PERFORMANCE EVALUATION OF SUBGRADE STABILIZATION WITH RECYCLED MATERIALS

MDOT RESEARCH PROJECT NO. OR14-009

MDOT CONTRACT NO. 2013-0065

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February 29, 2016

1. Report No. RC-1635	2. Government Accession No. N/A	3. MDOT Project Manager Mark Grazioli, P.E.
4. Title and Subtitle PERFORMANCE EVALUATION OF SUBGRADE STABILIZATION WITH RECYCLED MATERIALS		5. Report Date February 29, 2016
		6. Performing Organization Code N/A
7. Author(s) N. Bandara, E. Jensen, T. Binoy		8. Performing Org. Report No. N/A
9. Performing Organization Name and Address Lawrence Technological University 21000 West Ten Mile Road Southfield, Michigan 48301		10. Work Unit No. (TRAIS) N/A
		11. Contract No. 2013-0065
		11(a). Authorization No. Z3
12. Sponsoring Agency Name and Address Michigan Department of Transportation Research Administration 8885 Ricks Rd. P.O. Box 30049 Lansing MI 48909		13. Type of Report & Period Covered Final Report Period Covered: 10/01/2013 – 02/29/2016
		14. Sponsoring Agency Code N/A
15. Supplementary Notes		

16. Abstract

Due to rising costs of good quality acceptable materials for remove/replace options and traditional subgrade stabilization materials, MDOT is in need to identify potential recycled materials to treat unacceptable subgrade soils. Use of recycled materials may not only provide less costly alternatives for subgrade stabilization, their use may also alleviate landfill disposal challenges.
This research study is aimed at identifying short-term and long-term advantages and disadvantages associated with subgrade stabilization using recycled materials such as Cement Kiln Dust (CKD), Lime Kiln Dust (LKD), flyash, concrete fines and mixtures of LKD and FA. An extensive laboratory testing program was conducted to determine suitability of the above recycled stabilizers for subgrade stabilization for common problematic soils found in Michigan. The laboratory investigative program involved determining the basic soil properties, developing mix designs to select proper stabilizer percentage for each soil type, CBR testing to determine pavement design parameters, and laboratory freeze/thaw testing to determine durability of stabilized subgrade sections. A limited field investigation was performed to assess insitu performance of stabilized subgrades. Based on the findings of both investigations, stabilizers were selected for long-term subgrade stabilization for different soil types and their associated pavement design inputs were determined. A design matrix with cost considerations was also developed to aid the selection of subgrade treatment options.

17. Key Words	18. Distribution Statement
Subgrade, stabilization, recycled, materials, pavement design	No restrictions. This document is available to the public through the Michigan Department of Transportation.

19. Security Classification - report	20. Security Classification – page	21. No. of Pages	22. Price
Unclassified	Unclassified	300	N/A

ACKNOWLEDGEMENTS

This project was funded by the Michigan Department of Transportation (MDOT). The authors would like to acknowledge the support and effort of Mr. Mark Grazioli, MDOT Project Manager for this project, for his continuous assistance and support during the project. The authors also wish to acknowledge MDOT Research Advisory Panel (RAP) members – Mr. Andre Clover, Mr. James Siler, Mr. Mark Melchiori Jr., Mr. Gregory Bills, Mr. David Gauthier, and Mr. Andrew Bennett for their continuous support. The authors also wish to acknowledge MDOT Metro Region soils field crew for their support during field exploration activities in Detroit Metro Region.

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EXECUTIVE SUMMARY

At Michigan Department Transportation (MDOT) a standard practice in treating unsuitable subgrade is to remove and replace (undercutting) with acceptable materials. In a few MDOT projects, soil stabilization techniques were utilized to facilitate construction instead of undercutting. The aim of this research project is to understand the long term and short term performance benefits and potential risks of using subgrades stabilized with recycled materials. The selection of potential recycled materials for the project's laboratory investigation was based on their availability in large quantities in Michigan. The selected recycled materials for subgrade stabilization include Cement Kiln Dust (CKD), Lime Kiln Dust (LKD), Fly Ash (FA) and Concrete Fines (CF). MDOT supplied the researchers with three (3) types of soils considered most commonly problematic in Michigan. If these soils were encountered as subgrade soils, generally they require some type of treatment before constructing upper pavement layers.

The laboratory investigation included determination of (1) basic soil properties, (2) mix design with different recycled stabilizers with selected soil types, (3) California Bearing Ratio (CBR) testing to determine pavement design inputs, and (4) freeze/thaw testing to identify durability properties of stabilized subgrade materials.

Laboratory results showed CKD and mixtures of LKD+FA will provide long-term stabilization for all three soil types at different stabilizer percentages. FA and LKD only worked for some soil types as short-term modifier to construct upper pavement layers. Concrete Fines (CF) were ineffective as either a stabilizer or a short-term modifier for all three soil types. Using the laboratory data from the suitable stabilized subgrades, pavement design inputs were developed from a limited analytical investigation. The input parameters were determined for the stabilized layer modulus values for mechanistic-empirical pavement designs and structural layer coefficients for 1993 AASHTO pavement designs.

A limited field investigation program was performed on selected stabilized sites in Michigan and Ohio. The investigation included Dynamic Cone Penetrometer (DCP) and Falling Weight Deflectometer (FWD) testing. The result of the field investigation yielded pavement stabilized layer modulus values and layer coefficients for pavement design.

A cost evaluation was performed to determine the breakeven point between the options of subgrade stabilization and removal/replacement. The breakeven point is given in terms of the two primary variables; percentage of project area and type of recycled material. These guidelines will help MDOT engineers to select proper sites for treatment and most effective treatment option to suit encountered soil types.

CHAPTER 1: INTRODUCTION

Most often pavement structures in Michigan are constructed on silty and clayey subgrades, especially in Southeast Michigan. With the varying moisture conditions during spring and summer, sometimes these subgrades become soft and need some type of treatment before constructing upper pavement layers. According to the research report published by Kentucky Transportation Center (Hopkins et al, 2002), pavement construction problems can be classified into the following five categories:

1. Failure of weak soil subgrades under construction traffic loading
2. Failure of granular base courses under construction traffic loading
3. Failure of partially completed pavement/base materials under construction traffic loadings
4. Premature failure of pavement shortly after construction
5. Difficulties in achieving proper compaction of granular base and pavement materials due to inadequate bearing strength of the soil subgrade

Figure 1.1: Failure of Weak Soil Subgrade under Construction Traffic Loading

Neighboring states such as Indiana, Ohio, Wisconsin, and Minnesota frequently use soil stabilization techniques to treat unacceptable subgrade soils. Due to the rising costs of materials utilized in traditional treatment techniques, MDOT needs to identify potential recycled materials to treat unacceptable subgrade soils. Recycled materials not only provide a less costly alternative for subgrade stabilization, they also alleviate landfill problems.

Most of the previous research studies related to subgrade stabilization are limited to quantifying immediate benefits through construction facilitation. However, there is a need to identify the long-term benefits and/or risks of subgrade stabilization. With satisfactory long-term benefits, subgrade stabilization can be potentially used for optimizing pavement designs that will result in cost-effective pavement sections. Additionally, long-term risks associated with subgrade stabilization such as heaving and/or cracking of subgrade can be proactively addressed by remedial actions or limiting usage of those stabilizing materials.

This research study identifies short-term and long-term advantages and disadvantages of using recycled materials such as Cement Kiln Dust (CKD), Lime Kiln Dust (LKD), fly ash, concrete fines and other subgrade stabilization materials.

Subgrade stabilization materials can be divided into two categories; stabilizers and modifiers. Subgrade modifiers generally reduce the plasticity of soil and provide a short-term strength improvement. The short-term strength improvement occur shortly after mixing and can be used for construction facilitation. On the other hand, subgrade stabilizers, provide a long-term soil modification process through pozzolanic or cementitious reactions.

1.1 Research Approach

The following were identified as the primary objectives of this project:

1. Review and prepare a summary of existing research related to the application of recycled materials used for subgrade soil stabilization. Priority was on previous/current Michigan subgrade stabilization research and that in other states with similar climate. This included short-term and long-term performance studies, construction techniques, construction quality control/assurance methods, risks, environmental issues, etc.
2. Develop a list of potential recycled materials for subgrade stabilization including, but not limited to: crushed concrete fines (CF), Cement Kiln Dust (CKD), Lime Kiln Dust (LKD), fly ash (FA) or a combination.
3. Conduct a laboratory study to determine mix design proportions, strength, stiffness, expansion, pH properties, and to measure durability during freezing and thawing cycles.
4. Review long-term performance of stabilized subgrade considering climate conditions in Michigan.
5. Develop a decision-making matrix for soil stabilization evaluated in this study that will address soil conditions, cost, and short-term/long-term performance of different recycled materials used for stabilization.
6. Develop guidelines to include stabilized subgrade stiffness properties into pavement design using both 1993 AASHTO Pavement Design Guide and Mechanistic-Empirical (ME) Pavement Design Guide as references.

In order to address these objectives, six tasks were designed:

1. Conduct a comprehensive literature review and interviews.
2. Identify potential recycled materials.
3. Establish mix designs.
    a. Mix proportions and analyze the properties
    b. Test freeze/thaw durability of select mix proportions
4. Review long-term performance of stabilized sections.
    a. Identify pavement sections with stabilized subgrades in Michigan and neighboring states.

    b. Collect data.
        i. Mix designs
        ii. Construction details
        iii. Pavement performance data
    c. Collect field data.
        i. Falling Weight Deflectometer (FWD)
        ii. Coring
        iii. Dynamic Cone Penetrometer (DCP)
        iv. Visual surveys
    d. Understand the long-term performance of different stabilizing agents and mix proportions for different soil types.
5. Develop a Decision Matrix.
6. Develop guidelines to incorporate stabilized subgrade stiffness into pavement design.

CHAPTER 2: LITERATURE REVIEW

A comprehensive literature review was conducted to fully incorporate previous research studies into this project. The summary of relevant literature is given below.

2.1 History of Subgrade Stabilization

The history of subgrade stabilization dates back to the 1960's when Dempsey and Thompson (1968) performed several studies aimed at analyzing the properties and behavior of lime for soil stabilization. These studies looked at the durability properties of lime-soil mixtures, autogenesis healing of lime-soil mixtures, and lime reactivity of Illinois soils (Thompson, 1966); (Dempsey and Thompson, 1968). In the 1970's, Thompson (1970) developed a technical report outlining the state of the art developments in soil stabilization for pavement systems.

More recent work related to soil stabilization for pavement applications includes several studies performed by Little (2008) for the National Lime Association. He developed a Mixture Design and Testing Protocol (MDTP) for lime-stabilized soils. The protocol used a systematic approach of soil assessment for lime stabilization, mixture design, and testing methods (National Lime Association, 2006). This mix design procedure was the first systematic approach developed for soil stabilization using lime for pavement application. Little (2000) also performed an evaluation of structural properties of lime-stabilized soils and aggregate. An example application of previously developed MDTP to evaluate engineering properties of lime-treated subgrades for mechanistic pavement design and analysis was also presented by Little (2001).

Most recently, a study performed by Mallela (2004) provided the details on how to incorporate lime-stabilized bases in mechanistic-empirical pavement design. In general, most laboratory and field-based studies have proved that careful selection of materials, mixture designs, and proper construction methods assure improved pavement performance in lime-stabilized soil subgrades.

2.2 Subgrade Stabilization with Recycled Materials

A number of past studies on CKD explored whether or not it is a hazardous material (PCA, 1992 and EPA, 1995). Several studies have been performed on mixture design for soil/CKD mixtures to modify or stabilize pavement subgrade soils. These studies concluded that the same mixture design procedures developed for lime/fly ash can be used for mixture design for CKD/soil mixtures. Performance of CKD as a pavement subgrade stabilizer has been studied by several researchers. Laboratory performance was investigated by Collins and Emery (1983), where 33 CKDs and 12 LKDs were tested for engineering properties (compressive strength, durability and volume stability) and compared with conventional lime/fly ash/aggregate mixtures. This study concluded that a higher percentage of the LKD is required compared to hydrated limes to achieve similar performance.

Zaman et al. (1992) investigated the effect of freezing/thawing and wetting/drying cycles on the durability of CKD-stabilized clay samples. The test results showed significant strength decrease due to freezing/thawing and wetting/drying cycles (Zaman et al. 1992). As summarized by Button (2003), multiple researchers reported mixed field results taken from several states using soil/CKD stabilization techniques.

A field and laboratory evaluation of soil stabilization using CKD was conducted by Miller and Zaman (2000). Using a test section constructed in Ada, Oklahoma, three types of CKD were compared with quicklime. The subgrade was treated with 4% (by weight) quicklime and 15% CKD. After curing and submerging the samples in water, field-mixed samples were collected for Unconfined Compressive Strength (UCS) testing. All UCS samples were prepared using a calibrated Harvard Miniature Compaction procedure. Higher strengths were observed in all cured samples while strength loss was observed in all submerged samples. Field tests using DCP and FWD were conducted after 28 days and 56 days following compaction of the treated subgrade. Similar results were observed in both DCP and FWD data showing a competent subgrade due to stabilization. Laboratory testing of soils mixed with quicklime and three CKD types were conducted to establish the effect of curing time on strength and the effect of freeze/thaw and wet/dry cycles on strength. All laboratory samples were prepared by mixing 4% quicklime and 15% CKD using a calibrated Harvard Miniature Compaction method. Strength gain over time was tested by comparing UCS results at 3, 7, 14, 28 and 90 days after compaction. CKD samples show a strength gain during the first seven to 14 days followed by little change in strength. Durability testing using wet/dry cycles showed drastic effects on stabilized clayey soils. All clayey samples stabilized with quicklime and CKD fell apart before three wet/dry cycles. However, sandy soils stabilized with CKD showed strength gain during 12 wet/dry cycles.

In Michigan, MDOT constructed a test section with a CKD-stabilized subgrade and compared it with lime and lime with fly ash (Class F) stabilized subgrades in a project completed in 2008. It was concluded that a substantial increase in subgrade strength was made possible through the stabilization with lime, lime with fly ash, and CKD (Bandara, 2009). The following table shows calculated CBR values from DCP testing performed on stabilized subgrades.

Table 2.1: Average Calculated CBR Values from DCP Test Data (Bandara et al, 2009)

Test Area	Stabilized Thickness based on DCP (in)	Stabilized Subgrade CBR (%)	Insitu Soil (CBR)	Strength Gain (%)
Mostly clay (5% lime stabilization; 12 inches)	14.6	15.7	2.2	615
Mostly clay (5% lime stabilization; for 14 inches)	19.8	15.4	2.9	438
Mostly clay (5% lime stabilization; 18 inches)	17.7	18.7	1.0	1838
Sand over clay (4% lime and 8% fly ash stabilization; 12 inches)	12.9	15.5	5.2	197
Clay (8% CKD stabilization; 12 inches)	13.9	29.6	2.3	1195
Moist clay (8% CKD stabilization; 12 inches)	12.0	8.0	1.3	513
Retest on moist areas after installing underdrains	12.0	15.6	1.6	789
Sand over clay (8% CKD stabilization; 12 inches)	17.0	34.7	3.4	915
Moist sand over clay (8% CKD stabilization; 12 inches)	16.2	16.9	3.3	412

In 2001, the Illinois Department of Transportation (Heckel, 2001) conducted a laboratory and field performance study to evaluate alternate materials for subgrade modification of unstable [California Bearing Ratio (CBR) <6] subgrade soils. The alternative materials included by-product hydrated lime and Class C fly ash. Three experimental projects were constructed and performances of these sections were compared to a control section treated with LKD or dense graded aggregate base. The results showed that the application of alternate materials was successful during construction and no measurable differences in performance were noticed during the three-year monitoring period.

The Wisconsin Department of Transportation (WISDOT) sponsored a research project to evaluate the short-term and long-term performance of Class C fly ash stabilized subgrades (Edil et al, 2010). Three projects constructed by WISDOT with fly ash stabilization were evaluated during construction and one project was monitored for eight years. CBR, resilient modulus (M_r), and unconfined compression (q_u) tests were conducted on the in situ soils and fly ash stabilized soils. Properties of the laboratory mix fly ash contents and corresponding laboratory test results are shown in Table 2.2. Furthermore, field stiffness testing was conducted on the stabilized and in situ soils using Soil Stiffness Gauge (SSG), dynamic DCP, and FWD. To evaluate the quality of water percolating from the stabilized layers, pan lysimeters were installed beneath the roadway, beneath fly ash stabilized soils and beneath a control section having unstabilized soils. A complex relationship between soils types, fly ash content (FA%), water content (w%), CBR, M_r and q_u was established as shown in Table 2.2.

All test sites showed significant improvement in subgrade strength during construction and remained stiff in subsequent rain events. There were marked variations in soil types and hence different fly ash contents and moisture contents were used during construction. As recommended in the study, proper mix designs involving all potential soil types should be completed for fly ash stabilization projects. Based on the FWD testing, it was also concluded that the fly ash stabilized sections did not display any significant reduction in subgrade moduli after a number of freeze/thaw cycles. Distress surveys on test sections provided results comparable to control sections. Percolation test results were inconclusive due to both lysimeters (under the fly ash stabilized sections and control section) showing higher effluent concentrations.

Table 2.2: Properties of Lab Mix Fly Ash Stabilized Subgrade (Edil et al, 2010)

Station Number	Soil Classification		FA (%)	w (%)	$w-w_{opt}$	M_r (MPa)	q_u (kPa)
	USCS	AASHTO					
580+00	CL	A-7-6	12	13	-2	242	450
				15	0	115	510
				18	3	98	240
				22	7	61	350
			15	13	-2	192	570
				15	0	144	360
				18	3	172	570
				22	7	105	440
			18	13	-2	122	510
				15	0	109	470
				18	3	347	990
				22	7	130	590
582+00	SC	A-2-6	12	7	-2	102	430
				9	0	134	360
				12	3	161	480
				16	7	178	650
			15	7	-2	163	660
				9	0	183	520
				12	3	303	480
				16	7	160	660
			18	7	-2	366	600
				9	0	253	1160
				12	3	208	1010
				16	7	130	850
614+00	SP-SM	A-2-6	12	8	-2	152	310
				10	0	167	1120
				13	3	153	730
				17	7	207	430
			15	8	-2	111	720
				10	0	164	1330
				13	3	241	1280
				17	7	264	810
			18	8	-2	94	430
				10	0	129	1090
				13	3	195	2390
				17	7	178	1100

A research study conducted for Oklahoma Department of Transportation (ODOT) examined the validity of ODOT standard *OHD L-50: Soil Stabilization Mix Design Procedure* (Cerato and Miller, 2011). *OHD L-50* gives guidelines on stabilizer percentages for different soil types as shown below.

Table 2.3: ODOT Guidelines for Soil Stabilization (Cerato et al, 2011)

Additive	**AASHTO Soil Group**											
	A-1-		**A-2-**							**A-6**	**A-7-**	
	a	**b**	**4**	**5**	**6**	**7**	**A-3**	**A-4**	**A-5**		**5**	**6**
Cement	4	4	4	4	4	4	5	+	+	+		
Fly Ash					12	12	13	14	14	14		
CKD (Pre-calciner)	5	5	5	5	5	5	6	+	+			
CKD (other)	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-calciner plants are identified on the materials division approved list for cement kiln dust.
+Mix design required
*Reduce quantity by 20% when quick lime is used
**Use 6% when liquid limit is greater than 50

A study conducted by Miller and Diaz (2002) examined the influence of delayed compaction of CKD. A clayey sand [Liquid Limit (LL) =29%, Plasticity Index (PI) =13.5%] was mixed with 10% CKD and compacted at various elapsed times after mixing. At each delay, unconfined samples were prepared and tested after 14 days of curing. As shown in the following figure, substantial strength loss was observed after a compaction delay of about two hours. This is a major finding for field mixing requirements.

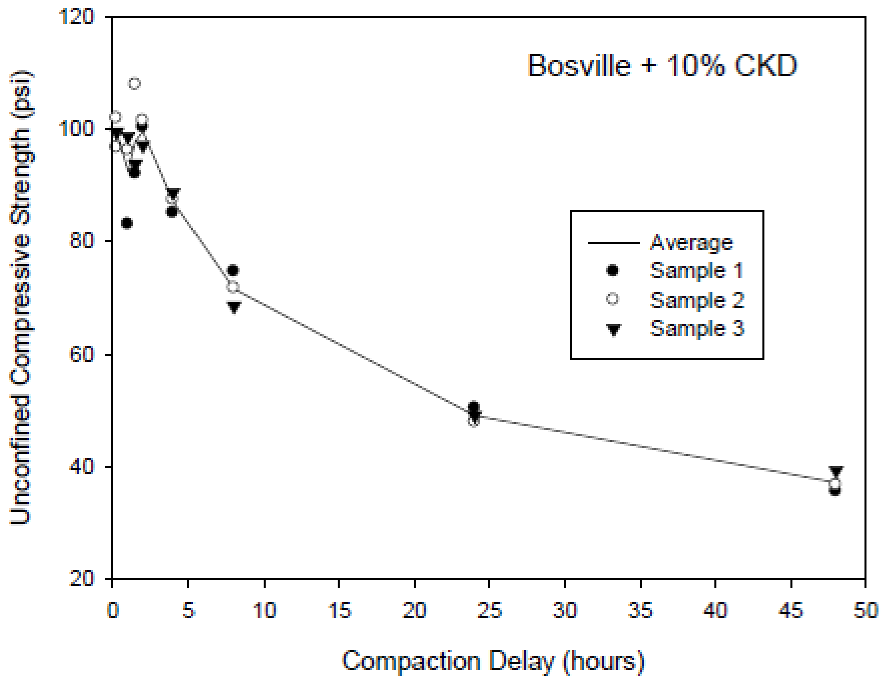


Figure 2.1: Reduction in Unconfined Compression Strength due to Compaction Delay (Cerato et al, 2011)

This study also reviewed different stabilizer percentages used by other researchers. The following table lists the recommended stabilizer percentages based on the soil type.

Table 2.4: Recommended Stabilizer Percentages (Cerato et al, 2011)

Study	Soil Type	CKD Percentage	Fly Ash Percentage
Si and Herrera, 2007	A-6	2% - 10%	N/A
Mohamed, 2002	Non-plastic Silty Sand	6%	N/A
Cokca, 2001	Expansive soil	N/A	20%

During this study, common fine-grained soils (A-4, A-6 or A-7-6) found in Oklahoma were sampled and tested with lime, CKD, and two types of Class C fly ash in varying amounts. The tests included UCS testing of cured samples prepared with a Harvard Miniature Compaction device, Atterberg limit tests of stabilized soils, and shrinkage. Based on the results of the laboratory study, the following additive percentages were recommended in order to achieve recommended 50 psi strength after stabilization.

