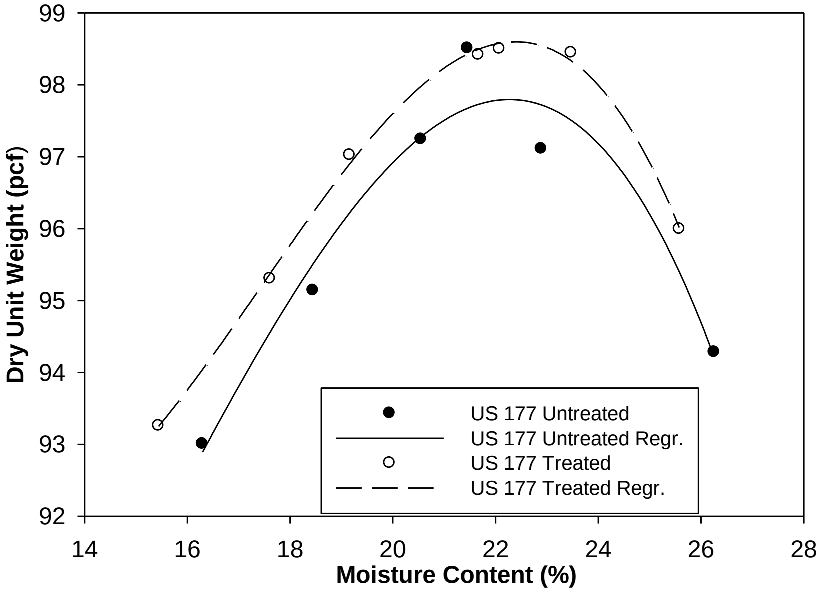

Figure A3: Moisture-Density Curves for Site #3, US 177

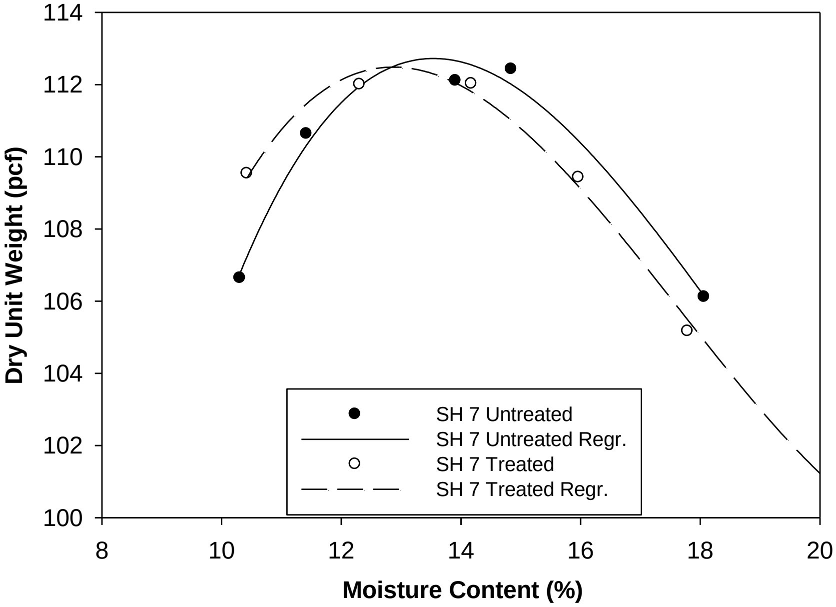

Figure A4: Moisture-Density Curves for Site #4, SH 7

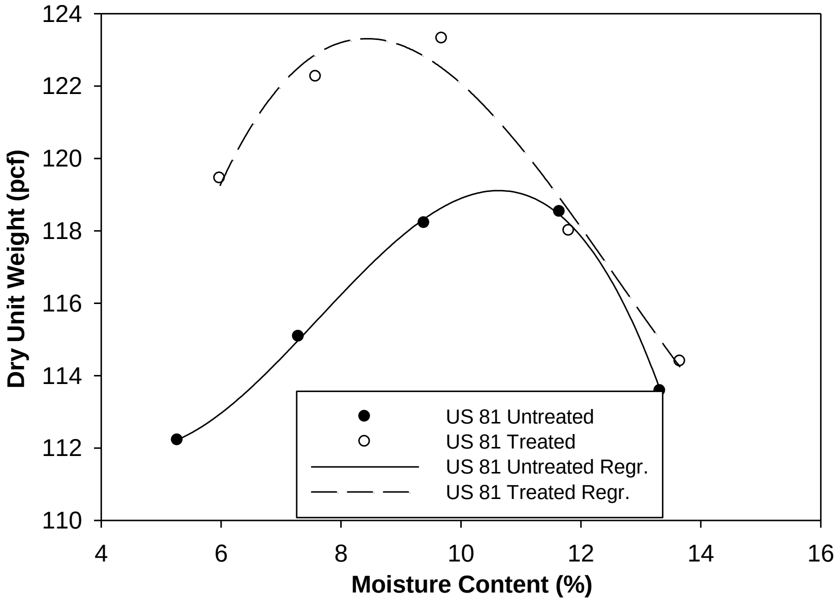

Figure A5: Moisture-Density Curves for Site #5, US 81

APPENDIX B: TABULATED SUMMARY OF UNCONFINED COMPRESSION STRENGTH

Table B1: Summary of UCS Data for Site #1, US 281

	Room Temperature Curing					40°F Curing				
Curing Time (days)	**UCS-1 (psi)**	**UCS-2 (psi)**	**UCS-3 (psi)**	**UCS-Avg (psi)**	**UCS-St.Dev. (psi)**	**UCS-1 (psi)**	**UCS-2 (psi)**	**UCS-3 (psi)**	**UCS-Avg (psi)**	**UCS-St.Dev. (psi)**
28	141.2	118.4	138.3	132.6	12.4	123.6	129.5	112.5	121.9	8.6
24	108.9	114.0	135.3	119.4	14.0	132.4	119.2	117.7	123.1	8.1
17	99.3	94.9	90.5	94.9	4.4	114.0	109.6	96.4	106.7	9.2
14	106.7	105.2	87.5	99.8	10.6	100.0	91.2	83.1	91.5	8.5
7	98.6	90.5	86.8	91.9	6.0	96.4	89.7	94.9	93.7	3.5