Table 2.5: Comparison of Existing OHD L-50 Stabilization Recommendation (Cerato et al, 2011)

Additive	AASHTO Soil Classification		
	A-4	A-6	A-7-6
Fly Ash	14*	14 - [$]6%	**9%
CKD	12 - [$]10%	**9%	**9%
Lime		4 - [$]4%	5-[$]3%

*Existing recommendation in OHD L-50. Stabilization, as defined by an increase in strength of 50 psi above soil's raw strength, was not seen in two of the three A-4 soils tested with fly ash in this study. In fact, even when the percentages of fly ash were increased to 15%, the strength of the two soils did not increase.
[$]Stabilizer needed to obtain 50 psi increase in strength above raw soil in this study.
**New addition to this table. No previous recommendations for these soil or stabilization categories were given in OHD L-50.

Only limited research studies related to LKD is available in the literature. However, the following Indiana Department Transportation design guideline included LKD as a chemical modification agent. Only quick-lime and cement are included as chemical stabilizing agents (INDOT, 2008).

Table 2.6: Design Guidelines for Soil Stabilization and Modification (INDOT, 2008)

Additive	Percentage for Stabilization or Modification
Lime or Lime by products	4% to 7%
Cement	4% to 6%
Class C Fly Ash	10% to 16%

2.3 Mix Design Procedures for Subgrade Stabilization

A comprehensive review of available materials, methods, and protocols for mix designs for subgrade and base stabilization was reported in *NCHRP W144: Recommended Practice for Stabilization of Subgrade Soils and Base Materials* (Little and Nair, 2009). Although this report focused on traditional stabilizers: Portland cement, lime, and fly ash; subgrade stabilization with by-products was mentioned in the document. This guide provided protocols for stabilizer selection, verification through laboratory studies, and mix design procedures for commonly used traditional stabilizers. One of the guidelines included in this document was a decision tree for selecting stabilizers for use in subgrade soils. This guideline was developed by Texas Department of Transportation. This decision tree is shown in the following figure.

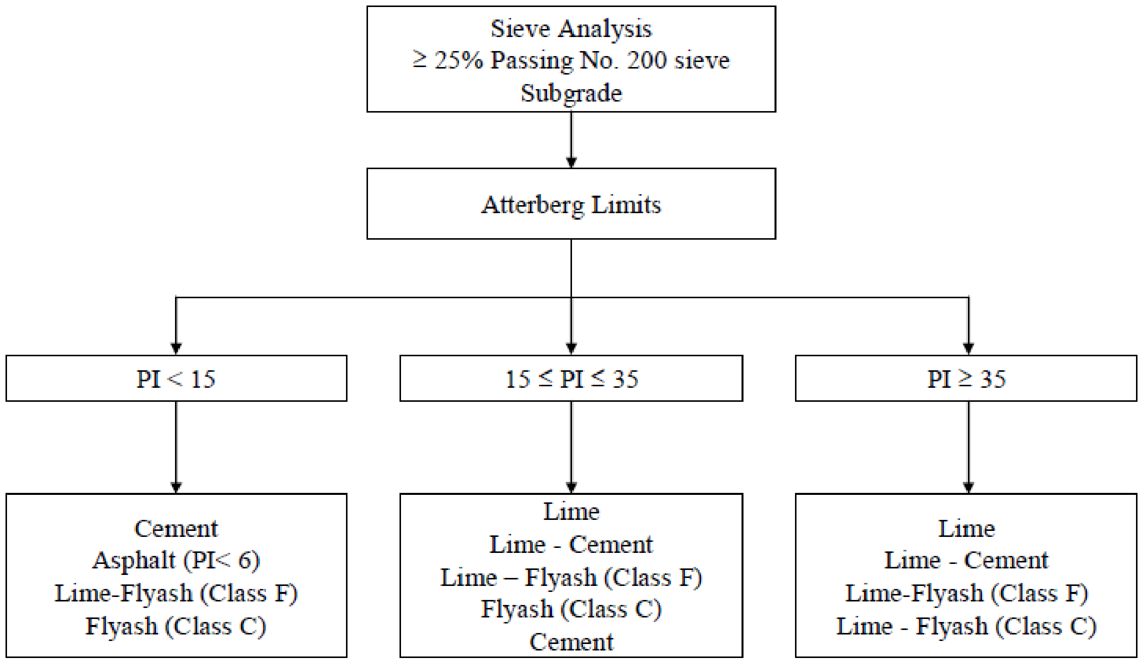


Figure 2.2: Decision Tree for Selecting Stabilizers for Use in Subgrade Soils (Little et al, 2009)

This guide provided procedures for mix design for lime, Portland cement and fly ash as shown below.

2.3.1 Lime Stabilization

The mix design guidelines for the lime stabilization given in *W144* are based on National Lime Association protocol. This method was designed for long-term strength gain and durability of lime-stabilized subgrades.

Soil Evaluation

Soil evaluations consisted of determining the Plasticity Index (PI) and Percent Passing (PP) No. 200 sieve. Soils with a PI of 10 or above and a PP No. 200 sieve minimum of 20% are suitable for stabilization with lime. This protocol also recommended testing for organic content and water soluble sulfate content. If the water soluble sulfate content exceeds 3,000 ppm, a swell test should be performed to evaluate the degree on expansion and implement remedial actions during construction.

Optimum Lime Content

The first step required to determine the optimum lime content for subgrade stabilization is based on the Eades and Grim pH test (*ASTM D 6276*). This test method determines the amount of lime needed to achieve a pH value of 12.45 at 25°C (77°F). The goal of this test is to determine the amount of lime necessary to maintain long-term pozzolonic reactions. However, the mix design guideline recommended this lime content should be validated with strength testing.

Moisture Density Relationships

Lime changes the optimum moisture content and maximum dry density of soils. Therefore, moisture-density tests are important for construction specifications for soil stabilized with lime. These moisture-density tests are conducted on a soil-lime mixture prepared with an amount of lime identified by the Eades and Grim test.

Fabrication and Curing of Samples for Compression Testing

For the compression testing, triplicate samples are prepared using *ASTM D* 5102: *Procedure B* with the lime content determined by the Eades and Grim test. Samples were fabricated between ± 1% of Optimum Moisture Content (OMC). Additional samples having a lime content 1% and 2% higher than the optimum lime content were prepared to verify the optimum lime content by strength testing.

After compaction, the samples were wrapped in a plastic wrap and stored in an air tight plastic bag containing about 10 ml of water and then cured for seven days at 40°C (104°F). This accelerated curing procedure provided sufficient moisture and time for strength gain by pozzolonic reactions with lime and clay minerals. However, it is advisable to cure one set of soil-lime specimens for 28 days.

In preparation for capillary socking process, the specimens were removed from the plastic bags/plastic wraps and re-wrapped with wet absorptive fabrics after the curing procedure. During capillary soaking process, wrapped samples with absorptive fabrics were placed on porous stones. These porous stones were submerged in water with the water level maintaining at top of the porous stones. Capillary soaking should continue until the moisture front move to the top of the sample or until moisture front stop moving.

Unconfined Compressive Strength (UCS) Testing

Following the capillary soak, UCS testing was performed accordance with *ASTM D 5102: Procedure B*. For soil stabilization, the UCS value should meet the requirement listed in the table below.

Table 2.7: Recommended UCS Values for Lime Stabilization (Little et al, 2009)

Anticipated use of Stabilized Layer	Compressive Strength Recommendations for Different Anticipated Conditions			
	Extended Soaking	Cyclic Freeze/Thaw		
	8 Days (psi)	3 cycles (psi)	7 cycles (psi)	10 cycles (psi)
Rigid Pavement	50	50	90	120
Flexible Pavement (>10 in)	60	60	100	130
Flexible Pavement (8 in -10 in)	70	70	100	140
Flexible Pavement (5 in – 8 in)	90	90	130	160

If the mix designs used more than one lime content, the design with the lowest amount of lime satisfying the above requirements should be used as the design lime content. If none of the lime content met the requirements in the above table, either additional lime should be added to the mix design or the design should be considered as subgrade modification and not stabilization.

Volume Change Measurements for Expansive Soils

Samples prepared for UCS testing can be used for volume change measurements. Vertical and circumferential measurements of samples before and after capillary soaking were taken to evaluate the volume change between dry and soaked conditions. Three dimensional expansion of 2% or less were considered acceptable. It should be noted that this test procedure was only applicable to expansive soils.

2.3.2 Cement Stabilization

Cement has been used to stabilize most soil types except those with high organic content, highly plastic clays, or poorly reacting sandy soils. Some of the limitations of cement stabilization include the shorter mixing time before the initial set of cement, usually not more than two hours before compaction. Portland Cement Association (PCA) has published a "Soil-Cement Laboratory Handbook" to aid the mix design procedure when using cement for soil stabilization.

Preliminary Estimate of Cement Content

The PCA Soil-Cement Handbook recommends the following initial cement requirements based on AASHTO soil group.

Table 2.8: Preliminary Cement Requirements for Cement Stabilization (Little et al, 2009)

AASHTO Soil Group	Usual Range in Cement Requirement		Estimated Cement Content, Percent by Weight
	Percent by Volume	Percent by Weight	
A-1-a	5-7	3-5	5
A-1-b	7-9	5-8	6
A-2	7-10	5-9	7
A-3	8-12	7-11	9
A-4	8-12	7-12	10
A-5	8-12	8-13	10
A-6	10-14	9-15	12
A-7	10-14	10-16	13

The above cement requirements were preliminary estimates only and must be verified and modified based on other laboratory testing such as strength testing and durability testing.

Determine the Moisture-Density Relationship

As reported in previous literature, changes in optimum moisture content and maximum dry density were highly variable and not always predictable. Therefore, it was recommended that cement contents specified in the above table should be used for sample preparation for moisture-density relationships. After the required amount of cement was mixed with soil, the blend should be thoroughly mixed until the color of the mixture is uniform.

Sample Preparation for Compressive Strength and Durability Testing

As the primary requirement, the ability to withstand adverse environmental conditions is a PCA criterion for mix design of soil-cement mixtures. Subsequent testing involved measuring weight loss under repeated wet/dry and freeze/thaw cycles. The research work by Thompson and Dempsey (1968) in lime-stabilized soils under freeze/thaw conditions can be used as a criterion in deciding durability of soil-cement mixtures. Thompson’s study suggested that the compressive strength decreases by approximately 8-10 psi for every freeze/thaw cycle. It is recommended to prepare three samples with the following cement contents for strength and durability testing; 1 sample at 2% below initial cement content, 1 sample at initial cement content and 1 sample at 2% above initial cement content.

Unconfined Compressive Strength Testing

Preparation and curing of samples should be performed according to *ASTM D 1633*. This test procedure requires curing of soil-cement samples in a moist room and then immerse them in water for four hours prior to testing. The following minimum seven days UCS values are recommended by the U.S. Army Corps of Engineers.

Table 2.9: Recommended UCS Values for Cement Stabilization (Little et al, 2009)

Type of Pavement	Minimum 7 day UCS (psi)
Flexible	250
Rigid	200

2.3.3 Fly Ash Stabilization

Fly ash is a by-product of coal burning in power plants and an excellent product for soil stabilization. There are two types of fly ash: Class C and Class F. They differ depending upon the amount of available free calcium. Class C refers as self-cementing fly ash and has sufficient free calcium to react with soil in the presence of water (more than 20% lime). On the other hand, Class F fly ash has a low concentration of free calcium and requires an additional agent such as lime or cement to initiate the hardening process during stabilization. Due to the complex process of this stabilization mechanism with fly ash, physical properties of fly ash treated materials should be tested prior to use in soil stabilization.

Class C Fly Ash Mix Design

Currently there are no standard test procedures for mix design of Class C fly ash stabilization. However, two important design considerations should be addressed; time delay in mixing and compaction of fly ash-soil mixtures due to high rate of hydration of Class C fly ash materials and the moisture content at which the maximum strength is achieved. Generally, the optimum moisture content for strength gain is 1% to 8% below optimum moisture content for maximum dry density.

The first step of mix design is to establish moisture-density relationships for each soil type at different fly ash contents. Once the optimum moisture content for the mix is determined, the moisture strength relationship is established by using different moisture levels below optimum to determine the moisture content at which the maximum strength is achieved. Test specimens are cured for seven days at 100°F and then immersed in water for four hours or subjected to capillary soak for 24 hours as observed with lime mix designs.

Class F Fly Ash Mix Design

When Class F fly ash is used for soil stabilization, an activator such as lime or cement (or LKD or CKD) is required to initiate stabilization reactions. The mix design process includes selection of proper fly ash content and determining optimum moisture content and maximum dry density of the fly ash/soil mixture. Generally five different samples with varying fly ash contents, starting from 6% to 20% (by weight), are used and mixes are molded to determine optimum moisture content according to *ASTM C 593*. The dry density of each mix is also determined. To account for materials lost during field mixing, an additional 2% fly ash is added to the sample that has the maximum density and optimum moisture content for the final field mix.

Optimal activator content is determined by trial and error. Typically one part lime to three parts fly ash (1:3 ratio) to one part lime to four parts fly ash (1:4 ratio) is used. If LKD or CKD is used as an activator, higher ratios are required based on the free lime content in the kiln dusts.

Curing and compressive strength testing are conducted similarly to Class C fly ash mixtures. A 7-day minimum compressive strength of 400 psi is considered acceptable for field applications.

2.3.4 Pavement Design Inputs for Stabilized Subgrade Layers

A study conducted by Kentucky Transportation Center (Hopkins et al, 2002) investigated the bearing strength, durability, structural stiffness, economics and performance of pavements with subgrades stabilized with different chemical mixtures. These stabilizing agent included hydrated lime, Portland cement, a combination of hydrated lime and Portland cement and byproducts such as LKD and Atmospheric Fluidized Bed Combustion Ash (AFBC). Fourteen roadways sites containing 20 different treated subgrade sections with different stabilizing agents were evaluated in this project. The age of these projects ranged from eight (8) to 15 years. On these projects over 450 soil borings were performed with in-situ CBR tests. Index tests and resilient modulus tests were performed collected samples. Furthermore, FWD tests were performed to evaluate in-situ pavement characteristics including subgrade moduli values. Based on the in situ CBR tests the following results were developed based on the 85th percentile test values.

Table 2.10: In situ CBR Values at the 85th Percentile Test and Structural Layer Coefficients (Hopking et al, 2002)

Chemical Admixture	In situ CBR at the 85th Percentile	Structural Layer Coefficient
Hydrated Lime	27	0.106
Portland Cement	59	0.127
Hydrated Lime/Portland Cement	32	0.11
Lime Kiln Dust	24	0.10
AFBC	9	0.08
Untreated Soil Subgrade	2	-

A more recent study conducted for Ohio Department of Transportation (Sargand et al, 2014) aimed at developing guidelines for incorporating chemical stabilization of the subgrade in pavement design and construction practices. As part of this project, the researchers conducted a survey of departments of transportation of all states and Canadian provinces. Twenty six states and three states and 3 provinces responded. The survey results indicated the following types of chemicals were used for subgrade stabilization.

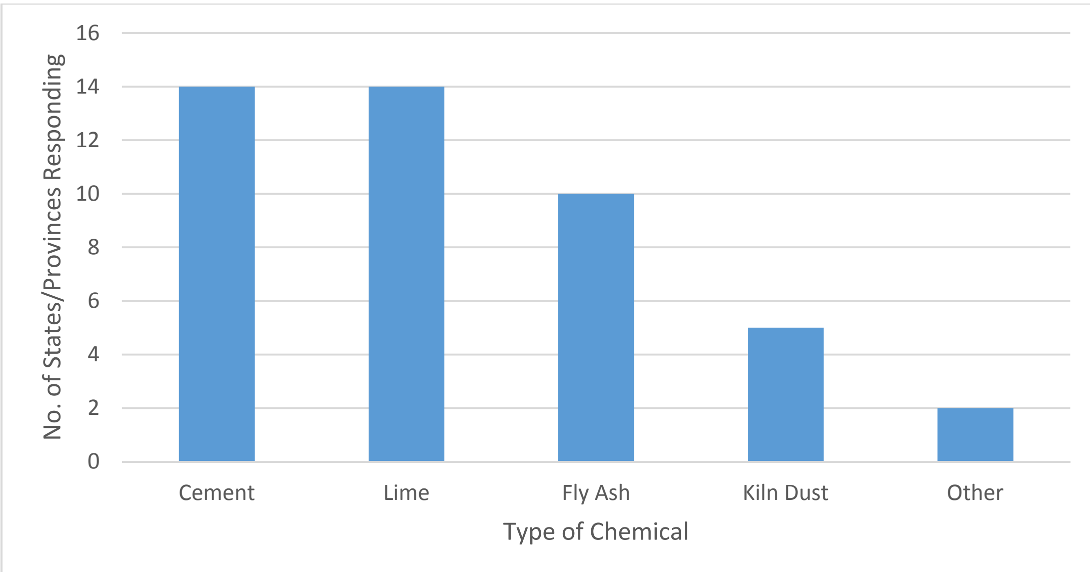


Figure 2.3: Types of Chemicals Used for Subgrade Stabilization by Various States (Sargand, 2014)

Additional important information obtained from this survey included how some of the states incorporated the stabilized subgrade into the pavement design process and strength criteria for design and acceptance of stabilized sections. Table 2.11 shows how some states incorporated stabilized subgrade into pavement design process and Table 2.12 shows acceptance/design criteria for stabilized subgrade.

Table 2.11: How Surveyed States Assign Credit for Stabilized Subgrade in Pavement Design (Sargand, 2014)

State	Cement-Stabilized		Lime-Stabilized			Fly Ash-Stabilized	
	Structural Coefficient	Other	Structural Coefficient	Modulus	Other	Structural Coefficient	Modulus
AR	0.2		0.07				
KS	0.11		0.11			0.11	
KY	0.1		0.08				
MD	Base: 0.15-0.25 Subgrade: 0.05-0.07						
MS	0.2					0.2	
NE				M_R of 30,000 psi			M_R of 30,000 psi
NC		1.0 towards the structural number			1.0 towards the structural number		
SC	0.15						

Table 2.12: Minimum Unconfined Compressive Strength Criteria for Design/Acceptance of Stabilized Subgrade (Sargand, 2014)

State	Cement	Lime	Fly Ash	Kiln Dust
AR	400 psi at 7 days			
IL	500 psi at 7 days	150 psi at 48 hours		
KY	Cores have UCS of 80 psi at 7 days	Cores have UCS of 80 psi at 7 days		
MD	450 psi for base/300 psi for subgrade at 7 days	450 psi for base/300 psi for subgrade at 7 days		
MI				125 psi (optimum CKD) at 7 days
MS	300 psi	CBR 20	400 psi	
NE	No minimum, test the specific soil with varying percentage of lime or fly ash and optimize strength vs. economy			
NC	200 psi at 7 days	58 psi at 7 days		
OH	100 psi at 8 days and minimum increase of 50 psi over unstabilized	100 psi at 8 days and minimum increase of 50 psi over unstabilized		100 psi at 8 days and minimum increase of 50 psi over unstabilized
OK	50 psi greater than untreated at 7 days	50 psi greater than untreated at 7 days	50 psi greater than untreated at 7 days	50 psi greater than untreated at 7 days
SC	300 psi at 8 days			
TX	No requirement for road mix, 175 psi for plant mix	No requirement	No requirement	

The main objective of Sargand's study was to determine how to incorporate the increase in stiffness of stabilized subgrade into pavement design. This was achieved by using stabilized pavement sections and Portable Seismic Properties Analyzer (PSPA), FWD, coring and DCP testing.After analyzing hundreds of stabilized pavement sections in Ohio, the layer coefficients shown in Figure 2.4 should be incorporated into the flexible pavement thickness design when using the 1993 AASHTO Guide for Design of Pavement Structures. The chart shown in Figure 2.4 should be used with an appropriate level of confidence for the pavement structure being designed.

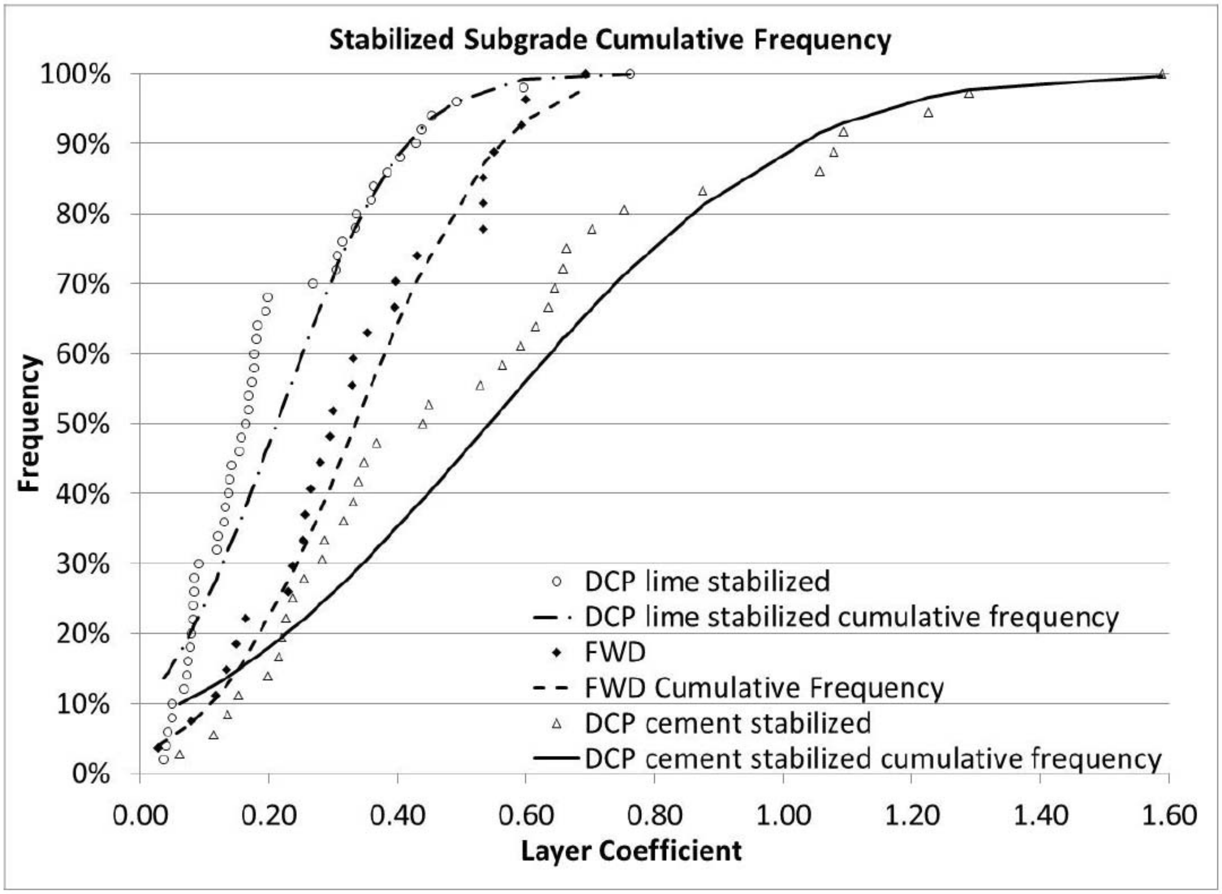


Figure 2.4: Cumulative Frequency of Stabilized Subgrade Layer Coeffcients – FWD vs. DCP (Sargand, 2014)

For the Mechanistic Empirical (ME) Pavement Design, this study recommended applying a multiplier to the natural subgrade to obtain the modulus of stabilized subgrade as shown in Table 2.13.

Table 2.13: Multiplier to the Natural Subgrade Modulus Recommended for ME Pavement Design (Sargand, 2014)

Stabilizing Material	Multiplier to the Natural Subgrade Modulus to Obtain the Modulus of Stabilized Subgrade
Cement	4.7
Lime	3.9

A Minnesota Department of Transportation study, titled *Subgrade Stabilization ME Properties Evaluation and Implementation*, investigated the procedure to include stabilized layer stiffness for ME pavement design (Budge, 2012). Through literature review, the research team has recommended using a Resistance Factor (RF) for pavement design. The RF is based on a ratio of the stiffness of the stabilized material to the stiffness of the native (untreated) materials. The RF value for ME design is obtained as follows,

$$RF = \frac{M_r(stabilized)}{M_r(native)} \leq 2$$

Where:

RF = Resistance Factor for Stiffness

M_r(stabilized) = Resilient modulus of the stabilized material

M_r (native) = Resilient modulus of the native (untreated) material

CHAPTER 3: LABORATORY INVESTIGATION

The laboratory investigation procedure was developed to explore the suitability of different recycled materials for stabilizing problematic soils such as those commonly found in Michigan. The first step of this activity was to identify potential recycled materials for subgrade stabilization. The literature review revealed a handful of recycled material that can be considered in subgrade stabilization. A brief introduction to potential recycled materials is found below.

Crushed Concrete Fines (CF): The addition of crushed concrete fines can improve the engineering properties of clayey soils for subgrade stabilization purposes. The improvement is partly due to the flocculation and coagulation of colloidal clay minerals that react with calcium hydroxide [$Ca(OH)_2$] to form larger grains in the silt fraction.

Cement Kiln Dust (CKD): CKD is a byproduct of the production of Portland cement. The fines captured in the exhaust gases of the production of Portland cement contain about 30 to 40 % Calcium Oxide (CaO) or lime and 20 to 25 % of pozzolanic materials. (NCHRP, 2009) Treatment with CKD was found to be an effective option for improvement of soil properties. Cement Kiln Dust increases strength and stiffness and substantially reduces plasticity and swell potential. However, Parsons et al. (2004) observed that strength decreased during freeze/thaw testing.

Lime Kiln Dust (LKD): Limestone ($CaCO_3$) is used to produce lime. LKD is a byproduct of lime production. This byproduct contains approximately 30 to 40% lime as well as potential pozzolans (silicious material). In most cases, the fuel used in the kiln to create the chemical environment needed to convert $CaCO_3$ to lime is of poor quality. The burning of poorer grade fuels can be a source of the pozzolans. In the presence of pozzolans, LKD may become somewhat pozzolanic reactive. However, if no pozzolans are produced or if they are of low quality, the LKD may be nonreactive. (NCHRP, 2009) One benefit of LKD is its size. Since it is very fine, it can be used to modify soil particle distribution. This makes LKD suitable for stabilizing a range of problematic soils.

Fly Ash (FA): Fly ash is the byproduct of the combustion of coal. It is generally rich in silica and alumina. There are two types of fly ash: Type F and Type C. Type F fly ash is generally available in large quantities, but the lime content is usually less than 15%. In the past, MDOT has used a mixture of Type F fly ash and lime as a soil subgrade stabilizer. Alternately, Type C fly ash has a lime content that is generally greater than 15% and often as high as 30%. The elevated lime content gives Type C fly ash a unique property of self-cementing which makes it effective in stabilizing fine-grained soils.

About 10 – 20% of fly ash (by dry weight) is usually used to stabilize soil. Fly ash is known to produce a soil mix that has improved compaction properties, i.e. higher maximum dry unit weight at lower optimum moisture content.

Lime-cement-fly ash and/or Lime-fly ash: Mixtures of lime-cement-fly ash and/or lime-fly ash have been successfully used as a subgrade stabilization material. The recycled portion of the material depends on the mix-proportion of fly ash found within it. Fly ash is known to speed up the pozzolanic reaction in lime and/or cement. Since lime and/or cement provide the cementitious bond, even non-cementitious fly ash can be used in these mixtures.

Once the list of potential recycled materials was identified, the laboratory testing program commenced. The testing set out to determine the short-term and long-term advantages and dis-advantages of using the selected recycled stabilizing materials in conjunction with the common problematic soils found in Michigan. The major soil types found in Michigan were described in the MDOT research report RC-1531 (Baladi et al., 2009). In this report, Baladi et al. (2009) categorized the majority of the roadbed soils in the State of Michigan into eight general soil types using the Unified Soil Classification System (USCS) and the AASHTO soil classification systems (Table 3.1).

Table 3.1: Major Soil Types Found in Michigan (Baladi et al., 2009)

USCS	AASHTO	Soil Type
SP-1	A-1-a, A-3	Sand, sand with gravel
SP-2	A-1-b, A-3	Sand, gravel with silt
SP-SM*	A-1-b, A-2-4, A-3	Sand with silt
SC-SM*	A-2-4, A-4	Silty clayey sands
SC*	A-2-6, A-6, A-7-6	Clayey sand
SM	A-2-4, A-4	Silty sand
CL*	A-4, A-6, A-7-6	Clay
ML	A-4	Silt

*MDOT critical soil type

Out of these eight soil types, the following soil types were identified as critical for Michigan based on the discussions with the MDOT Research Advisory Panel (MDOT-RAP) members and the Project Manager (PM). These include SC-SM, SP-SM, SC, and CL. SP-SM soil usually does not need any stabilization. However, this type of soil is sometimes encountered when old sand subbase is left in place during new construction activities. These layers of old sand subbase, when found on unstable clay subgrades, often need stabilization.

Once the soil types were identified and samples were obtained from the different test regions in Michigan, three mix proportions were considered for each stabilization method (Figure 3.1).

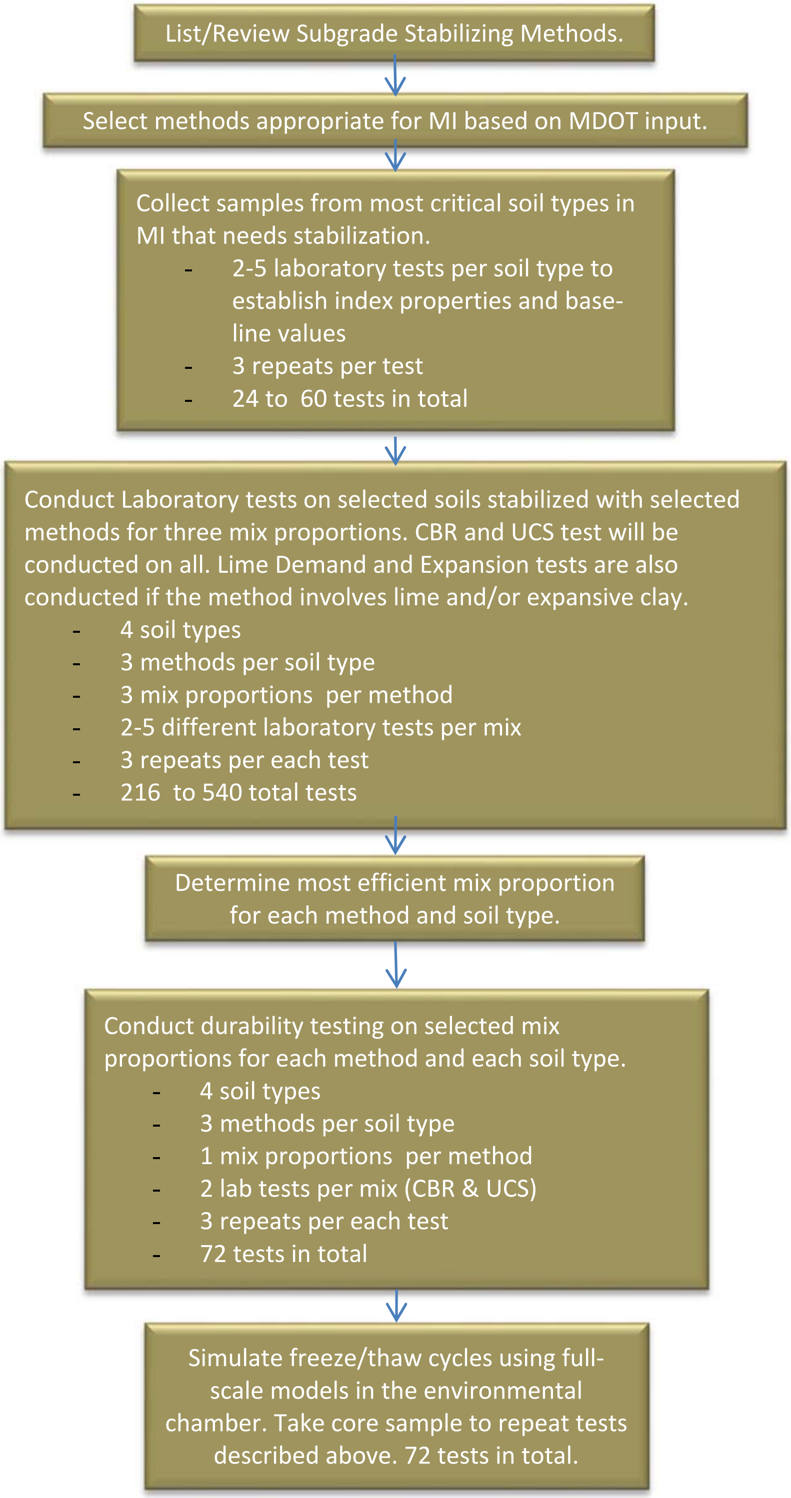


Figure 3.1: Summary of the Laboratory Test Program Process

Each mix proportion was tested in triplicate using the tests listed above. This combination of testing was conducted for all the critical Michigan soil types. More details of the laboratory testing program are described in the sections following the final list of selected recyclable material.

3.1 Develop a List of Potential Recycled Materials for Subgrade Stabilization

The following recycled stabilizing materials were used in this laboratory program to determine their pavement subgrade stabilization performance. These materials were selected because they are readily available in large quantities and found in Michigan.

1. Cement Kiln Dust (CKD) – CKD was supplied by Lafarge from their Alpena, Michigan, cement plant
2. Lime Kiln Dust (LKD) – Two types of LKDs were supplied by Mintek Resources, Michigan:
    a. LKD – LKD is obtained by burning limestone
    b. DLKD – LKD is obtained by burning dolomitic limestone
3. Fly Ash (FA) – FA was provided by the Detroit Edison, Monroe, Michigan, power plant[1]
4. Crushed Concrete Fines (CF) – CF were generated by crushing Portland Cement Concrete pavement materials taken from I-96, Livonia, Michigan
5. LKD/FA mix[2]- to provide free lime to FA for hydration Supplied by Minteck and Detroit Edison, respectively

3.2 Soil Selection

Taken from three different MDOT construction sites, these untreated soils were submitted for laboratory testing:

1. Soil Sample 1 – MDOT Bascule Bridge Reconstruction Project in Detroit, Michigan
2. Soil Sample 2 – MDOT I-96 Reconstruction Project in Livonia, Michigan
3. Soil Sample 3 – MDOT Construction Project in the Upper Peninsula, Michigan

These soils were deemed unsuitable construction materials by MDOT due to poor field performance. As a result, the soils were removed and replaced by MDOT with suitable materials.

3.3 Laboratory Testing – Untreated Soils

3.3.1 Grain Size Analysis

Pursuant to *ASTM D442 – Standard Test Method for Particle-Size Analysis of Soils*, a grain size analysis was used to classify the particle size of the untreated soils. The results were classified as the percentage of soil passing, or Percent Passing (PP), through an ASTM sieve number 200 (0.075 mm). A wet sieve analysis was utilized**.**

Approximately 600 g oven dry soil sample was tested with the wet sieve method and more than 50% for all three soil types, passed through the ASTM standard sieve number 200 (0.075 mm).

[1] Laboratory results showed the amount of free lime (CaO) was 21.5% by weight.
[2] A mix was used to provide free lime to FA for hydration.

The results were 99.5%, 65.8% and 99% for untreated Soil Sample-1, Soil Sample-2 and Soil Sample-3 respectively. Figure 3.2 shows the test set for wet sieve analysis used during this project.

Figure 3.2: Wet Sieve Testing

3.3.2 Atterberg Limit Tests

The Atterberg Limit Tests were conducted according to ASTM D4318. Figure 3.3 shows the Cassagrande apparatus used for liquid limit testing. Average results of the liquid limit tests and plastic limits tests of untreated soil samples soils are shown in Table 3.2.

Table 3.2: Atterberg Limit Test Results of the Selected Soil Samples

Sample Number	Liquid Limit %	Plastic Limit %	Plasticity Index %
Soil Sample-1	31.3	19.2	12.1
Soil Sample-2	16.0	12.4	3.6
Soil Sample-3	48.1	26.6	21.5

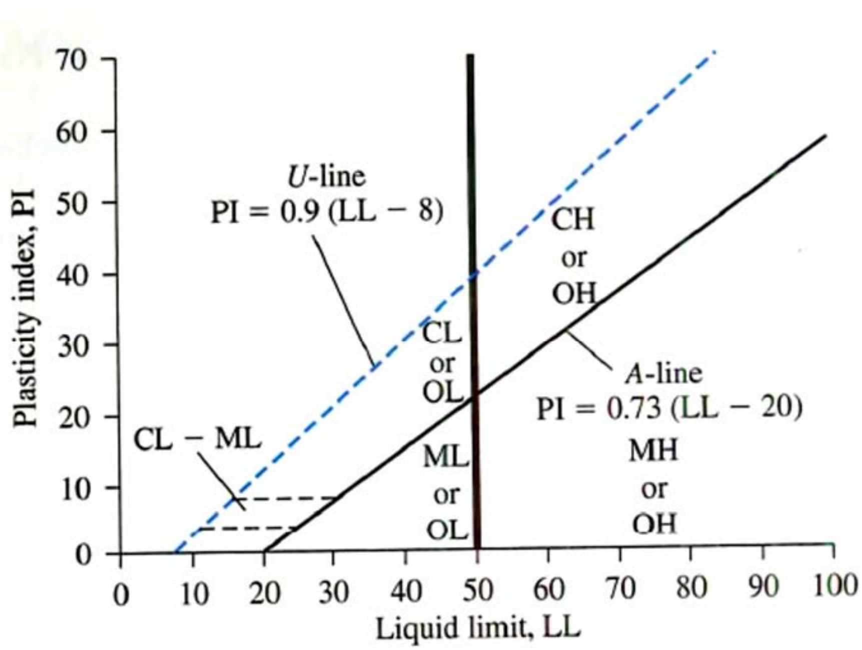
Figure 3.3: Cassagrande Apparatus for Liquid Limit Testing

3.3.3 Soil Classification

After completion of the Atterberg Limit Test and the sieve analysis, the soil samples were classified according to AASHTO and USCS. Figure 3.4 shows the soil classification criteria according to the USCS. Figure 3.5 lists the AASHTO soil classification criteria. The test results used for soil classification and results of the soil classification based on USCS and AASHTO procedures are shown Table 3.3.

Table 3.3: Classification of Selected Soils

Sample Number	Passing ASTM Sieve # 200 %	Liquid Limit %	Plastic Limit %	Plasticity Index %	Classification	
					USCS	AASHTO
Soil Sample-1	99.5	31.3	19.2	12.1	CL	A-6
Soil Sample-2	65.8	16.0	12.4	3.6	ML	A-4
Soil Sample-3	98.9	48.1	26.6	21.5	CL	A-7-6



Criteria for assigning group symbols				Group symbol
Coarse-grained soils More than 50% of retained on No. 200 sieve	**Gravels** More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[a]	$C_u \geq 4$ and $1 \leq C_c \leq 3$[c]	GW
			$C_u < 4$ and/or $1 > C_c > 3$[c]	GP
		Gravels with Fines More than 12% fines[a,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	GM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	GC
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[b]	$C_u \geq 6$ and $1 \leq C_c \leq 3$[c]	SW
			$C_u < 6$ and/or $1 > C_c > 3$[c]	SP
		Sands with Fines More than 12% fines[b,d]	$PI < 4$ or plots below "A" line (Figure 5.3)	SM
			$PI > 7$ and plots on or above "A" line (Figure 5.3)	SC
Fine-grained soils 50% or more passes No. 200 sieve	**Silts and clays** Liquid limit less than 50	Inorganic	$PI > 7$ and plots on or above "A" line (Figure 5.3)[e]	CL
			$PI < 4$ or plots below "A" line (Figure 5.3)[e]	ML
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OL zone	OL
	Silts and clays Liquid limit 50 or more	Inorganic	PI plots on or above "A" line (Figure 5.3)	CH
			PI plots below "A" line (Figure 5.3)	MH
		Organic	$\frac{\text{Liquid limit — oven dried}}{\text{Liquid limit — not dried}} < 0.75$; see Figure 5.3; OH zone	OH
Highly Organic Soils	Primarily organic matter, dark in color, and organic odor			Pt

Figure 3.4: USCS Soil Classification Chart

General classification	Granular materials (35% or less of total sample passing No. 200)						
	A-1			A-2			
Group classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7
Sieve analysis (percentage passing)							
No. 10	50 max.						
No. 40	30 max.	50 max.	51 min.				
No. 200	15 max.	25 max.	10 max.	35 max.	35 max.	35 max.	35 max.
Characteristics of fraction passing No. 40							
Liquid limit				40 max.	41 min.	40 max.	41 min.
Plasticity index	6 max.		NP	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Stone fragments, gravel, and sand		Fine sand	Silty or clayey gravel and sand			
General subgrade rating	Excellent to good						

General classification	Silt-clay materials (more than 35% of total sample passing No. 200)			
Group classification	A-4	A-5	A-6	A-7 A-7-5[a] A-7-6[b]
Sieve analysis (percentage passing)				
No. 10				
No. 40				
No. 200	36 min.	36 min.	36 min.	36 min.
Characteristics of fraction passing No. 40				
Liquid limit	40 max.	41 min.	40 max.	41 min.
Plasticity index	10 max.	10 max.	11 min.	11 min.
Usual types of significant constituent materials	Silty soils		Clayey soils	
General subgrade rating	Fair to poor			

[a]For A-7-5, $PI \leq LL - 30$
[b]For A-7-6, $PI > LL - 30$

Figure 3.5: AASHTO Soil Classification Chart

3.3.4 Standard Proctor Test

A Standard Proctor Test was performed according to ASTM D698 - Method A. A 4-inch diameter mold with a 1/30 ft^3 volume with three layers of compaction and 25 blows were used per layer. Water was added to the soil samples and compacted using standard effort. The bulk weight of the soil in the 1/30 ft^3 mold was measured.

Moisture content was measured according to ASTM D2216. The dry unit soil weight was calculated from the moisture content results. This procedure was repeated by increasing the moisture content in order to plot a parabolic dry density – moisture content curve. The ordinate of the apex gives the Optimum Moisture Content (OMC) while the corresponding abscissa provides the Maximum Dry Density (MDD). Figure 3.6 illustrates a prepared sample for a Standard Proctor Test. The average results of this test are shown in Table 3.4.

Figure 3.6: Sample Prepared for Standard Proctor Test

Table 3.4: Maximum Dry Density (MDD) and OMC of Untreated Soil

Sample Number*	MDD (pcf)$^+$	OMC (%)
Soil-1 (A-6)	108.80	16.20
Soil-2 (A-4)	120.69	11.68
Soil-3 (A-7-6)	95.32	20.01

*Includes AASHTO Classification

$^+$ pcf = pounds per cubic foot

3.3.5 Calibration of Harvard Miniature Compaction Apparatus

A Harvard Miniature Compaction Apparatus was used to prepare the soil samples for UCS testing. Water was added to the soil in order to achieve an OMC. The samples were compacted at their MDD. Calibration of the Harvard Miniature Compaction Apparatus was performed to determine the required moisture content and compaction effort needed to achieve the MDD. This was necessary as the sample sizes generated with this apparatus differ from the Standard Proctor Test sample size. Calibration of Harvard Miniature Compaction apparatus was performed according to ASTM D4609 ANNEX A1. The Harvard Miniature Compaction Apparatus (Figure 3.7) includes a cylindrical mold having an inside diameter of 1.3125 inches, a height of 2.816 inches, and a volume of 1/454 ft^3 (62.4cm^3); a spring loaded plunger; three springs (20 lbs., 37.5 lbs. and 40 lbs.); and a sample extruder. Calibration of this device involved determining the correct spring and weight, the number of blows needed per layer and number of layers required to match the Standard Proctor Test dry density value. This was achieved through a series of trial and error tests of different combinations. Soil was compacted at various moisture contents using different springs, a number of layers of compaction, and a number of blows per layer. The dry densities were calculated based on the different compaction efforts. These results were then plotted against the corresponding moisture content along with a dry density moisture content graph obtained from Standard Proctor Test. The compaction effort having a density within one pcf of the MDD was selected for preparing samples for the UCS test. This calibration procedure was performed on all untreated soils and the soil/treatment mix ratios.