6	88.3	78.7	79.4	82.1	5.3	83.1	80.9	69.9	78.0	7.1
3	64.7	70.6	66.9	67.4	3.0	86.1	77.2	74.3	79.2	6.1
1	69.9	61.1	62.5	64.5	4.7	64.0	61.8	49.3	58.4	7.9
Untreated	10.3	13.2	11.8	11.8	1.5					

Table B2: Summary of UCS Data for Site #2, Penn Ave.

	Room Temperature Curing					40°F Curing				
Curing Time (days)	**UCS-1 (psi)**	**UCS-2 (psi)**	**UCS-3 (psi)**	**UCS-Avg (psi)**	**UCS-St.Dev. (psi)**	**UCS-1 (psi)**	**UCS-2 (psi)**	**UCS-3 (psi)**	**UCS-Avg (psi)**	**UCS-St.Dev. (psi)**
28	225.8	240.5	192.7	219.7	24.5	155.9	135.3	128.7	140.0	14.2
26	222.1	203.0	198.6	207.9	12.5	154.5	138.3	130.2	141.0	12.4
14	161.1	179.5	133.9	158.1	22.9	137.5	152.3	151.5	147.1	8.3
10	200.1	218.5	178.0	198.8	20.3	130.2	153.7	131.7	138.5	13.2
7	192.7	146.4	149.3	162.8	25.9	144.2	136.8	128.0	136.3	8.1
3	142.0	147.8	135.3	141.7	6.3	143.4	125.0	130.2	132.9	9.5
1	121.4	115.5	119.9	118.9	3.1	104.4	105.9	97.8	102.7	4.3
Untreated	47.8	45.6	46.3	46.6	1.1					

Table B3: Summary of UCS Data for Site #3, US 177

	Room Temperature Curing					40^{o}F Curing				
Curing Time (days)	UCS-1 (psi)	UCS-2 (psi)	UCS-3 (psi)	UCS-Avg (psi)	UCS-St.Dev. (psi)	UCS-1 (psi)	UCS-2 (psi)	UCS-3 (psi)	UCS-Avg (psi)	UCS-St.Dev. (psi)
28	230.2	224.3	195.7	216.7	18.5	206.0	166.2	172.1	181.4	21.4
15	108.9	114.0	135.3	119.4	14.0	132.4	119.2	117.7	123.1	8.1
14	158.9	183.9	172.1	171.6	12.5	142.7	154.5	119.2	138.8	18.0
7	117.0	108.1	109.6	111.6	4.7	92.7	103.0	83.1	92.9	9.9
6	167.0	188.3	158.1	171.1	15.5	153.0	139.8	132.4	141.7	10.4
4	155.9	125.0	149.3	143.4	16.3	133.9	120.6	146.4	133.6	12.9
3	163.3	164.0	172.9	166.7	5.3	105.9	124.3	105.2	111.8	10.8
2	143.4	147.1	130.2	140.2	8.9	103.0	94.2	100.8	99.3	4.6
1	94.9	94.9	72.1	87.3	13.2	82.4	92.7	83.9	86.3	5.6
Untreated	45.6	41.2	39.0	41.9	0.0					

Table B4: Summary of UCS Data for Site #4, SH 7

	Room Temperature Curing					40^{o}F Curing				
Curing Time (days)	UCS-1 (psi)	UCS-2 (psi)	UCS-3 (psi)	UCS-Avg (psi)	UCS-St.Dev. (psi)	UCS-1 (psi)	UCS-2 (psi)	UCS-3 (psi)	UCS-Avg (psi)	UCS-St.Dev. (psi)
28	212.6	164.8	175.1	184.1	25.2	189.0	179.5	168.4	179.0	10.3
14	211.1	226.6	207.4	215.0	10.1	188.3	183.9	163.3	178.5	13.3
7	183.9	208.9	189.0	193.9	13.2	179.5	180.2	192.0	183.9	7.0
3	144.9	180.2	168.4	164.5	18.0	183.2	155.9	139.0	159.4	22.3
1	100.0	114.0	101.5	105.2	7.7	110.3	114.7	93.4	106.2	11.3
Untreated	48.5	47.1	40.5	45.4	4.3					

Table B5: Summary of UCS Data for Site #5, US 81

	Room Temperature Curing					**40^{o}F Curing**				
Curing Time (days)	**UCS-1 (psi)**	**UCS-2 (psi)**	**UCS-3 (psi)**	**UCS-Avg (psi)**	**UCS-St.Dev. (psi)**	**UCS-1 (psi)**	**UCS-2 (psi)**	**UCS-3 (psi)**	**UCS-Avg (psi)**	**UCS-St.Dev. (psi)**
28	128.0	114.0	96.4	112.8	15.9	101.5	99.3	85.3	95.4	8.8
16	109.6	99.3	91.9	100.3	8.9	111.8	85.3	85.3	94.2	15.3
14	117.7	89.0	89.0	98.6	16.6	83.9	86.1	82.4	84.1	1.9
8	106.7	69.9	80.2	85.6	19.0	104.4	83.1	80.2	89.2	13.2
7	102.2	90.5	81.6	91.5	10.3	97.8	83.1	77.2	86.1	10.6
4	101.5	75.0	79.4	85.3	14.2	90.5	83.1	82.4	85.3	4.5
3	91.9	87.5	72.1	83.9	10.4	109.6	78.0	72.1	86.6	20.2
2	91.9	80.2	75.8	82.6	8.4	96.4	77.2	66.9	80.2	14.9
1	100.8	77.2	78.0	85.3	13.4	81.6	70.6	58.1	70.1	11.8
Untreated	13.2	11.0	11.8	12.0	0.0					