Figure 3.8 shows the calibration graph for untreated Soil Sample-1. The dry density-moisture content graph from ASTM D698 test coincides with a graph of the sample generated with the 37.5-lb. spring, five compaction layers and 20 blows per layer. Hence, in the case of untreated Soil Sample-1 (A-6), the 37.5-lb. spring with a compaction effort of 20 standard blows and five layers at OMC was sufficient to achieve a MDD. Table 3.5 summarizes the calibration results of all three untreated soils.

Figure 3.7: Harvard Miniature Compaction Apparatus (from left to lower right): Extruder, Tamper and Springs, Mold and Samples

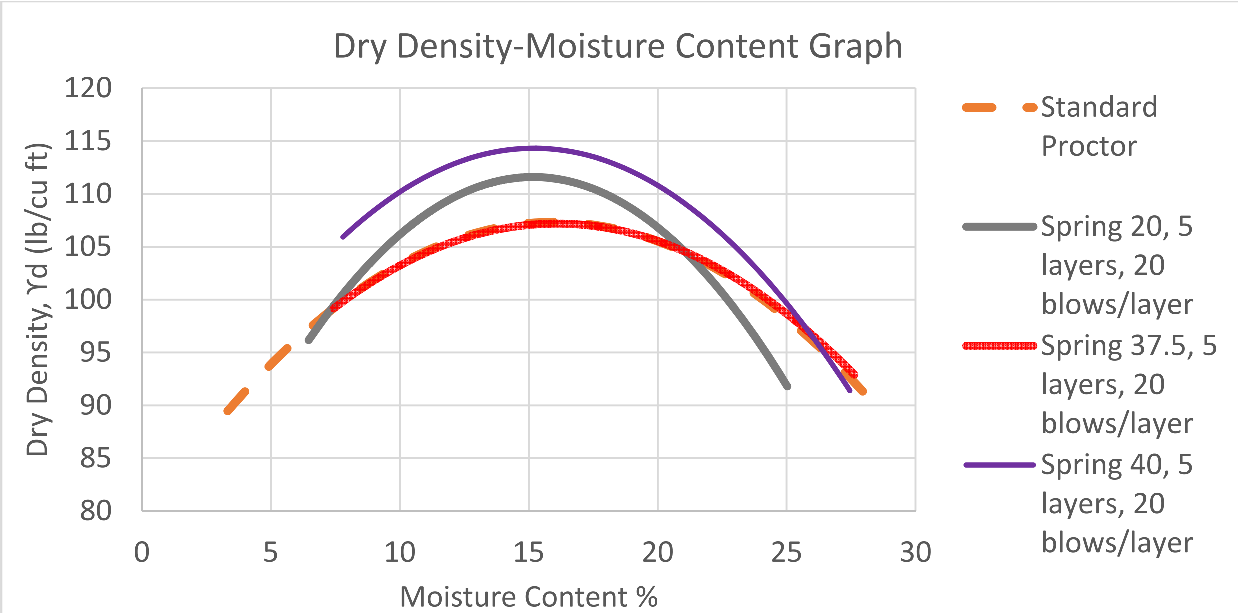


Figure 3.8: Calibration Graph of Harvard Miniature Compaction Apparatus

Table 3.5: Calibration Results of Untreated Soils

Soil Number	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
Soil-1 (A-6)	37.5	5	20	27.63
Soil-2 (A-4)	37.5	5	20	10.58
Soil-3 (A-7-6)	37.5	5	15	19.87

3.3.6 Curing

After calibration of the compaction apparatus, different soil mixes were compacted at their respective MDD and kept moist in order to cure. The samples were then tested after intermittent curing periods in order to determine the change in UCS relative to cure time. For curing, each compacted sample was placed in a small open plastic bag and stored at room temperature within a larger, sealed plastic bag half-filled with water. The opening of the smaller plastic bag was kept above the water level (Figure 3.9). This curing technique allowed the samples to retain moisture without coming into direct contact with the water.

The curing period varied, e.g. zero days, one day, three days, seven days, 14 days and 28 days, for different samples. The curing period allowed the soil to react with the stabilizer. The untreated soils were not cured.

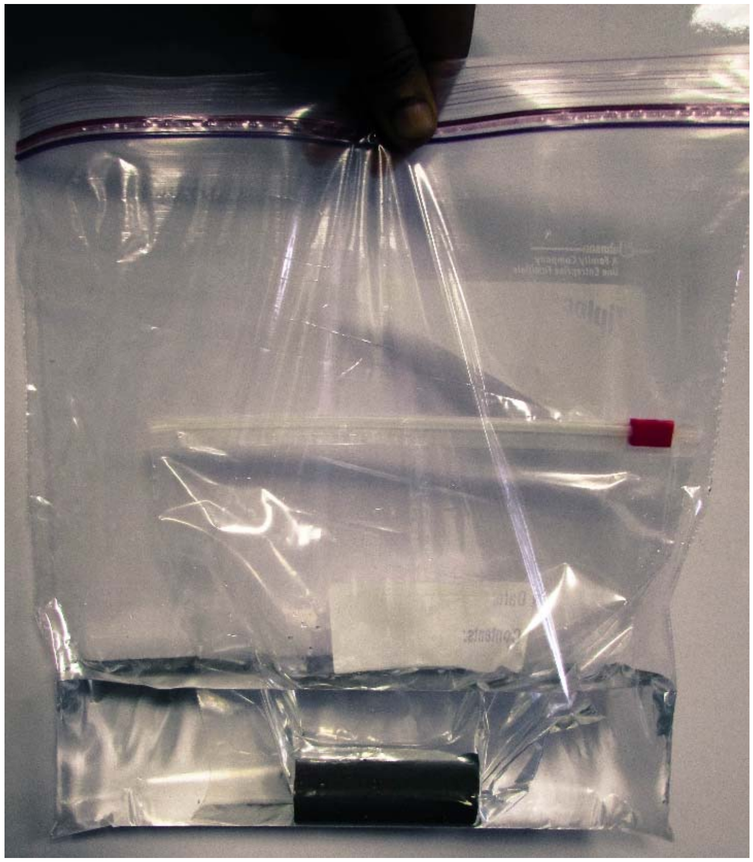


Figure 3.9: Sample Curing Procedure

3.3.7 Capillary Soaking

The moisture state equivalent to UCS of soaked samples was introduced via capillary soaking. As a result of this process, strength loss due to the presence of moisture was determined. Unconfined Compressive Strength Tests were performed on both soaked and unsoaked samples. A 24-hr capillary soaking period was started either immediately after compaction or after compaction and curing. Samples were wrapped individually with water absorbent paper and placed on partially submerged porous stones. The water level was maintained just below the top of the porous stone. As a result, the soil sample could absorb moisture by capillary soaking without being in direct contact with the water (Figure 3.10). Capillary soaking simulates actual water movement in field conditions.

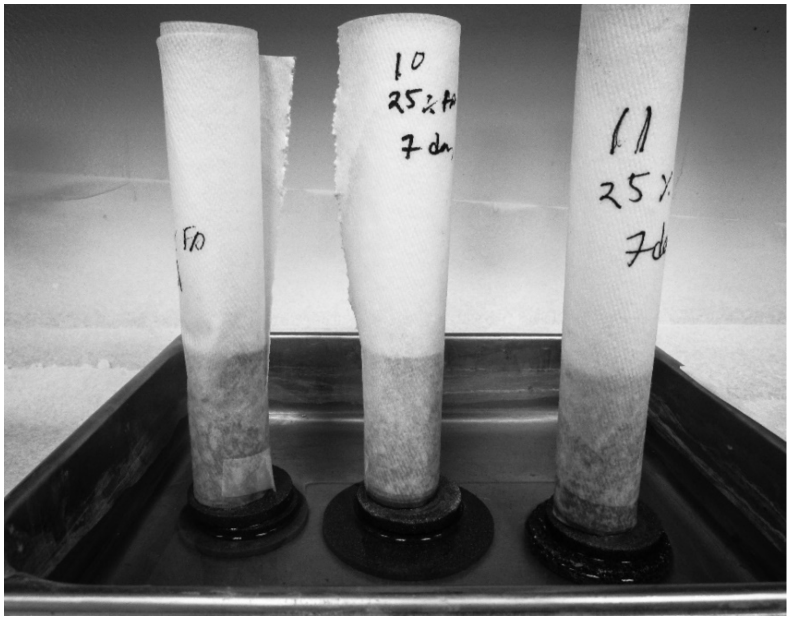


Figure 3.10: Capillary Soaking Procedure

3.3.8 Unconfined Compressive Strength (UCS) Test

Unconfined Compressive Strength tests of the untreated soil were performed using samples prepared by the Harvard Miniature Compaction Apparatus. All samples were compacted to OMC and MDD levels. The OMC determined from the calibration process was used to prepare soils for UCS testing in lieu of the using the OMC results obtained from the Standard Proctor Test. ASTM D2611 was followed for the UCS tests with a strain rate of 1%. Figures 3.11 and 3.12 show typical stress-strain curve produced for UCS testing and failed soil samples, respectively.

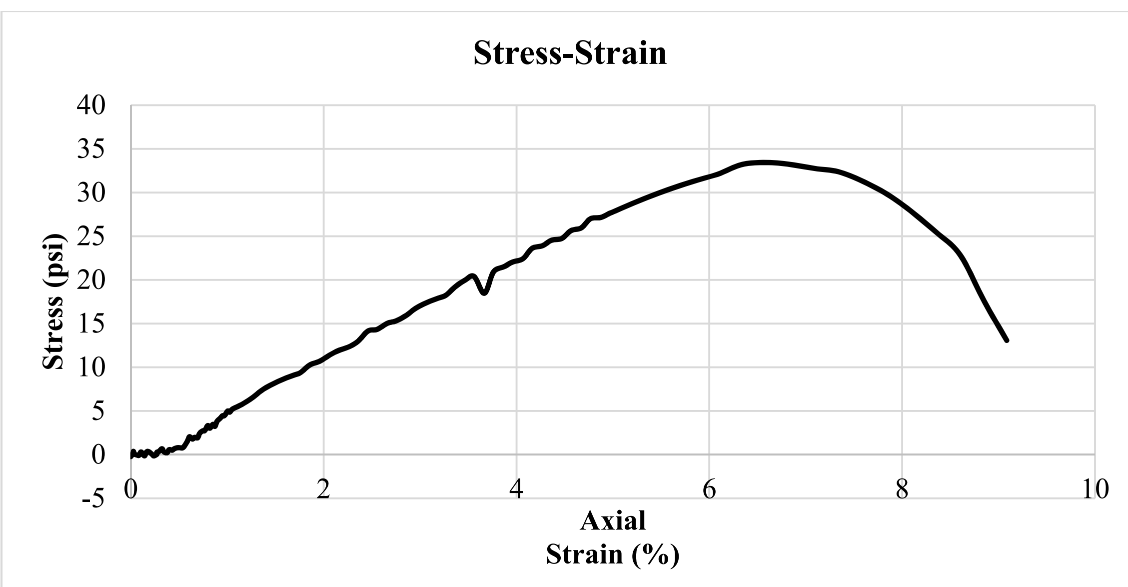


Figure 3.11: Stress-Strain Curve for the UCS Soil Sample-1, Specimen- 1 (Unsoaked)

Figure 3.12: Soil Specimen after Failure

Table 3.6 presents the UCS results of the untreated soils with soaked and unsoaked conditions.

Table 3.6: Properties of Selected Soils

Soil Number	MDD, Υ_d (lb/ft^3)	OMC (%)	UCS (psi)	
			Soaked	Unsoaked
Soil-1 (A-6)	108.8	16.2	2.61	32.26
Soil-2 (A-4)	120.7	11.7	3.25	36.00
Soil-3 (A-7-6)	95.3	20.0	1.43	62.49

3.4 Laboratory Mix Design and Testing

3.4.1 Grain Size Analysis - Stabilizer Materials

No grain size analysis was performed on CKD, LKD, DLKD, and FA. These stabilizers were finer than the original untreated soils. As such, they did not impact grain size when mixed with the untreated soils.

As CF are a courser material, they were first sieved before being mixed with the untreated soils. Only fines passing through a # 8 sieve were used for mixing purposes.

3.4.2 Selection of Mix Ratio for Long-Term Stabilization

ASTM D4609 - Standard Guide for Evaluating Effectiveness of Chemicals for Soil Stabilization was used to evaluate the effectiveness of chemical stabilization. Based on this standard, an increase of UCS by 50 psi or more over the USC of the untreated soils, after seven days of curing and 24 hours of capillary soaking, was considered to be the benchmark for long-term stabilization. Similarly, an increase of UCS by 50 psi or more over the initial USC of the untreated soils, after three days of curing and without capillary soaking, was considered to be the benchmark for short-term subgrade modification. Short-term subgrade modification would provide sufficient subgrade strength for movement of construction traffic. Laboratory test results are summarized in the following sections. The selected recycled materials and their required mix ratio with different types of soil were developed from the laboratory tests.

3.5 Subgrade Stabilization and Modification

In order for a chemical treatment, or this case – subbase stabilizer, to be considered "effective," an UCS increase of 50 psi over the initial soil must be observed. This guideline is provided by *ASTM D4609 - Standard Guide for Evaluating Effectiveness of Chemicals for Soil Stabilization.*

The research team recommends following as the guideline to use recycled materials for long-term subgrade stabilization or soil modification for construction facilitation. These guidelines are developed based on the prior literature research and discussions with MDOT personnel.

1. Long-term Subgrade Stabilization – 50 psi more increase of UCS due to stabilization over untreated UCS values after seven days of curing and 24 hour of capillary soaking. Capillary soaking simulates the ground water movement during the life of the pavement and resultant strength loss due to presence of moisture.
2. Short-term Subgrade Modification (for construction facilitation) – 50 psi or more increase of UCS due to stabilization over untreated values after three days of curing (without capillary soaking). A curing period of three days was selected for short-term modification since it is not practical to wait more than three days to construct upper pavement layers after subgrade modification.

3.5.1 Cement Kiln Dust (CKD)

All three soils were mixed with different percentages of CKD (Table 3.7). The tests mentioned in Section 3.3 were repeated on each representative soil/CKD mix.

Table 3.7: Percentages of CKD Mixed with Different Soil Types

Soil	CKD (%)
Soil-1 (A-6)	6, 8, 12
Soil-2 (A-4)	4, 6, 8
Soil-3 (A-7-6)	4, 6, 8

3.5.1.1 Atterberg Limit Tests

The Atterberg Limit Test results and the updated soil classifications of CKD-stabilized Soil-1, Soil-2 and Soil-3 are shown in Tables 3.8, 3.9 and 3.10 respectively.

Table 3.8: Atterberg Limit Test Results of CKD and Soil-1 (A-6) Mix

Percentage CKD	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
6	Plastic Limit, PL	23.4	26.6	22.9	24.3	A-4
	Liquid Limit, LL	33.0	32.4	34.1	33.2	
	Plasticity Index, PI	9.6	5.8	11.2	8.9	
8	Plastic Limit, PL	33.5	40.9	38.6	37.7	A-5
	Liquid Limit, LL	40.5	48.5	50.2	46.4	
	Plasticity Index, PI	7.0	7.6	11.6	8.7	
12	Plastic Limit, PL	28.5	24.3	25.2	26.0	A-4
	Liquid Limit, LL	35.2	35.0	33.9	34.7	
	Plasticity Index, PI	6.7	10.7	8.7	8.7	

*Plasticity Index (PI) = LL - PL

A significant reduction of the Plasticity Index (PI) was observed for the Soil-1 stabilized with CKD. Due to the decrease in PI, the soil classification moved to the left of the AASHTO soil classification chart. This would indicate an improvement in soil quality.

Table 3.9: Atterberg Limit Test Results of CKD and Soil-2 (A-4) Mix

Percentage CKD	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
4	Plastic Limit, PL	26.0	22.6	19.2	22.6	A-4
	Liquid Limit, LL	28.2	28.0	28.0	28.1	
	Plasticity Index, PI	2.3	5.4	8.8	5.5	
6	Plastic Limit, PL	25.5	20.8	22.8	23.0	A-4
	Liquid Limit, LL	32.2	26.8	33.6	30.8	
	Plasticity Index, PI	6.7	6.01	10.8	7.8	
8	Plastic Limit, PL	18.6	17.7	21.3	19.2	A-4
	Liquid Limit, LL	24.5	24.5	24.5	24.5	
	Plasticity Index, PI	5.9	6.8	3.3	5.3	

*Plasticity Index, PI = LL - PL

In the case of Soil-2 (A-4) and CKD mixes, both the Liquid Limit and the Plasticity Index increased slightly at all percentages of CKD. Despite the increase, the classification remained the same (A-4) as the untreated soil.

Table 3.10: Atterberg Limit Test Results of CKD and Soil-3 (A-7-6) Mix

Percentage CKD	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
4	Plastic Limit, PL	25.7	26.7	26.74	26.4	A-7-6
	Liquid Limit, LL	41.1	41.0	40.78	40.9	
	Plasticity Index, PI	15.4	14.2	14.04	14.5	
6	Plastic Limit, PL	23.1	21.7	5.43	16.7	A-6
	Liquid Limit, LL	38.4	38.1	38.73	38.4	
	Plasticity Index, PI	15.3	16.4	33.31	21.7	
8	Plastic Limit, PL	32.5	31.9	30.99	31.8	A-7-6
	Liquid Limit, LL	46.6	46.5	47.59	46.9	
	Plasticity Index, PI	14.2	14.7	16.60	15.2	

*Plasticity Index, PI = LL – PL

For all percentages, the Liquid Limit and the Plasticity Index were reduced for Soil-3 stabilized with CKD. The classification for 4% CKD and 8% CKD was the same as untreated Soil-3 (A-7-6). In the case of 6% CKD, the AASHTO classification was changed to A-6.

3.5.1.2 Standard Proctor Test

Standard Proctor Tests were performed on the soil/CKD mixes according to the ASTM D698. Based on these tests, the MDD and the OMC of these mixes were determined and are shown in the Tables 3.11, 3.12 and 3.13.

Table 3.11: MDD and OMC of Soil-1 (A-6) mixed with CKD

Test	6% CKD		8% CKD		12% CKD	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	102.1	17.8	102.9	15.0	99.7	17.5
2	102.1	16.5	103.4	17.0	97.5	18.3
3	103.8	17.9	103.0	15.2	100.1	18.0
Average	102.7	17.4	103.1	15.7	99.1	17.9

Table 3.12: MDD and OMC of Soil-2 (A-4) mixed with CKD

Test	4% CKD		6% CKD		8% CKD	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	112.3	13.2	109.3	13.5	113.7	12.7
2	111.5	13.9	109.9	15.6	113.4	13.4
3	113.2	14.2	110.8	13.2	114.2	12.6
Average	112.4	13.8	110.0	14.1	113.8	12.9

Table 3.13: MDD and OMC of Soil-3 (A-7-6) mixed with CKD

Test	4% CKD		6% CKD		8% CKD	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	94.7	22.2	95.7	24.3	95.7	20.6
2	94.9	21.6	96.3	26.0	95.3	21.5
3	94.1	29.3	95.7	23.6	94.3	21.0
Average	94.5	24.4	95.9	24.6	95.1	21.1

3.5.1.3 Calibration of Harvard Miniature Compaction Apparatus

As outlined in Section 3.3.5, the Harvard Miniature Compaction Apparatus was calibrated every time a different mix ratio of soil and CKD was used. As a result, new spring weights, layer numbers, and blow/layer numbers were generated. Results of the calibration are summarized in Tables 3.14 through 3.16.

Table 3.14: Harvard Miniature Compactor Apparatus Calibration for Soil-1 (A-6) Mixed with CKD

Percentage CKD	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
6	37.5	5	20	17.4
8	37.5	5	20	19.9
12	37.5	5	20	17.5

Table 3.15: Harvard Miniature Compactor Apparatus Calibration for Soil-2 (A-4) Mixed with CKD

Percentage CKD	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
4	37.5	5	20	13.0
6	37.5	5	20	13.3
8	37.5	5	20	12.7

Table 3.16: Harvard Miniature Compactor Apparatus Calibration for Soil-3 (A-7-6) Mixed with CKD

Percentage CKD	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
4	37.5	5	20	22.3
6	37.5	5	20	20.0
8	37.5	5	20	21.6

3.5.1.4 Unconfined Compressive Strength (UCS) Test

Using samples prepared by the Harvard Miniature Compaction Apparatus, UCS tests were performed on the soil/CKD mixes as outlined in Section 3.3.8. After compaction, the samples were cured for 0, 1, 3, 7, 14 and 28 days prior to UCS testing. After curing, some soil samples were subjected to capillary soaking. The cured and soaked samples had an increased UCS (Figures 3.13, 3.15 and 3.17). The cured and unsoaked samples also had an increased UCS (Figures 3.14, 3.16 and 3.18).

As stated in Section 3.4.2, a UCS increase of 50 psi greater than that of the untreated soil fabricated and cured under the same conditions as the stabilized material can be used to define long-term stabilization. The UCS results of the Soil-1 (A-6) and CKD mixes, show that 6% CKD did not achieve a 50 psi or greater strength increase (Figure 3.13). However, both the 8% CKD and 12% CKD mixes exceeded the 50 psi strength increase after seven days of curing. As the UCS results were very similar between the 8% CKD and the 12% CKD, 8 8% CKD was selected for Soil-1. As shown in Figures 3.15 and 3.17, 4% CKD was selected for stabilization for Soil-2 (A-4) and Soil-3 (A-7-6).