APPENDIX C: RESILIENT MODULUS TEST RESULTS

Table C1: Summary of M_R Results (Treated) for Test Sites 1, 2 and 3

Site #	Site Name	Additive/ Amount (%)	w (%)	γ_d (pcf)	Curing Time (day)	M_R s3=4,sd=10 (psi)
1	US281	CFA/15.38	10.8	114.0	1	50411
1	US281	CFA/15.38	10.8	114.1	1	54550
1	US281	CFA/15.38	10.5	114.1	3	70450
1	US281	CFA/15.38	11.0	113.6	3	63104
1	US281	CFA/15.38	10.4	115.3	7	65245
1	US281	CFA/15.38	10.9	114.5	7	54394
1	US281	CFA/15.38	10.2	115.0	14	70327
1	US281	CFA/15.38	10.5	116.3	14	75178
1	US281	CFA/15.38	10.3	113.8	28	78051
1	US281	CFA/15.38	10.4	115.5	28	85753
2	Penn Ave.	CFA/13.39	15.3	106.0	1	46990
2	Penn Ave.	CFA/13.39	15.3	106.0	1	47150
2	Penn Ave.	CFA/13.39	16.1	104.5	3	51369
2	Penn Ave.	CFA/13.39	16.1	104.4	3	39338
2	Penn Ave.	CFA/13.39	15.9	104.5	7	68948
2	Penn Ave.	CFA/13.39	16.0	103.7	7	45649
2	Penn Ave.	CFA/13.39	15.8	105.1	14	61720
2	Penn Ave.	CFA/13.39	15.7	104.8	14	66183
2	Penn Ave.	CFA/13.39	14.6	105.8	28	42416
2	Penn Ave.	CFA/13.39	15.4	107.6	28	62234
3	US177	Lime/2.34	24.6	92.9	1	31506
3	US177	Lime/2.34	24.4	93.5	1	31667
3	US177	Lime/2.34	22.8	93.9	3	50005
3	US177	Lime/2.34	21.7	94.5	3	42600
3	US177	Lime/2.34	21.4	96.6	7	45878
3	US177	Lime/2.34	22.1	104.9	7	44095
3	US177	Lime/2.34	21.7	93.9	14	58672
3	US177	Lime/2.34	23.7	93.9	14	59128
3	US177	Lime/2.34	21.9	94.7	28	46084
3	US177	Lime/2.34	22.2	94.6	28	53559

Table C2: Summary of M_R Results (Treated) for Test Sites 4 and 5

Site #	Site Name	Additive/ Amount (%)	w (%)	γ_d (pcf)	Curing Time (day)	M_R s3=4,sd=10 (psi)
4	SH7	CFA/12, Lime/4	12.6	112.0	1	49984
4	SH7	CFA/12, Lime/4	12.9	111.4	1	47918
4	SH7	CFA/12, Lime/4	12.7	111.9	3	55351
4	SH7	CFA/12, Lime/4	12.5	111.5	3	56836
4	SH7	CFA/12, Lime/4	12.3	113.7	7	51918
4	SH7	CFA/12, Lime/4	12.5	112.3	7	80081
4	SH7	CFA/12, Lime/4	12.8	113.1	14	58892
4	SH7	CFA/12, Lime/4	13.1	112.8	14	62587
4	SH7	CFA/12, Lime/4	13.5	110.1	28	53637
4	SH7	CFA/12, Lime/4	12.9	111.9	28	67553
5	US81	CFA/12.15	9.0	118.5	1	41002
5	US81	CFA/12.15	8.8	119.6	1	38511
5	US81	CFA/12.15	8.6	118.7	3	49316
5	US81	CFA/12.15	9.4	117.4	3	39133
5	US81	CFA/12.15	8.4	118.7	7	57127
5	US81	CFA/12.15	9.3	118.0	7	42966
5	US81	CFA/12.15	8.5	117.4	14	64381
5	US81	CFA/12.15	8.7	119.7	14	31338
5	US81	CFA/12.15	8.8	118.6	28	59173
5	US81	CFA/12.15	8.3	117.8	28	58904

Table C3: Summary of M_R Results for Untreated Samples

Site #	Site Name	OMC (%)	γ_{dmax} (pcf)	w (%)	γ_d (pcf)	M_R s3=4,sd=10 (psi)
1	US281	12.0	116.5	11.23	113.21	19728
1	US281	12.0	116.5	11.45	112.98	18568
2	Penn Ave	17.5	108.8	16.17	103.54	30246
2	Penn Ave	17.5	108.8	15.80	103.30	27540
3	US177	21.5	98.5	22.15	86.93	13356
3	US177	21.5	98.5	21.82	92.81	13435
4	SH7	13.5	112.7	11.36	112.42	34885
4	SH7	13.5	112.7	11.51	112.29	34442
5	US81	10.8	119.0	10.94	115.12	13878
5	US81	10.8	119.0	10.90	112.29	19898

APPENDIX D: TABULATED SUMMARY OF FIELD TEST RESULTS

Table D1: Summary of Field Test Data for Site #1, US 281

Time (days)	Test Point	Average PFWD E_{VD} (psi)	Average DCP 1/DCI (mm/blow)	Average PANDA Tip Resistance (psi)
0	1a	1059	0.043	702
6	1	17082	0.118	1408
17	1	16154	0.105	1176
24	1	9745	0.087	893
0	2a	3538	0.049	945
6	2	22042	0.167	1987
17	2	12311	0.155	1510
24	2	10267	0.090	1098
0	3a	3857	0.068	1046
6	3	12355	0.132	1526
17	3	11615	0.136	2272
24	3	5394	0.103	1483

Table D2: Summary of Field Test Data for Site #2, Penn Ave.

Time (days)	Test Point	Average PFWD E_{VD} (psi)	Average DCP 1/DCI (blows/mm)	Average PANDA Tip Resistance (psi)
0	1	14186	0.119	394
1	1	12110	0.047	681
3	1	8975	0.034	906
10	1	9175	0.048	820
14	1	8589	0.045	668
26	1	5926	0.077	803
0	2	7544	0.147	316
1	2	4294	0.053	1054
3	2	8703	0.034	830
10	2	10135	0.046	714
14	2	7816	0.064	1144
26	2	7672	0.074	923
0	3	9419	0.130	439
1	3	2662	0.020	451
3	3	5797	0.020	540
10	3	11380	0.058	855
14	3	8946	0.059	637
26	3	7830	0.080	1377

Table D3: Summary of Field Test Data for Site #3, US 177

Time (days)	Test Point	Average PFWD E_{VD} (psi)	Average DCP 1/DCI (blows/mm)	Average PANDA Tip Resistance (psi)
0	1	4953	0.030	338
2	1	6771	0.050	624
4	1	10664	0.082	1254
6	1	9505	0.086	1739
15	1	11065	0.076	989
0	2	3693	0.024	465
2	2	8259	0.049	553
4	2	12940	0.065	1137
6	2	8131	0.075	1386
15	2	8918	0.037	899
0	3	6370	0.039	661
2	3	9118	0.050	973
4	3	6069	0.063	1218
6	3	6685	0.063	1049
15	3	8345	0.055	973

Table D4: Summary of Field Test Data for Site #4, SH 7

Time (days)	Test Point	Average PFWD E_{VD} (psi)	Average DCP 1/DCI (blows/mm)	Average PANDA Tip Resistance (psi)
0	1	9505	0.122	1119
1	1	8474	0.075	935
3	1	11838	0.137	1861
0	2	4867	0.054	806
1	2	7143	0.093	1169
3	2	8546	0.148	1546
0	3	4022	0.051	527
1	3	7529	0.081	873
3	3	10135	0.159	2010