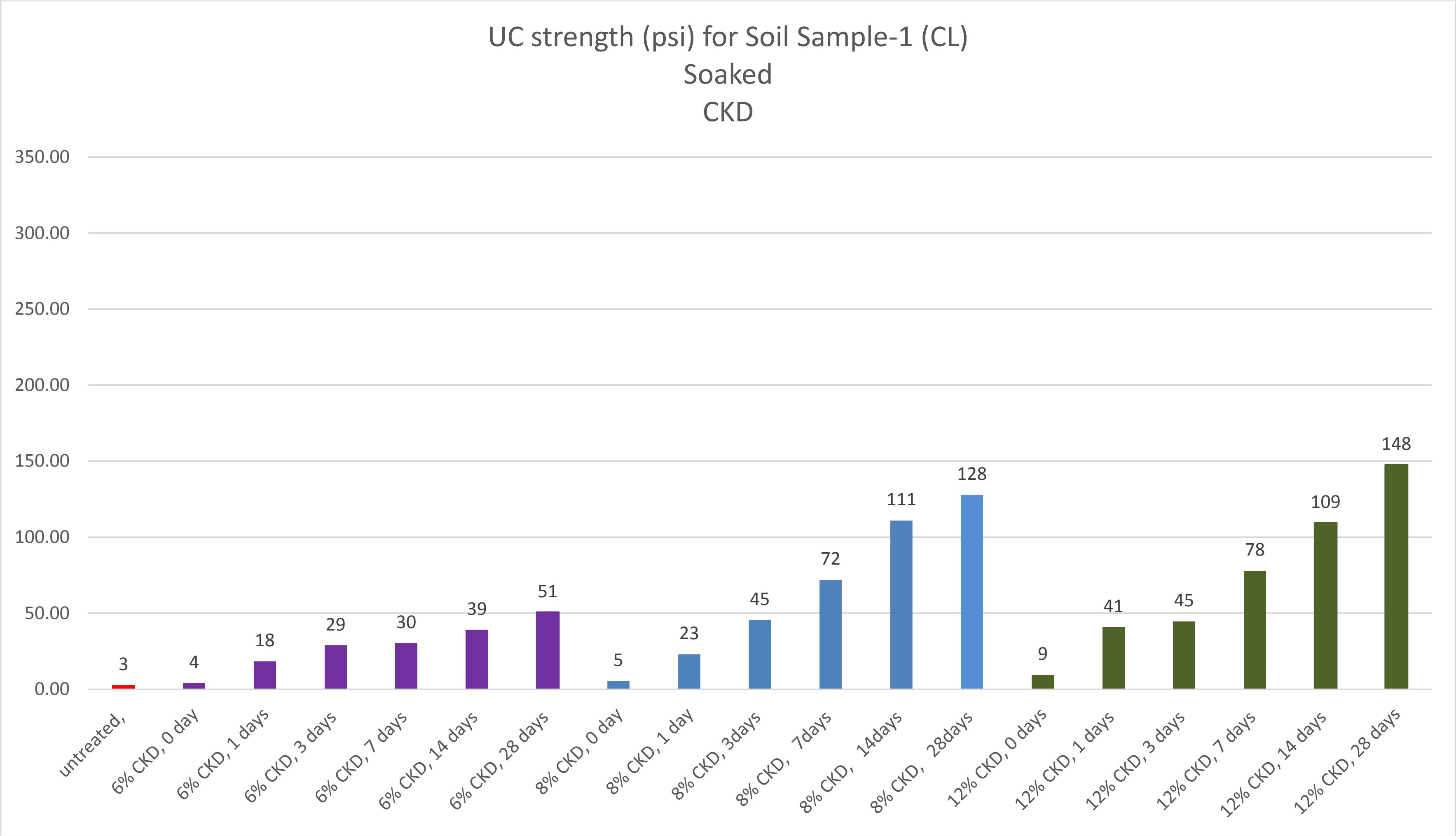


Figure 3.13: Comparison of Soaked UCS of Soil-1 (A-6) & CKD Mixes

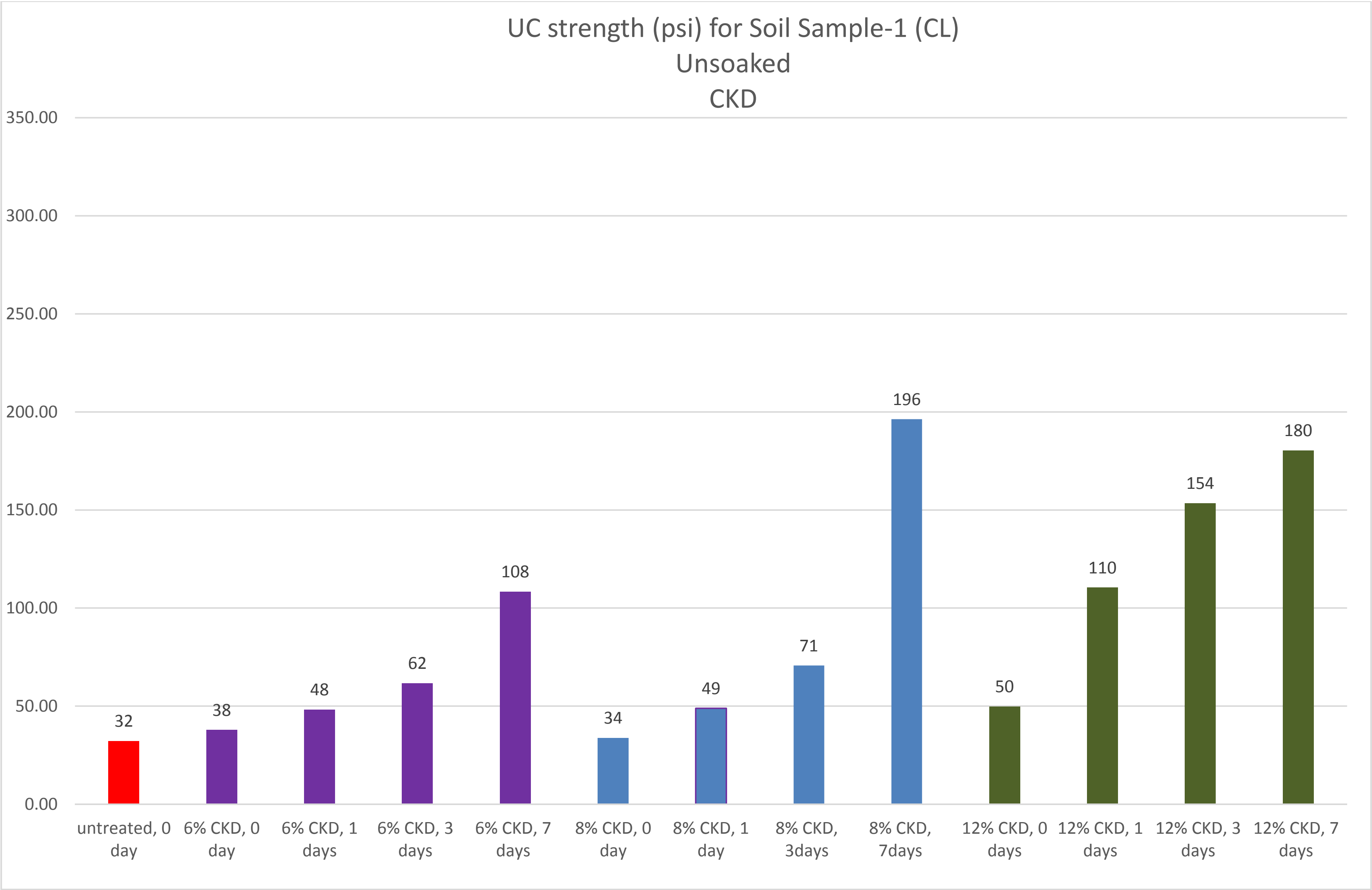


Figure 3.14: Comparison of Unsoaked UCS of Soil-1 (A-6) & CKD Mixes

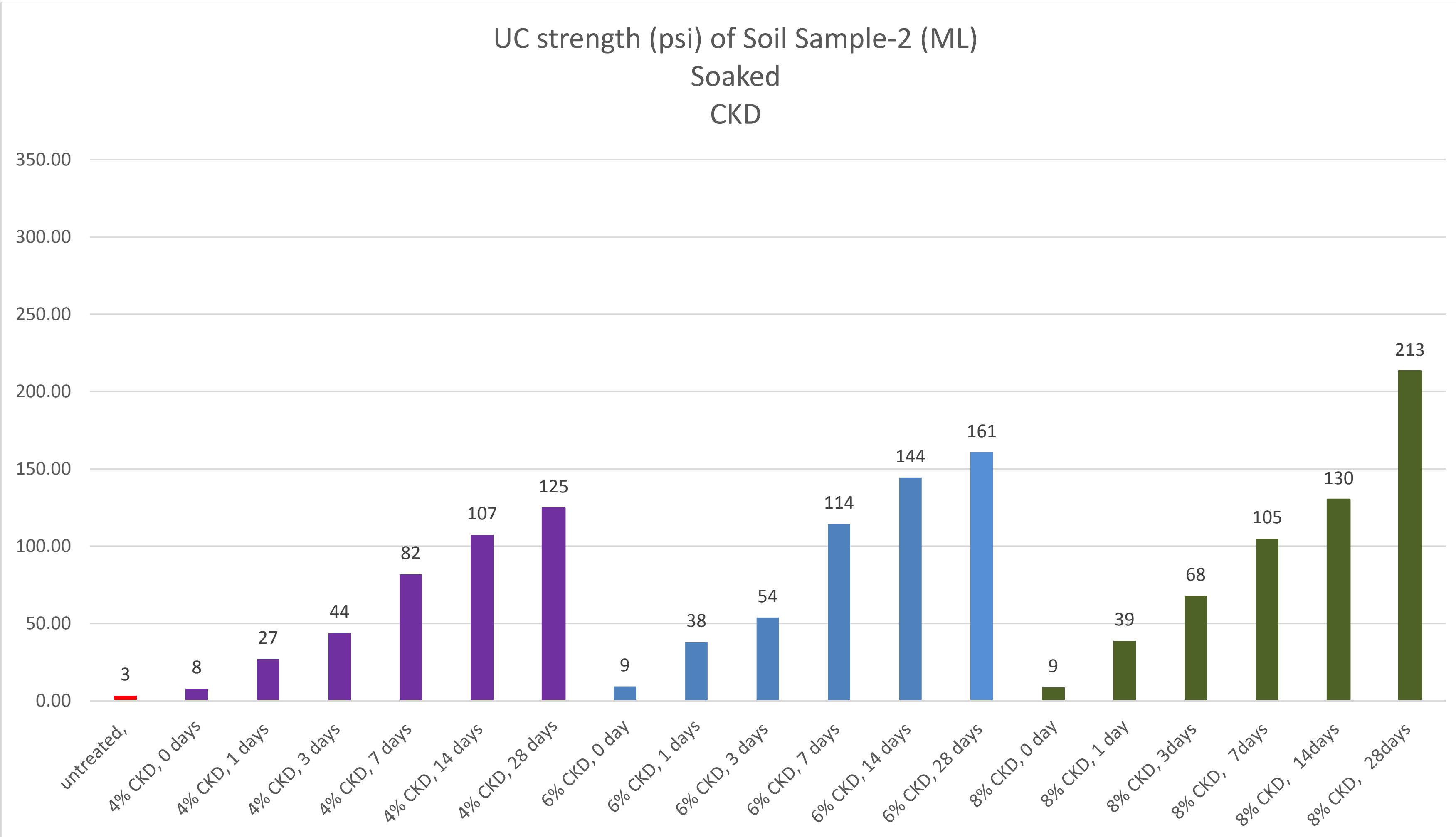


Figure 3.15: Comparison of Soaked UCS of Soil-2 (A-4) & CKD Mixes

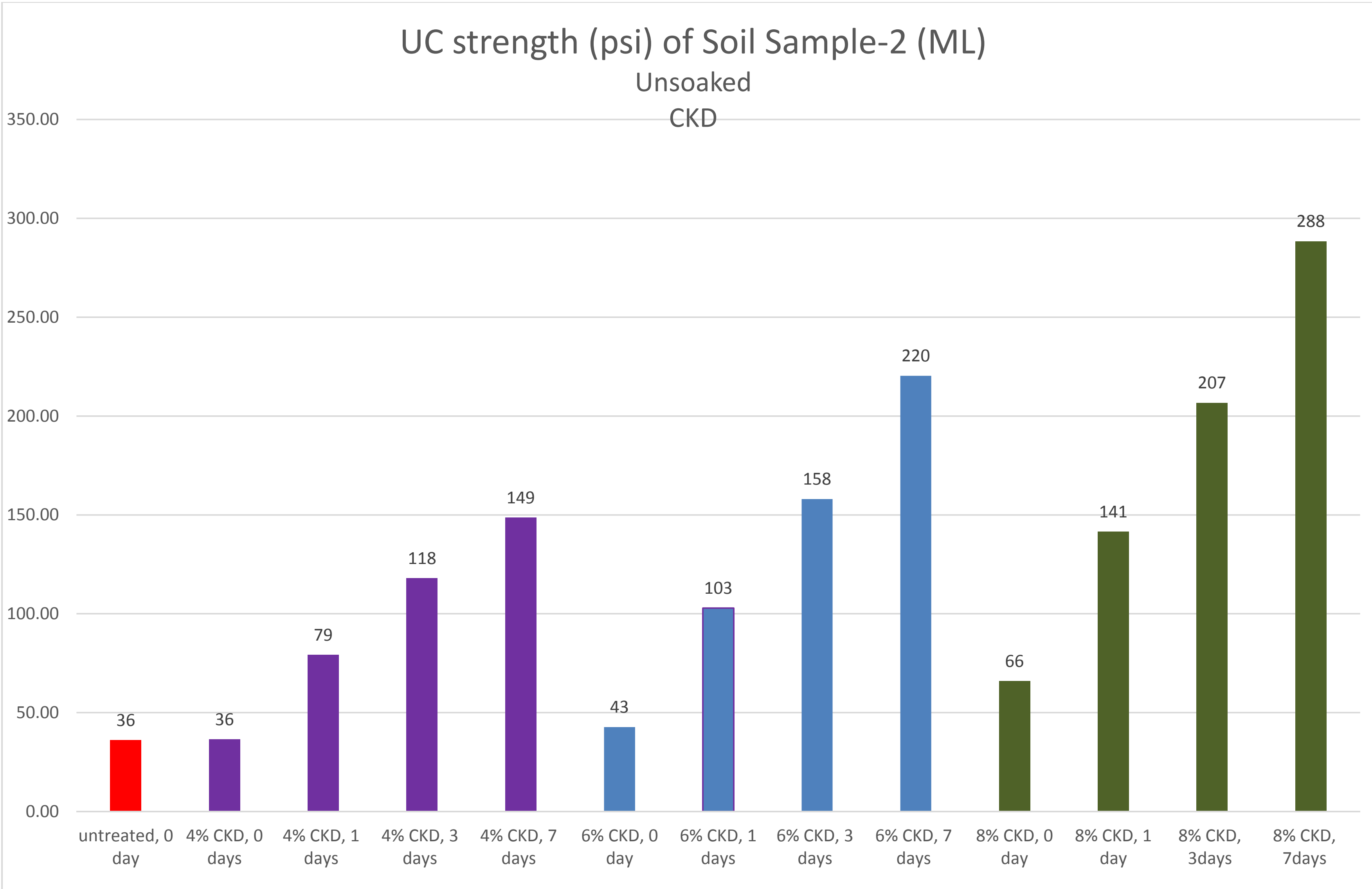


Figure 3.16: Comparison of Unsoaked UCS of Soil-2 (A-4) & CKD Mixes

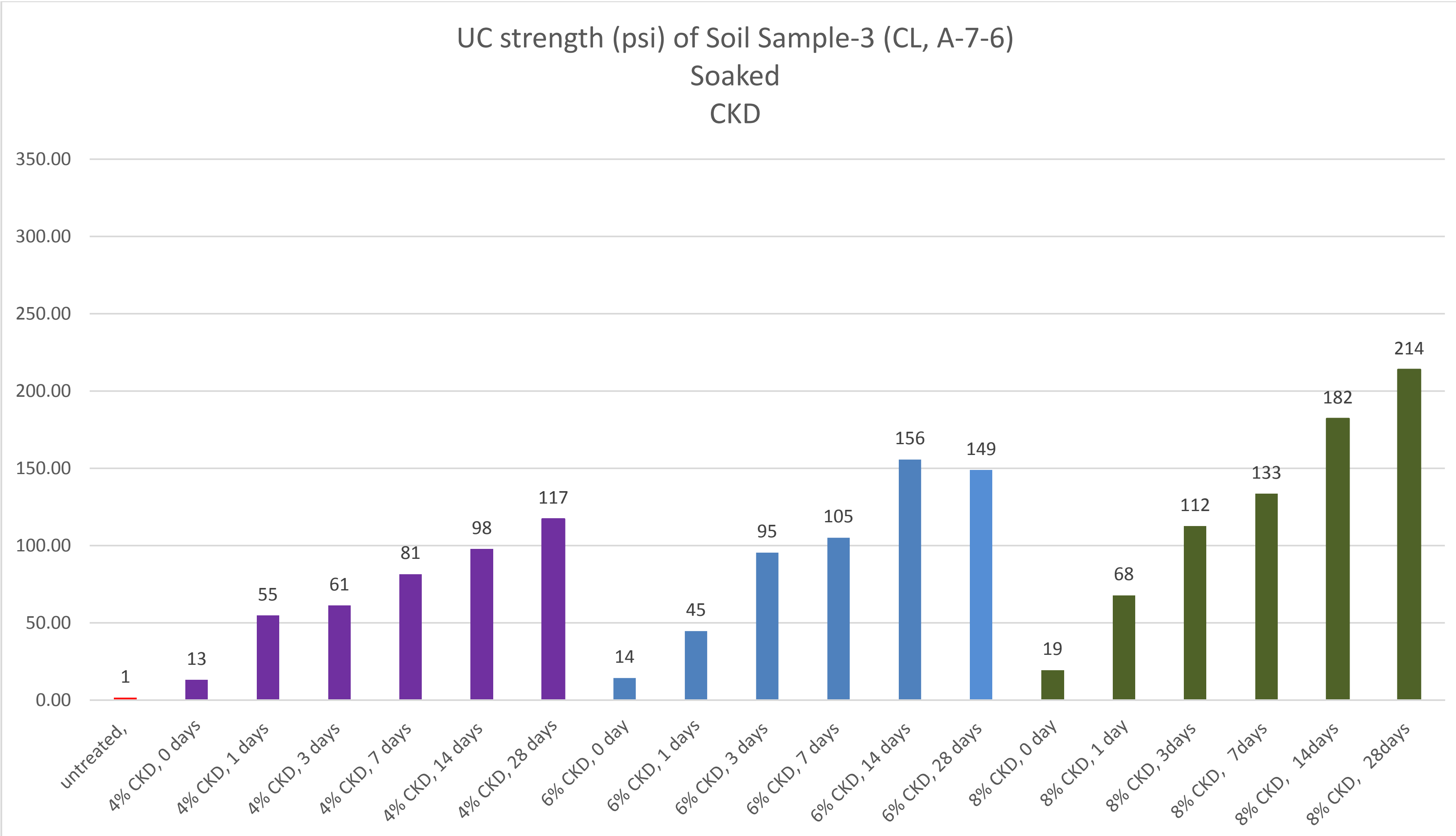


Figure 3.17: Comparison of Soaked UCS of Soil-3 (A-7-6) & CKD Mixes

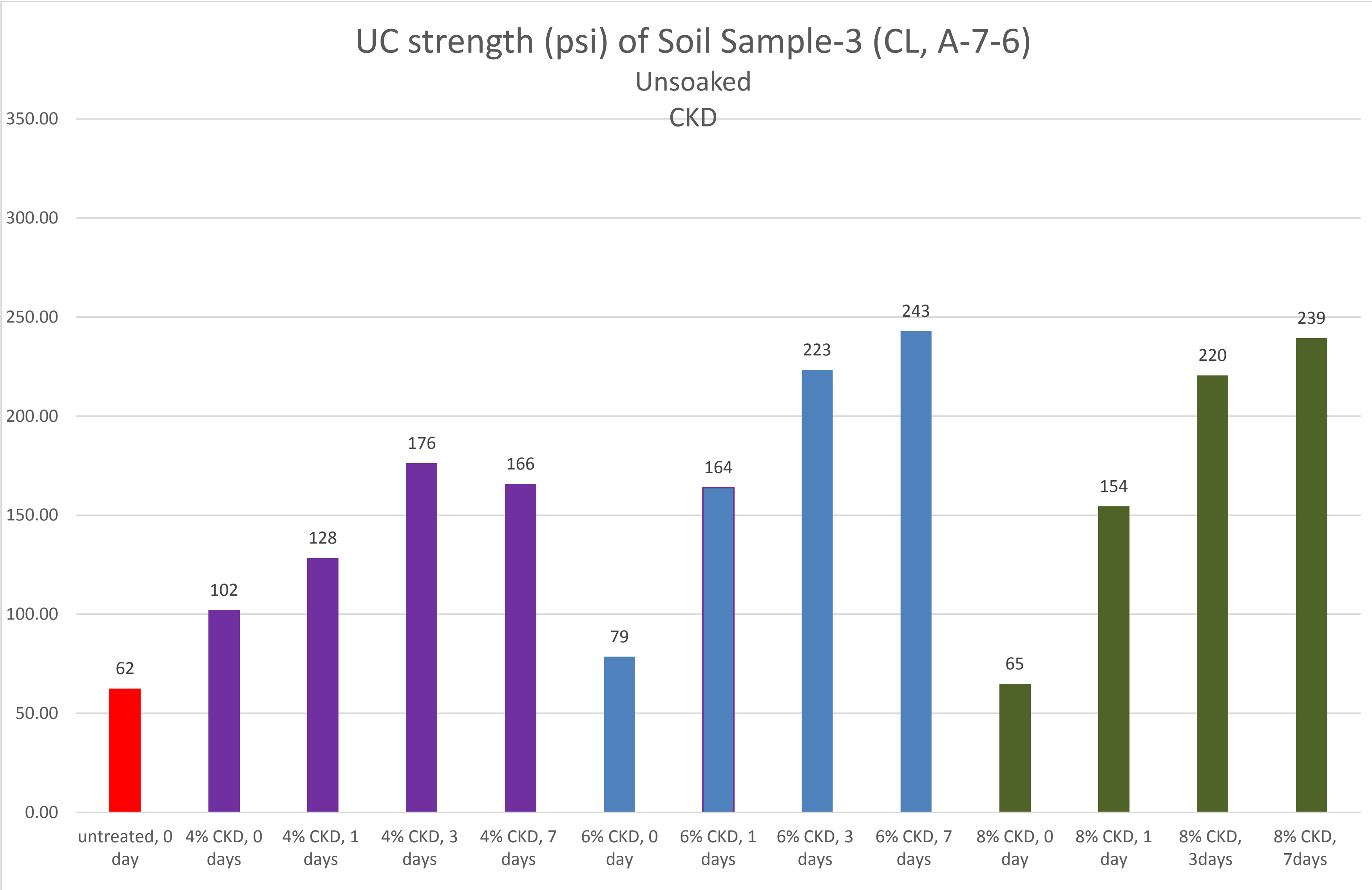


Figure 3.18: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & CKD Mixes

3.5.2 Concrete Fines (CF)

All three soils were mixed with different percentages of CF. The laboratory tests described in Section 3.3 were repeated on each proportion of the soil/CF mix. The percentages of CF mixed with the different soils are shown in Table 3.17.

Table 3.17: Percentages of CF Mixed with Different Soil Types.

Soil	CF (%)
Soil-1 (A-6)	4, 12, 25
Soil-2 (A-4)	4, 12, 25
Soil-3 (A-7-6)	4, 15, 25

3.5.2.1 Atterberg Limit Tests

Atterberg Limit Tests were performed according to ASTM D4318. The values of LL, PL, PI and soil classification of the stabilized soils of CF/Soil-1, CF/Soil-2 and CF/Soil-3 are shown in Tables 3.18, 3.19 and 3.20 respectively.

Table 3.18: Atterberg Limit Test Results of CF and Soil-1 (A-6) Mix

Percentage CF	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
4	Plastic Limit, PL	17.6	17.0	17.0	17.2	A-4
	Liquid Limit, LL	24.5	24.5	24.9	24.6	
	Plasticity Index, PI	6.8	7.6	7.9	7.4	
12	Plastic Limit, PL	19.5	20.1	22.5	20.7	A-6
	Liquid Limit, LL	40.4	40.5	40.9	40.5	
	Plasticity Index, PI	20.9	20.4	18.2	19.8	
25	Plastic Limit, PL	21.4	21.2	22.6	21.8	A-4
	Liquid Limit, LL	24.2	25.0	24.6	24.6	
	Plasticity Index, PI	2.8	3.8	2.0	2.8	

*Plasticity Index (PI) = LL-PL

The Atterberg Limit Test results for Soil-1 (A-6) and CF mixes showed a decrease in the Liquid Limit and the Plasticity Index for 4% CF and 25% CF. This AASHTO classification was changed to A-4. There was a slight increase in the Liquid Limit and the Plasticity Index for the 12% CF and Soil-1 mix. This classification remained the same as the untreated soil (A-6).

Table 3.19: Atterberg Limit Test Results of CF and Soil-2 (A-4) Mix

Percentage CF	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
4	Plastic Limit, PL	15.2	14.9	16.7	15.6	A-6
	Liquid Limit, LL	21.7	21.6	22.3	21.9	
	Plasticity Index, PI	6.5	6.8	5.6	6.3	
12	Plastic Limit, PL	13.3	15.5	15.7	14.9	A-4
	Liquid Limit, LL	21.1	21.4	21.5	21.3	
	Plasticity Index, PI	7.7	5.8	5.7	6.4	
25	Plastic Limit, PL	13.5	12.3	14.7	13.5	A-7-6
	Liquid Limit, LL	19.0	18.8	19.0	18.9	
	Plasticity Index, PI	5.5	6.4	4.3	5.4	

* Plasticity Index (PI) = LL - PL

Table 3.20: Atterberg Limit Test Results of CF and Soil-3 (A-7-6) Mix

Percentage CF	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
4	Plastic Limit, PL	24.9	24.2	24.5	24.5	A-6
	Liquid Limit, LL	37.4	37.7	37.9	37.7	
	Plasticity Index, PI	12.5	13.4	13.4	13.1	
12	Plastic Limit, PL	24.0	24.5	24.6	24.4	A-6
	Liquid Limit, LL	39.7	40.3	40.5	40.2	
	Plasticity Index, PI	15.7	15.8	16.0	15.8	
25	Plastic Limit, PL	25.2	23.6	24.6	24.5	A-6
	Liquid Limit, LL	39.4	39.7	40.9	40.0	
	Plasticity Index, PI	14.2	16.1	16.3	15.5	

*Plasticity Index (PI) = LL - PL

The Liquid Limit and the Plasticity Index both decreased when CF was mixed with Soil-3 (A-7-6). At all percentages of CF, the classification changed to A-6.

3.5.2.2 Standard Proctor Test

Performed according to ASTM D698, the results of Standard Proctor Test of CF/soil mixes are shown in Tables 3.21, 3.22 and 3.23.

Table 3.21: MDD and OMC of Soil-1 (A-6) Mixed with CF

Test Number	4% CF		12% CF		25% CF	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	108.0	15.0	108.1	15.8	105.4	13.4
2	108.4	15.5	109.3	15.9	107.0	14.0
3	105.4	16.7	109.1	15.1	108.9	14.8
Average	107.27	15.7	108.8	15.6	107.1	14.1

Table 3.22: MDD and OMC of Soil-2 (A-4) Mixed with CF

Test Number	4% CF		12% CF		25% CF	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	114.6	13.51	116	12.67	115.72	12.07
2	114.7	12.95	115.29	12.67	117.63	11.32
3	114.18	13.18	113.73	12.38	115.95	11.28
Average	114.49	13.22	115.01	12.57	116.43	11.56

Table 3.23: MDD and OMC of Soil-3 (A-7-6) Mixed with CF

Test Number	4% CF		15% CF		25% CF	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	100.25	21.02	100.13	19.25	99.81	18.69
2	99.87	21.47	100.17	19.53	99.98	19.21
3	98.55	22	99.54	19.56	99.66	20.1
Average	99.55	21.5	99.94	19.45	99.82	19.33

3.5.2.3 Calibration of Harvard Miniature Compaction Apparatus

The summaries of the calibration of the Harvard Miniature Compaction Apparatus for CF mixed with Soil-1, Soil-2 and Soil-3 are shown in Tables 3.24, 3.25 and 3.26 respectively.

Table 3.24: Harvard Miniature Compactor Apparatus Calibration for Soil-1 (A-6) Mixed with CF

Percentage CF	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
4	37.5	5	20	15.0
12	37.5	5	20	15.6
25	37.5	5	20	14.3

Table 3.25: Harvard Miniature Compactor Apparatus Calibration for Soil-2 (A-4) Mixed with CF

Percentage CF	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
4	37.5	5	20	12.2
12	37.5	5	20	11.5
25	37.5	5	20	10.8

Table 3.26: Harvard Miniature Compactor Apparatus Calibration for Soil-3 (A-7-6) Mixed with CF

Percentage CF	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
4	37.5	5	20	19.0
15	37.5	5	20	19.3
25	37.5	5	20	19.2

3.5.2.4 Unconfined Compressive Strength (UCS) Test

The CF mixed samples did not achieve the necessary 50 psi increase over the untreated soil needed for stabilization purposes. Changes in the UCS for the soaked samples with the respective curing period are shown in Figures 3.19, 3.21 and 3.23. Changes in the UCS of the unsoaked samples are shown in Figures 3.20, 3.22 and 3.24.

While a slight increase in USC was observed over the untreated soil, none of the CF mixed soaked or unsoaked samples achieved a 50 psi increase in UCS. The increase in percentage of CF did not equate to an increase in UCS. Hence, CF was not selected for stabilization or modification.