Table D5: Summary of Field Test Data for Site #5, US 81

Time (days)	Test Point	Average PFWD E_{VD} (psi)	Average DCP 1/DCI (blows/mm)	Average PANDA Tip Resistance (psi)
0	1	2720	0.112	2475
2	1	2233	0.058	989
4	1	6356	0.097	1712
8	1	8517	0.116	2010
16	1	12582	0.157	2602
0	2	973	0.048	1002
2	2	601	0.048	1122
4	2	1975	0.072	1358
8	2	3593	0.114	2212
16	2	8517	0.176	3009
0	3	1016	0.044	949
2	3	544	0.031	646
4	3	701	0.043	825
8	3	730	0.062	1167
16	3	1188	0.062	1085

APPENDIX E: TABULATED SUMMARY OF FIELD AND LABORATORY TEST RESULTS AT DIFFERENT CURING TIMES

Table E1: Summary of Lab and Field Data for Five Test Sites

Site	M_R lab time (days)	Avg. M_R (psi)	UCS lab time (days)	Avg. UCS (psi)	Field time (days)	Avg. PFWD E_{vd} (psi)	Avg. DCP (1/DCI) (blows/mm)	Avg. PANDA q_d (psi)
US 281	0	19148	0	9	0	2818	0.053	898
US 281	7	59820	6	82	6	17160	0.139	1640
US 281	14	72753	17	95	17	13360	0.132	1653
US 281	28	81902	24	119	24	8469	0.093	1158
Penn Ave.	0	28893	0	47	0	10383	0.132	383
Penn Ave.	1	47070	1	119	1	6356	0.040	729
Penn Ave.	3	45354	3	142	3	7825	0.029	759
Penn Ave.	7	57299	10	199	10	10230	0.051	796
Penn Ave.	14	63952	14	158	14	8450	0.056	816
Penn Ave.	28	52325	26	208	26	7143	0.077	1034
US 177	0	13395	0	42	0	5005	0.031	488
US 177	1	31587	2	140	2	8049	0.050	717
US 177	3	46303	4	143	4	9891	0.070	1203
US 177	7	44987	6	171	6	8107	0.075	1391
US 177	14	58900	15	119	15	9443	0.056	954
SH 7	0	34664	0	45	0	6131	0.076	817
SH 7	1	48951	1	105	1	7715	0.083	992
SH 7	3	56094	3	164	3	10173	0.148	1806
US 81	0	16888	0	12	0	1570	0.068	1475
US 81	1	39757	2	83	2	1126	0.046	919
US 81	3	44225	4	85	4	3011	0.071	1298
US 81	7	50047	8	86	8	4280	0.097	1796
US 81	14	47860	16	100	16	7429	0.131	2232

Table E2: Summary of Lab and Field Data for Five Test Sites from Previous Study (Snethen et al. 2008)

Site	M_R lab time (days)	Avg. M_R (psi)	UCS lab time (days)	Avg. UCS (psi)	Field time (days)	Avg. PFWD E_{vd} (psi)	Avg. DCP (1/DCI) (blows/mm)	Avg. PANDA q_d (psi)
Enid N	0	16652	0	23	0	4148	0.034	305
Enid N	1	121915	1	81	1	4627	0.079	732
Enid N	3	101005	3	147	3	4583	0.101	911
Enid N	7	268972	7	211	9	5062	0.099	779
Enid S	0	13212	0	6	0	6860	0.069	692
Enid S	1	28496	1	18	1	7078	0.208	1524
Enid S	3	58322	3	33	3	11269	0.345	2582
Enid S	7	138361	7	76	7	21566	0.294	2624
US 62	0	12319	0	15	0	7803	0.147	1160
US 62	1	35990	1	42	1	9398	0.095	689
US 62	3	60550	3	42	4	7542	0.192	1085
Perry	0	17741	0	31	0	5091	0.023	226
Perry	3	162252	3	78	3	5714	0.044	470
Perry	7	139826	7	98	6	6976	0.052	442
Perry	14	194200	14	100	10	10747	0.085	624
Payne	0	6314	0	28	0	6657	0.058	482
Payne	3	38020	3	69	4	9688	0.120	2191
Payne	7	44524	7	89	7	8992	0.132	2434
Payne	14	55399	14	100	12	8963	0.169	3096