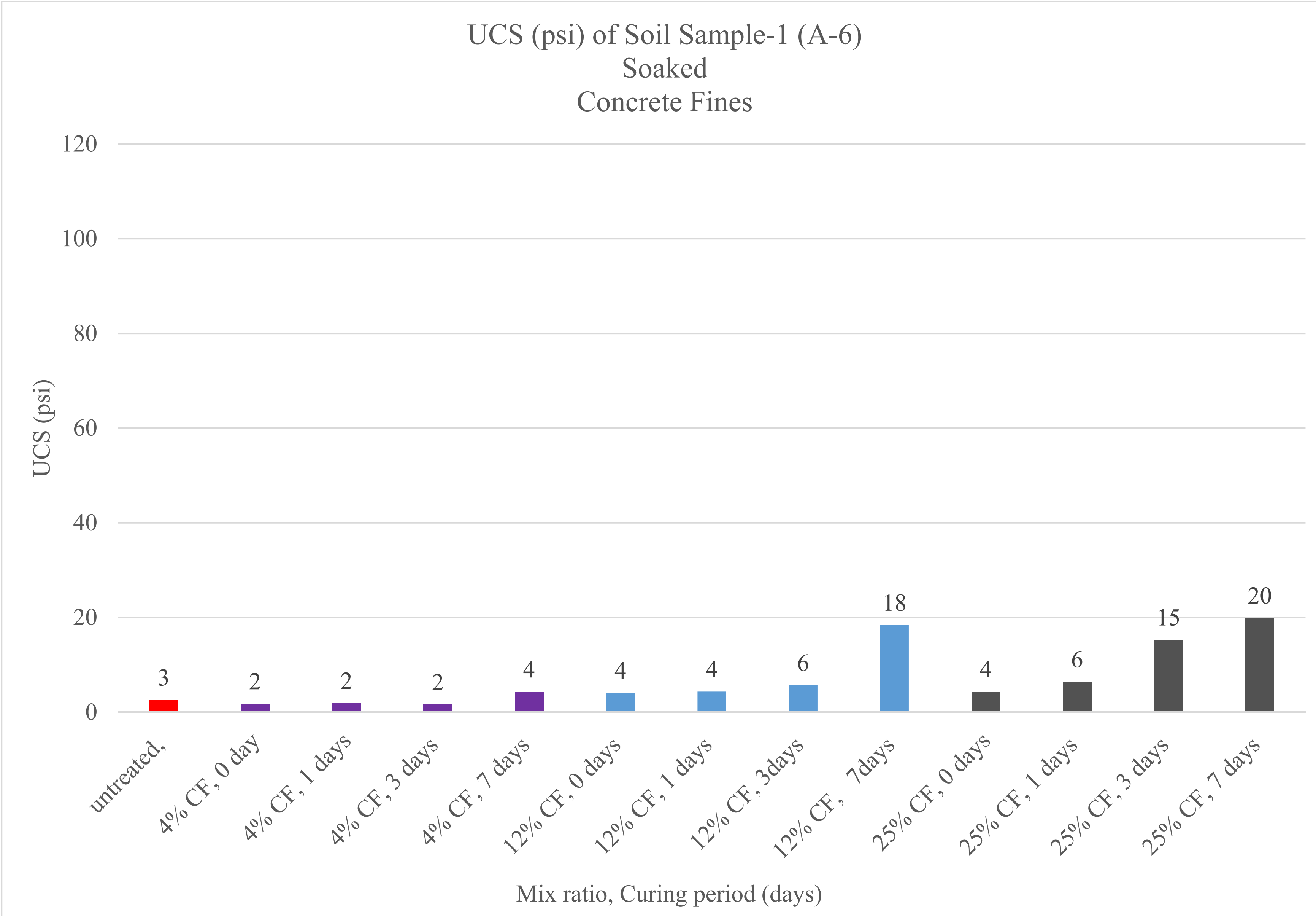


Figure 3.19: Comparison of Soaked UCS of Soil-1 (A-6) & CF Mixes

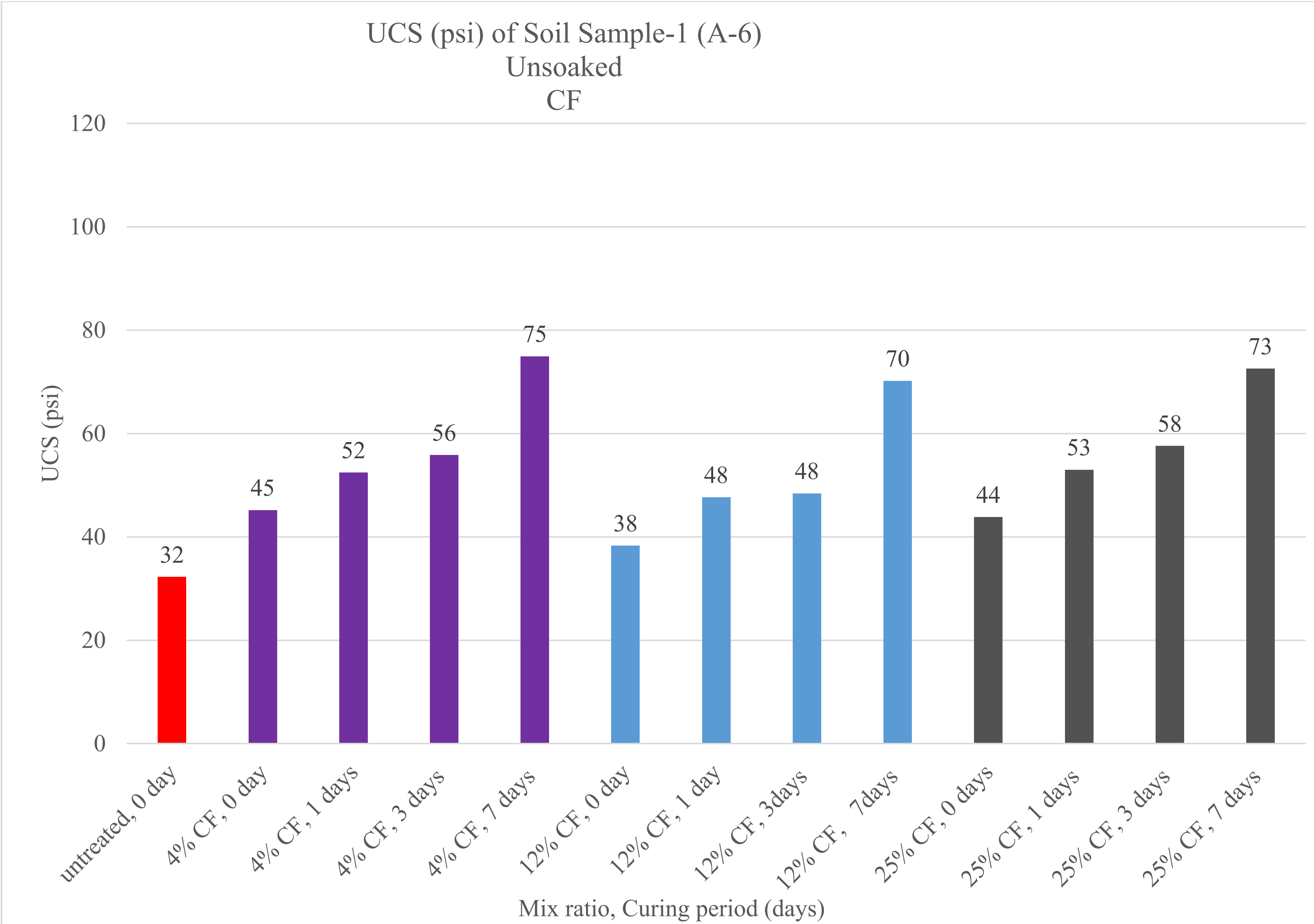


Figure 3.20: Comparison of Unsoaked UCS of Soil-1 (A-6) & CF Mixes

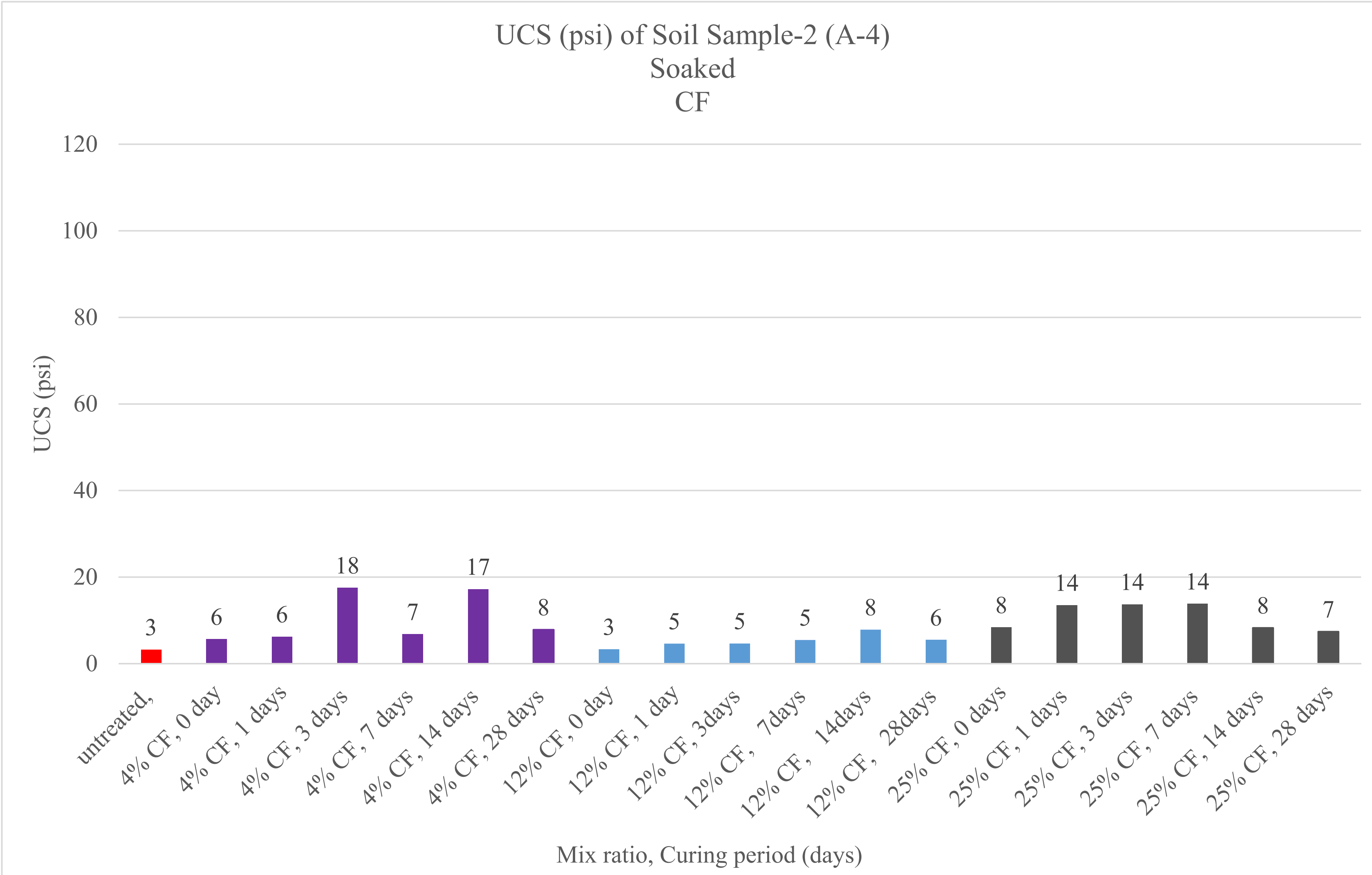


Figure 3.21: Comparison of Soaked UCS of Soil-2 (A-4) & CF Mixes

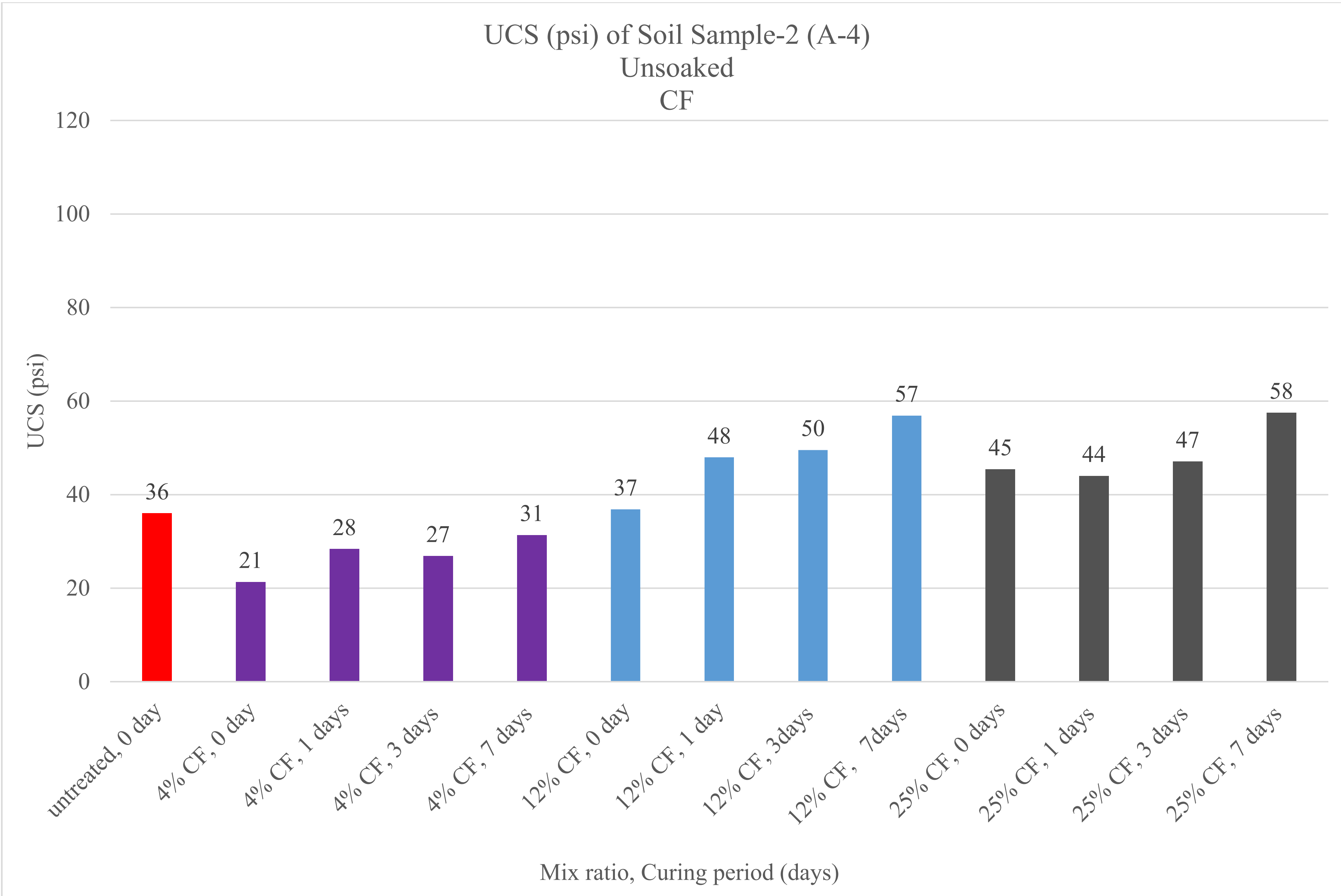


Figure 3.22: Comparison of Unsoaked UCS of Soil-2 (A-4) & CF Mixes

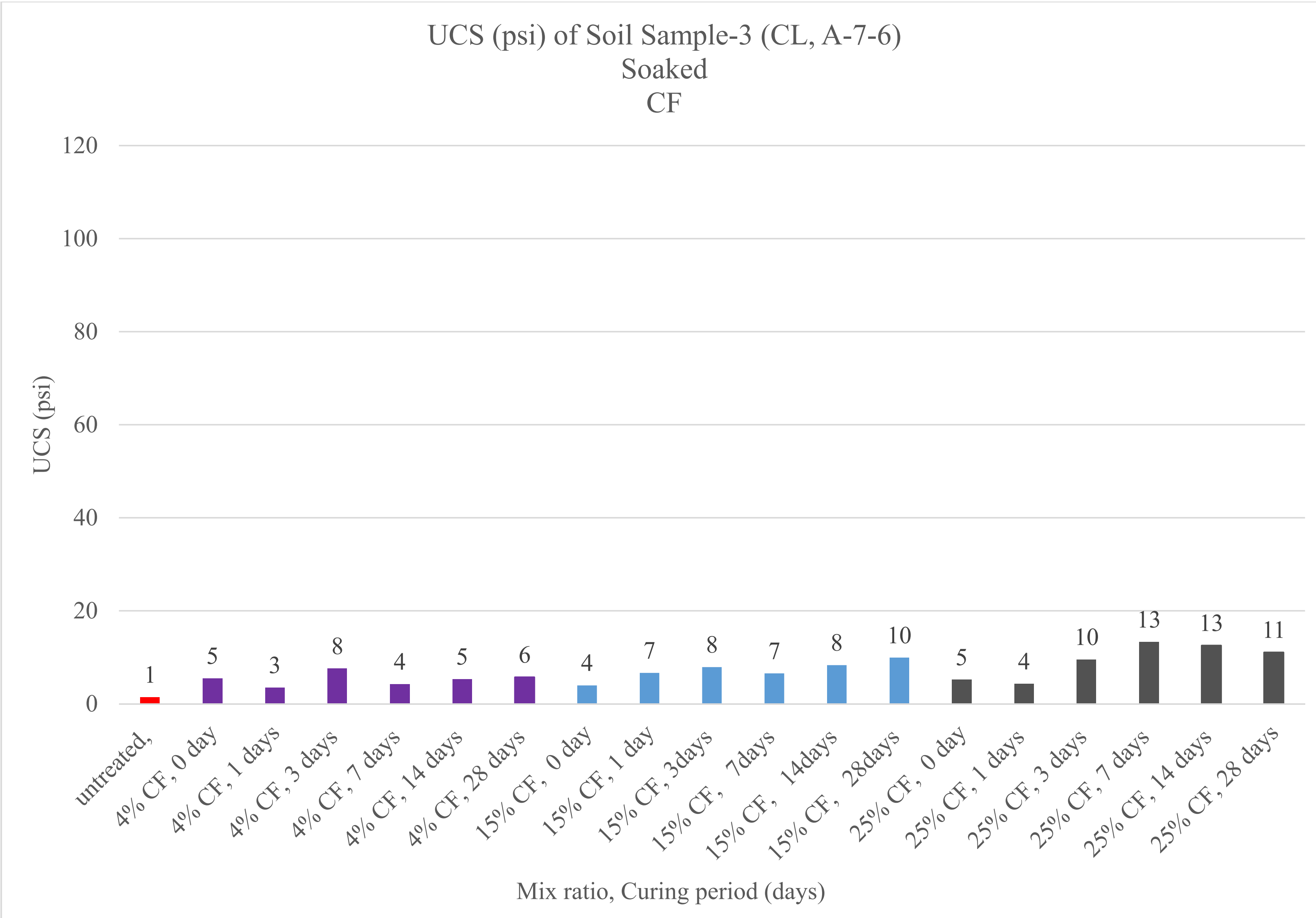


Figure 3.23: Comparison of Soaked UCS of Soil-3 (A-7-6) & CF Mixes

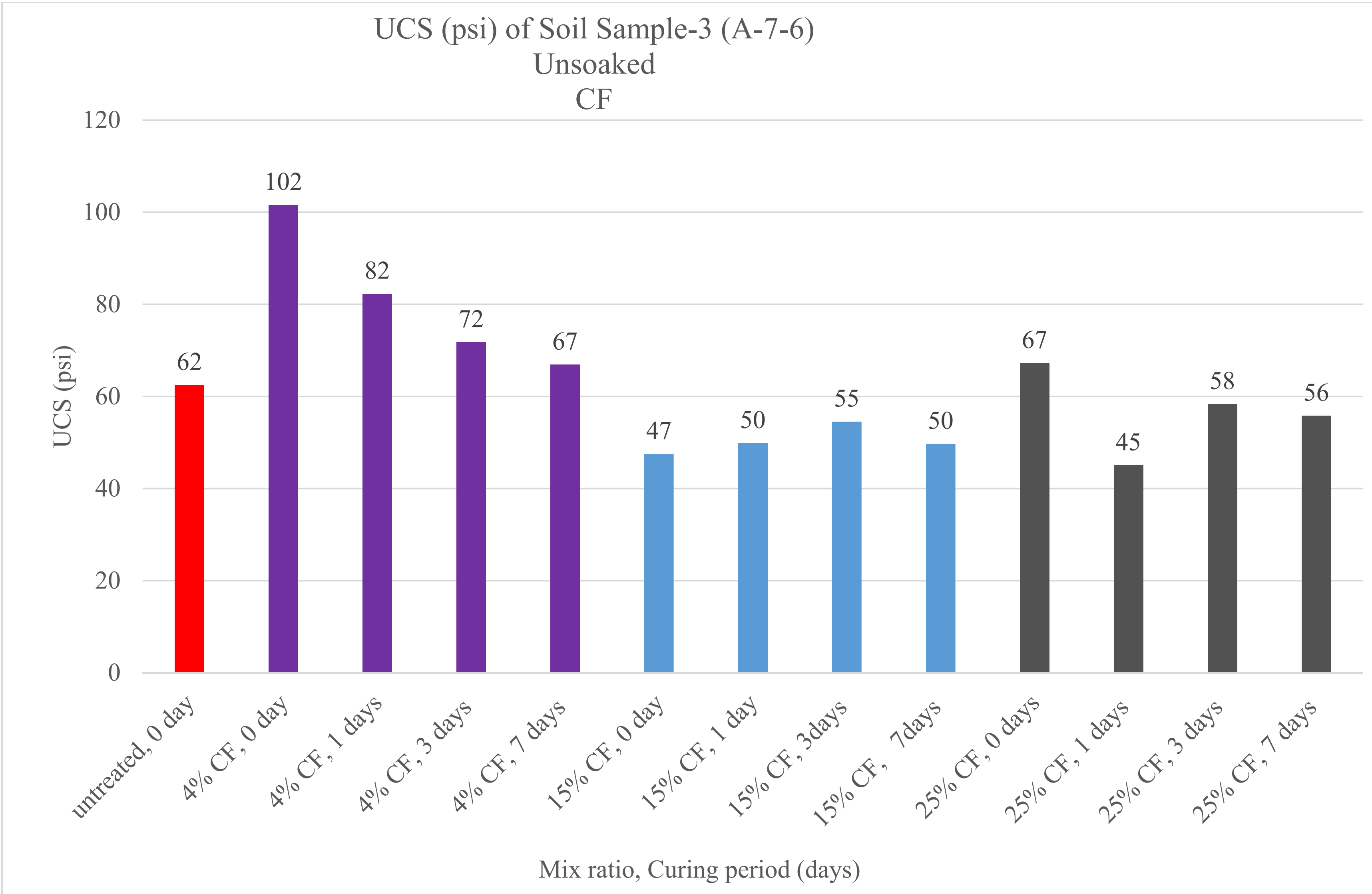


Figure 3.24: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & CF Mixes

3.5.3 Fly Ash (FA)

All three soils were mixed with different percentages of FA. The tests described in Section 3.3 were repeated on each proportions of soil/FA mixes. The percent of FA mixed with different types of soils is shown in Table 3.27. The CaO content of FA is desirable for its self-cementing properties. Usually Class C Fly Ash, which contains more than 20% CaO, is a self-cementing material. Fly Ash used for these tests contained 21% CaO, making it a marginal Class C Fly Ash.

Table 3.27: Percentages of FA Mixed with Different Soil Types

Soil	FA (%)
Soil-1 (A-6)	10, 15, 25
Soil-2 (A-4)	10, 15, 25
Soil-3 (A-7-6)	10, 15, 25

3.5.3.1 Atterberg Limit Test

The Atterberg Limit Test results of Soil-1, Soil-2 and Soil-3 stabilized with FA and the resultant soil classification after stabilization are shown in Tables 3.28, 3.29 and 3.30 respectively.

Table 3.28: Atterberg Limit Test Results of FA and Soil-1 (A-6) Mix

Percentage FA	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
10	Plastic Limit, PL	18.9	19.5	19.5	19.3	A-4
	Liquid Limit, LL	28.3	28.6	28.9	28.6	
	Plasticity Index, PI	9.4	9.0	9.3	9.3	
15	Plastic Limit, PL	16.5	17.6	18.0	17.3	A-6
	Liquid Limit, LL	27.1	28.3	28.9	28.1	
	Plasticity Index, PI	10.6	10.8	10.9	10.8	
25	Plastic Limit, PL	19.9	21.8	22.3	21.3	A-4
	Liquid Limit, LL	30.2	29.2	29.1	29.5	
	Plasticity Index, PI	10.3	7.4	6.7	8.2	

* Plasticity Index (PI) = LL - PL

The Atterberg Limit Test results of Soil-1 (A-6) and all FA mixes showed that the Liquid Limit and the Plasticity Index both decreased when FA was mixed with soil. The AASHTO classification of 10% FA and 25% FA-treated Soil-1 changed to A-4. In the case of 15% FA and Soil-1 (A-6) mix, the classification remained the same.

Table 3.29: Atterberg Limit Test Results of FA and Soil-2 (A-4) Mix

Percentage FA	Test*	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
10	Plastic Limit, PL	14.3	14.5	13.8	14.2	A-4
	Liquid Limit, LL	19.6	19.5	19.3	19.5	
	Plasticity Index, PI	5.2	5.0	5.5	5.2	
15	Plastic Limit, PL	15.3	19.0	18.4	17.6	A-4
	Liquid Limit, LL	19.5	19.1	19.1	19.2	
	Plasticity Index, PI	4.2	0.1	0.7	1.7	
25	Plastic Limit, PL	12.6	13.4	14.3	13.4	A-4
	Liquid Limit, LL	24.5	23.6	24.0	24.0	
	Plasticity Index, PI	11.9	10.1	9.7	10.6	

* Plasticity Index (PI) = LL - PL

The Liquid Limit increased slightly when FA was mixed with Soil-2 (A-4). The Plasticity Index change was irregular. The addition of FA did not alter the original AASHTO classification.

Table 3.30: Atterberg Limit Test Results of FA and Soil-3 (A-7-6) Mix

Percentage FA	Test	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
10	Plastic Limit, PL	26.0	24.5	23.8	24.8	A-7-6
	Liquid Limit, LL	40.9	41.5	40.7	41.0	
	Plasticity Index, PI	14.9	17.0	16.9	16.3	
15	Plastic Limit, PL	26.1	25.1	24.0	25.1	A-7-6
	Liquid Limit, LL	41.4	41.2	41.2	41.3	
	Plasticity Index, PI	15.3	16.2	17.2	16.2	
25	Plastic Limit, PL	31.5	32.4	32.1	32.0	A-7-5
	Liquid Limit, LL	45.1	43.5	43.0	43.9	
	Plasticity Index, PI	26.0	24.5	23.8	24.8	

* Plasticity Index (PI) = LL - PL

The Liquid Limit and the Plasticity Index both decreased when FA was mixed with Soil-3 (A-7-6). At lower percentages (10% FA & 15% FA), the AASHTO classification remained the same as the untreated soil (A-7-6). At the higher percentage of 25% FA, the AASHTO classification changed to A-7-5.

3.5.3.2 Standard Proctor Test

Results of Standard Proctor Test of FA/soil mixes performed according to ASTM D698 are shown in Tables 3.31, 3.32 and 3.33.

Table 3.31: MDD and OMC of Soil-1 (A-6) Mixed with FA

Test Number	10% FA		15% FA		25% FA	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	106.69	15.59	112.20	10.44	105.26	13.34
2	106.67	16.76	113.13	10.01	107.51	13.32
3	105.95	16.17	112.84	10.98	106.24	13.22
Average	106.44	16.17	112.72	10.48	106.34	13.29

Table 3.32: MDD and OMC of Soil-2 (A-4) Mixed with FA

Test Number	10% FA		15% FA		25% FA	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	116.74	11.94	114.93	12.56	115.33	12.52
2	115.45	12.87	116.02	13.24	114.43	11.50
3	116.11	12.25	114.09	11.88	114.63	12.93
Average	116.10	12.35	115.01	12.56	114.80	12.32

Table 3.33: MDD and OMC of Soil-3 (A-7-6) Mixed with FA

Test Number	10% FA		15% FA		25% FA	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	99.79	20.89	98.9	21.03	101.56	19.13
2	99.16	21.09	98.87	20.78	101.42	18.88
3	99.01	20.86	98.80	20.71	101.82	18.12
Average	99.32	20.95	98.86	20.84	101.60	18.71

3.5.3.3 Calibration of Harvard Miniature Compaction Apparatus

The summaries of the calibration of the Harvard Miniature Compaction Apparatus for FA mixed with Soil-1, Soil-2 and Soil-3 are shown in Tables 3.34, 3.35 and 3.36 respectively.

Table 3.34: Harvard Miniature Compactor Apparatus Calibration for Soil-1 (A-6) Mixed with FA

Percentage FA	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
10	37.5	5	20	17.6
15	37.5	5	20	13.3
25	37.5	5	20	14.1

Table 3.35: Harvard Miniature Compactor Apparatus Calibration for Soil-2 (A-4) Mixed with FA

Percentage FA	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
10	37.5	5	20	10.4
15	37.5	5	20	10.2
25	37.5	5	20	11.7

Table 3.36: Harvard Miniature Compactor Apparatus Calibration for Soil-3 (A-7-6) Mixed with FA

Percentage FA	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
10	37.5	5	20	21.51
15	37.5	5	20	20.73
25	37.5	5	20	19.20

3.5.3.4 Unconfined Compressive Strength (UCS) Test

The changes in UCS for the cured and soaked samples are shown in Figures 3.25, 3.27 and 3.29. The unsoaked sample results are shown Figures 3.26, 3.28 and 3.30.

The soaked UCS values of the FA-treated Soil-1 and Soil-2 samples are less than 50 psi of the required strength gain to be considered for stabilization purposes. However, the soaked UCS value of the 15% FA and Soil-3 mix was more than 50 psi after seven days over the untreated Soil-3. Hence, 15% FA was selected for stabilization of Soil-3.

The unsoaked UCS results of 15% FA in Soil-1 and 25% FA in Soil-2 showed more than 50 psi strength gain in three days. Therefore, 15% FA and 25% FA were selected for short-term modification of Soil-1 and Soil-2 respectively.

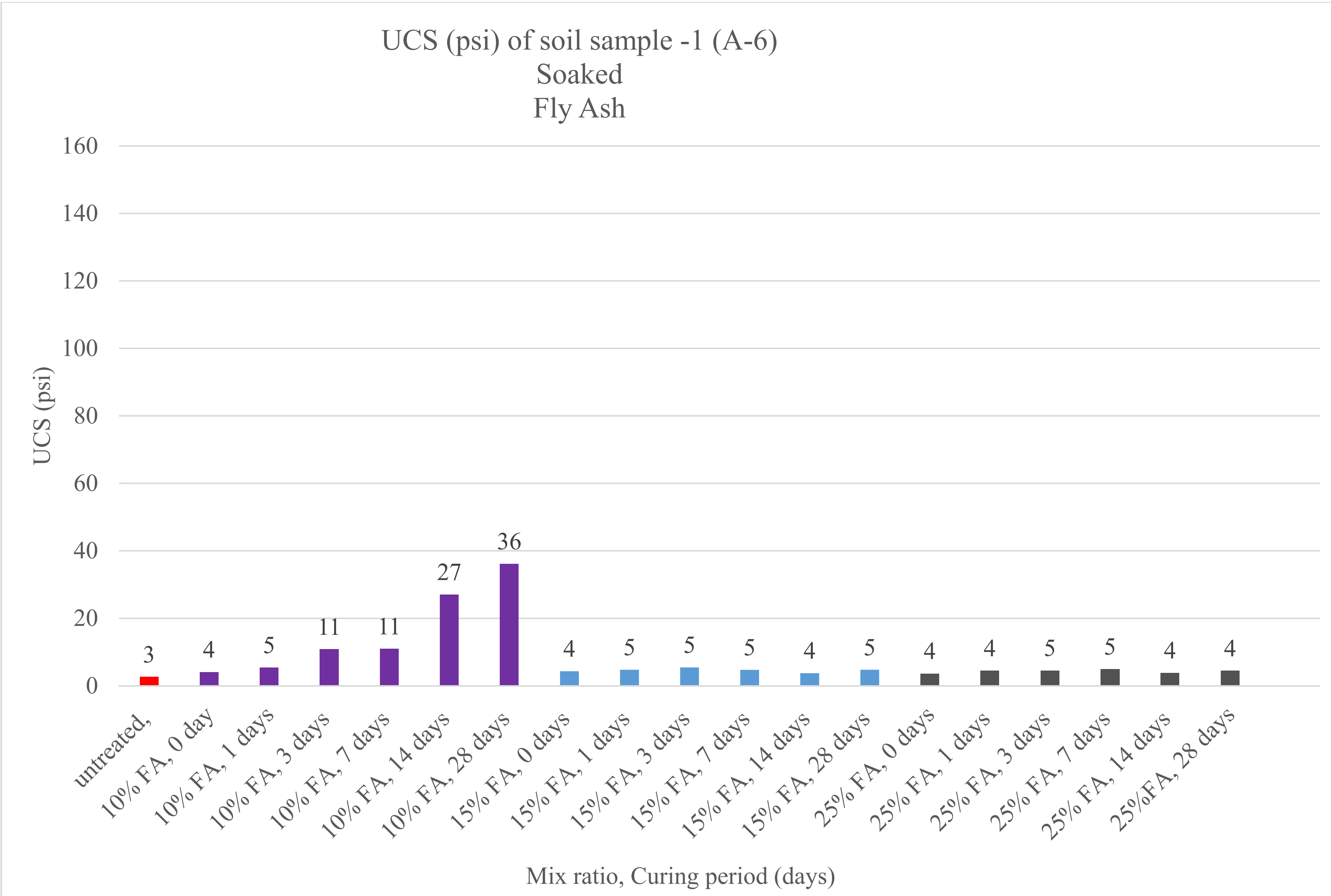


Figure 3.25: Comparison of Soaked UCS of Soil-1 (A-6) & FA Mixes

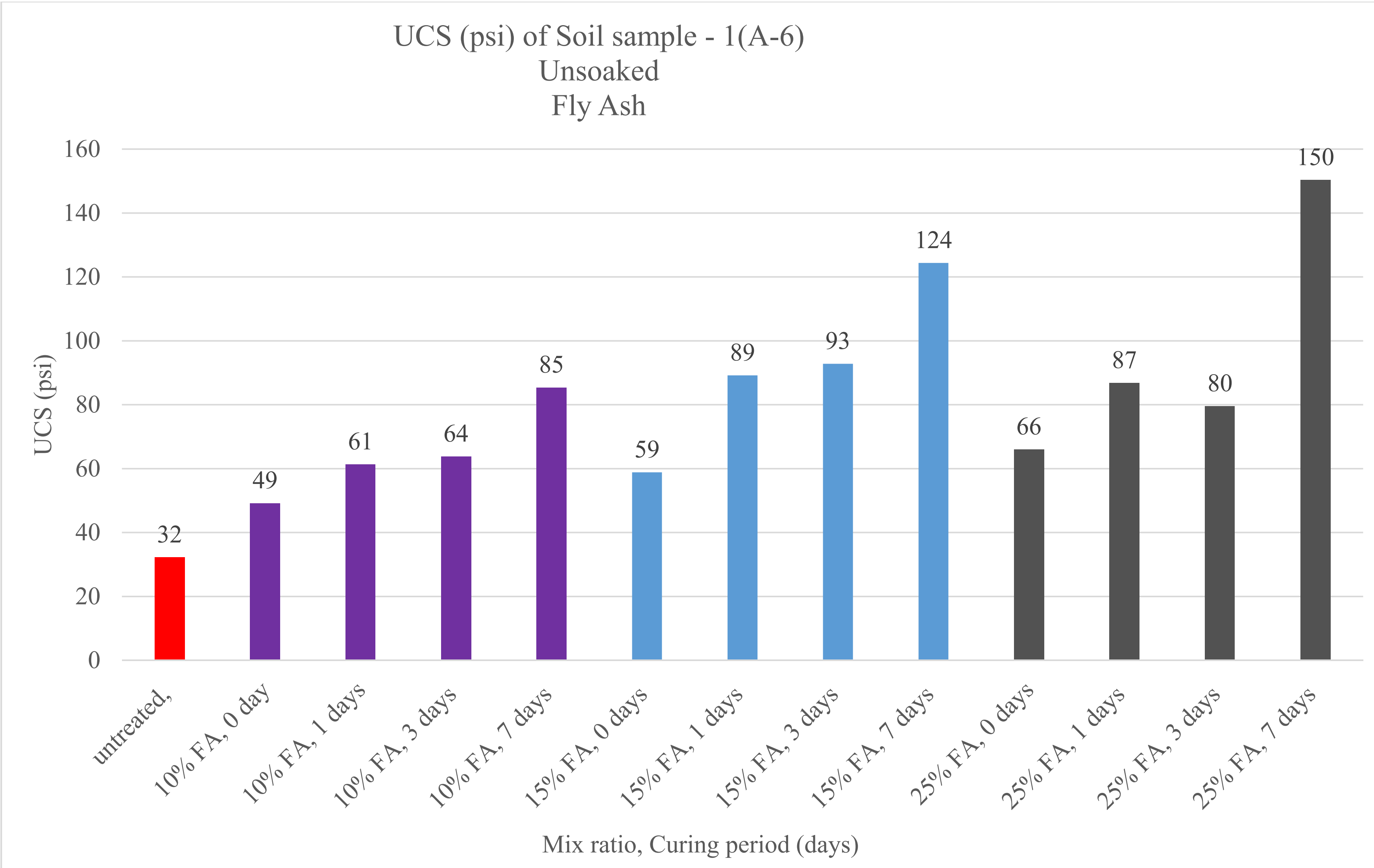


Figure 3.26: Comparison of Unsoaked UCS of Soil-1 (A-6) & FA Mixes

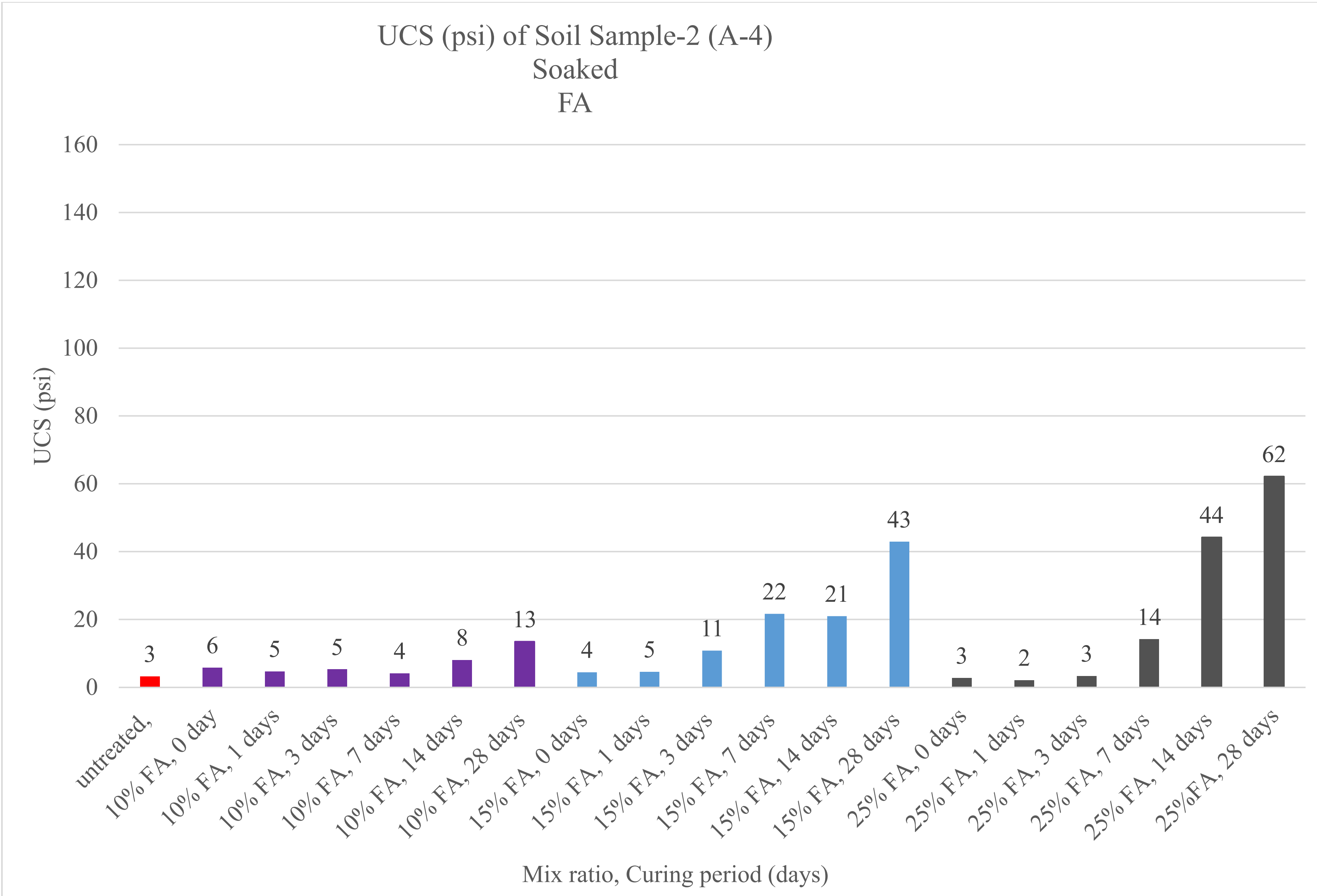


Figure 3.27: Comparison of Soaked UCS of Soil-2(A-4) & FA Mixes

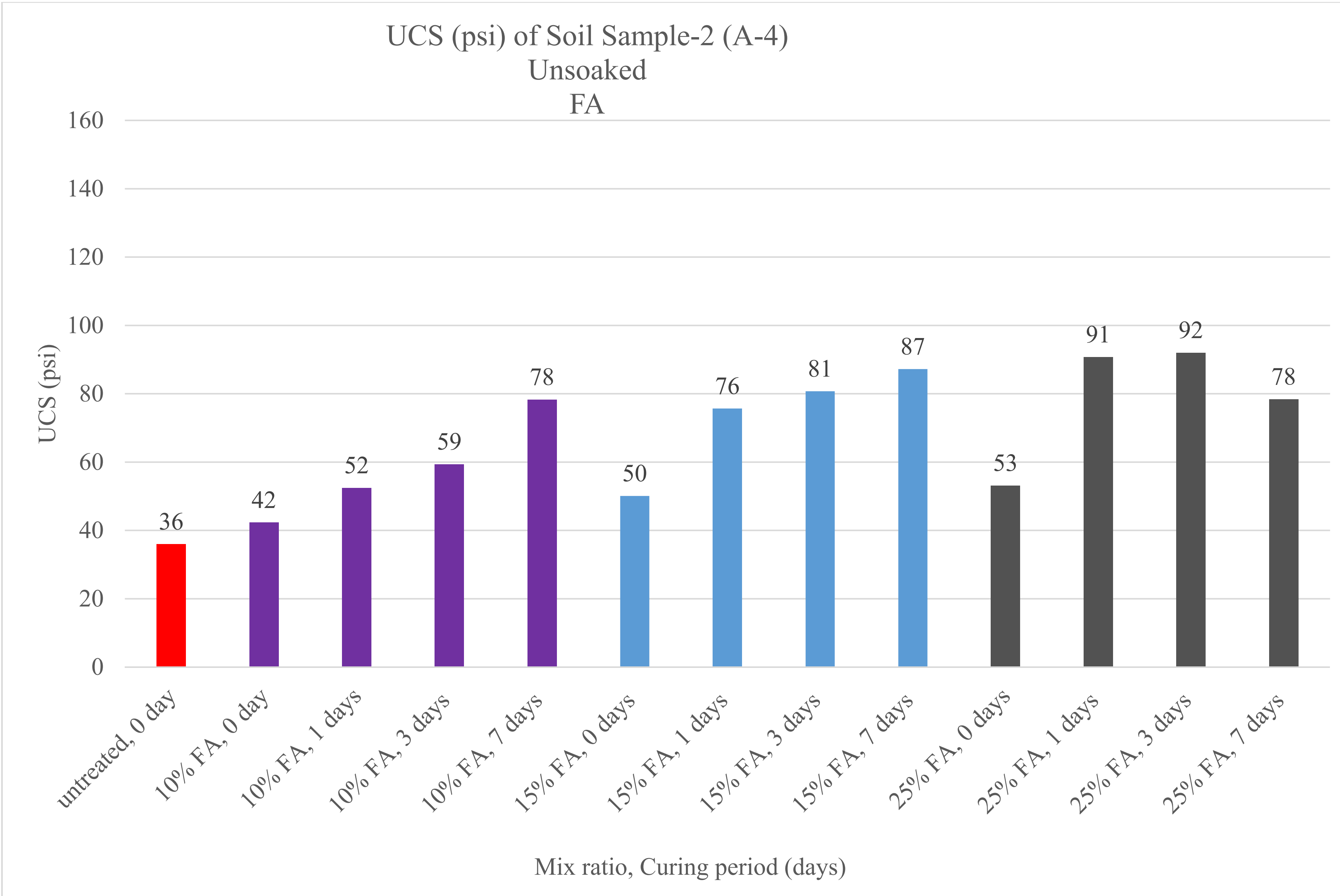


Figure 3.28: Comparison of Unsoaked UCS of Soil-2 (A-4) & FA Mixes

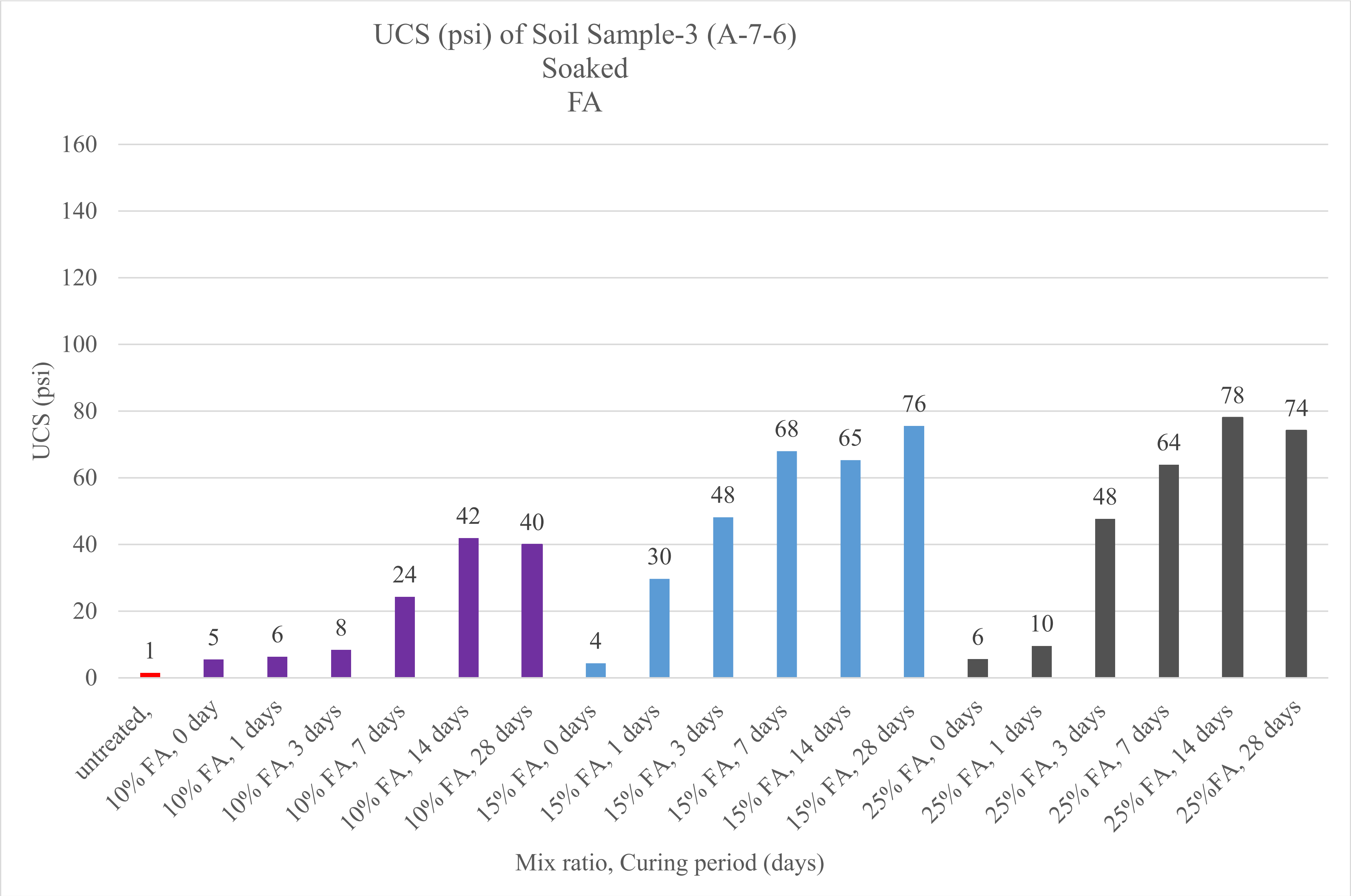


Figure 3.29: Comparison of Soaked UCS of Soil-3 (A-7-6) & FA Mixes

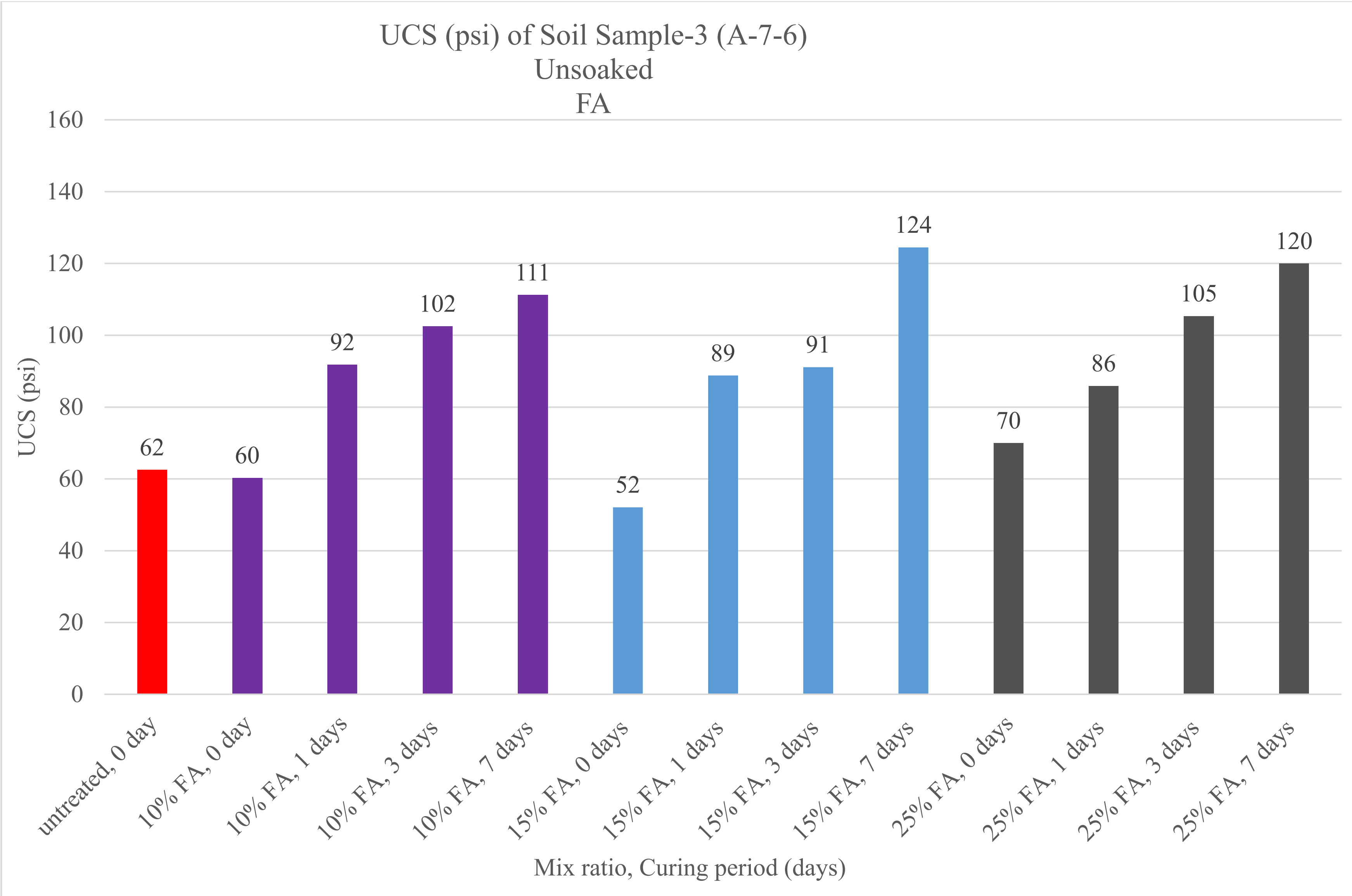


Figure 3.30: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & FA Mixes

3.5.4 Lime Kiln Dust and Fly Ash Mix (LKD/FA)

As noted above, the FA used for this test had a low percentage of available CaO (21%). To improve the self-cementing properties of the FA, LKD was added to provide additional CaO. The trial percentages of LKD/FA mixed with different types of soils are shown in Table 3.37.

Table 3.37: Percentages of LKD/FA Mixed with Different Soil Types

Soil	LKD (%)/FA (%)
Soil-1 (A-6)	2/5, 3/9, 5/15
Soil-2 (A-4)	2/5, 2/8
Soil-3 (A-7-6)	2/5, 2/8, 3/9

3.5.4.1 Atterberg Limit Test

The Atterberg Limit Test results of the LKD/FA and Soil-1, Soil-2 and Soil-3 mixes and the soil classifications of these mixed soils are shown in Tables 3.38, 3.39 and 3.40 respectively [Plasticity Index (PI) = Liquid Limit (LL) – Plastic Limit (PL)].

Table 3.38: Atterberg Limit Test Results of LKD/FA and Soil-1 (A-6) Mix

Percentage LKD/FA	Test	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
2/5	Plastic Limit, PL	25.3	26.6	31.5	27.8	A-6
	Liquid Limit, LL	33.4	33.3	32.5	33.1	
	Plasticity Index, PI	8.1	6.7	1.5	5.3	
3/9	Plastic Limit, PL	23.5	21.6	22.1	22.4	A-4
	Liquid Limit, LL	32.9	33.1	33.6	33.2	
	Plasticity Index, PI	9.4	11.5	11.5	10.8	
5/15	Plastic Limit, PL	22.4	21.2	23.0	22.2	A-6
	Liquid Limit, LL	32.7	32.6	32.4	32.5	
	Plasticity Index, PI	10.3	11.4	9.4	10.4	

Table 3.39: Atterberg Limit Test Results of LKD/FA and Soil-2 (A-4) Mix

Percentage LKD/FA	Test	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
2/5	Plastic Limit, PL	26.2	19.8	18.9	21.6	A-4
	Liquid Limit, LL	25.6	25.6	25.4	25.5	
	Plasticity Index, PI	0.0	5.8	6.5	3.9	
3/8	Plastic Limit, PL	19.7	19.4	18.1	19.1	A-4
	Liquid Limit, LL	25.6	25.5	25.4	25.5	
	Plasticity Index, PL	5.9	6.1	7.4	6.4	

Table 3.40: Atterberg Limit Test Results of LKD/FA and Soil-3 (A-7-6) Mix

Percentage LKD/FA	Test	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
2/5	Plastic Limit, PL	24.3	25.9	24.4	24.9	A-6
	Liquid Limit, LL	39.2	39.2	39.2	39.2	
	Plasticity Index, PI	14.9	13.2	14.8	14.3	
2/8	Plastic Limit, PL	22.5	25.5	27.2	25.1	A-6
	Liquid Limit, LL	38.7	38.6	39.1	38.8	
	Plasticity Index, PI	16.2	13.1	11.9	13.7	
3/9	Plastic Limit, PL	24.8	26.6	25.9	25.8	A-6
	Liquid Limit, LL	39.2	39.0	38.8	39.0	
	Plasticity Index, PI	14.4	12.4	12.9	13.2	

Soil-1 (A-6), at all percentages of LKD/FA, showed a slight increase in Liquid Limit and decrease in Plasticity Index. Soil-1 (A-6) at 2% LKD/5% FA and 5% LKD/15% FA retained the same classification as the untreated soil. At 3% LKD/9% FA, Soil-1 changed to AASHTO classification to A-4.

Classification remained unchanged when LKD and FA were mixed with Soil-2 (A-4). Soil-3 (A-7-6) becomes A-6 when treated with both LKD and FA due to decrease of the Liquid Limit and the Plasticity Index.

3.5.4.2 Standard Proctor Test

Results of the Standard Proctor Test of the LKD/FA-stabilized soil mixes are shown in Tables 3.41, 3.42 and 3.43.

Table 3.41: MDD and OMC of Soil-1 (A-6) Mixed with LKD/FA

Test Number	2%LKD/5%FA		3%LKD/9%FA		5%LKD/15%FA	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	102.6	17.2	103.0	16.4	102.3	17.4
2	103.0	18.2	103.2	17.3	104.0	16.7
3	102.7	17.7	103.6	16.9	102.9	16.9
Average	102.8	17.7	103.3	16.9	103.1	17.0

Table 3.42: MDD and OMC of Soil-2(A-4) Mixed with LKD/FA

Test Number	2%LKD/5%FA		2%LKD/8%FA	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	114.2	14.1	114.3	13.4
2	114.8	13.4	113.2	12.6
3	113.8	13.7	114.7	13.7
Average	114.3	13.8	114.1	13.2

Table 3.43: MDD and OMC of Soil-3 (A-7-6) Mixed with LKD/FA

Test Number	2%LKD/5%FA		2%LKD/8%FA		3%LKD/9%FA	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	96.1	21.5	97.9	21.9	97.1	21.0
2	96.5	20.3	97.3	21.8	97.2	21.8
3	97.1	18.0	97.4	22.0	97.1	20.5
Average	96.6	19.9	97.5	22.0	97.1	21.1

3.5.4.3 Calibration of Harvard Miniature Compaction Apparatus

The calibration summaries of the Harvard Miniature Compaction Apparatus for LKD/FA mixed with Soil-1, Soil-2 and Soil-3 are shown in Tables 3.44, 3.45 and 3.46 respectively.

Table 3.44: Harvard Miniature Compactor Apparatus Calibration for Soil-1 (A-6) Mixed with LKD/FA

Percentage of LKD/FA	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
2/5	37.5	5	20	15.2
3/9	37.5	5	20	15.6
5/15	37.5	5	20	16.7

Table 3.45: Calibration of compactor for Soil-2 (A-4) mixed with LKD/FA

Percentage of LKD/FA	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
2/5	37.5	5	15	11.83
2/8	37.5	5	15	11.61

Table 3.46: Calibration of compactor for Soil-3 (A-7-6) mixed with LKD/FA

Percentage of LKD/FA	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
2/5	37.5	5	20	18.67
2/8	37.5	5	20	20.75
3/9	37.5	5	20	21.04

3.5.4.4 Unconfined Compressive Strength (UCS) Test

Changes in the UCS of the soaked samples with respect to curing period are shown in Figures 3.31, 3.33 and 3.35. Changes in the UCS of the unsoaked samples are shown in Figure 3.32, 3.34 and 3.36.

The soaked UCS of LKD/FA-treated Soil-1, Soil-2 and Soil-3 samples showed more than 50 psi of required strength gain over the unstabilized strength values. For long-term stabilization, 3% LKD/9% FA is recommended for Soil-1 and Soil-3 - 3% LKD/9% FA, and 2% LKD/5% FA is recommended for Soil-2.

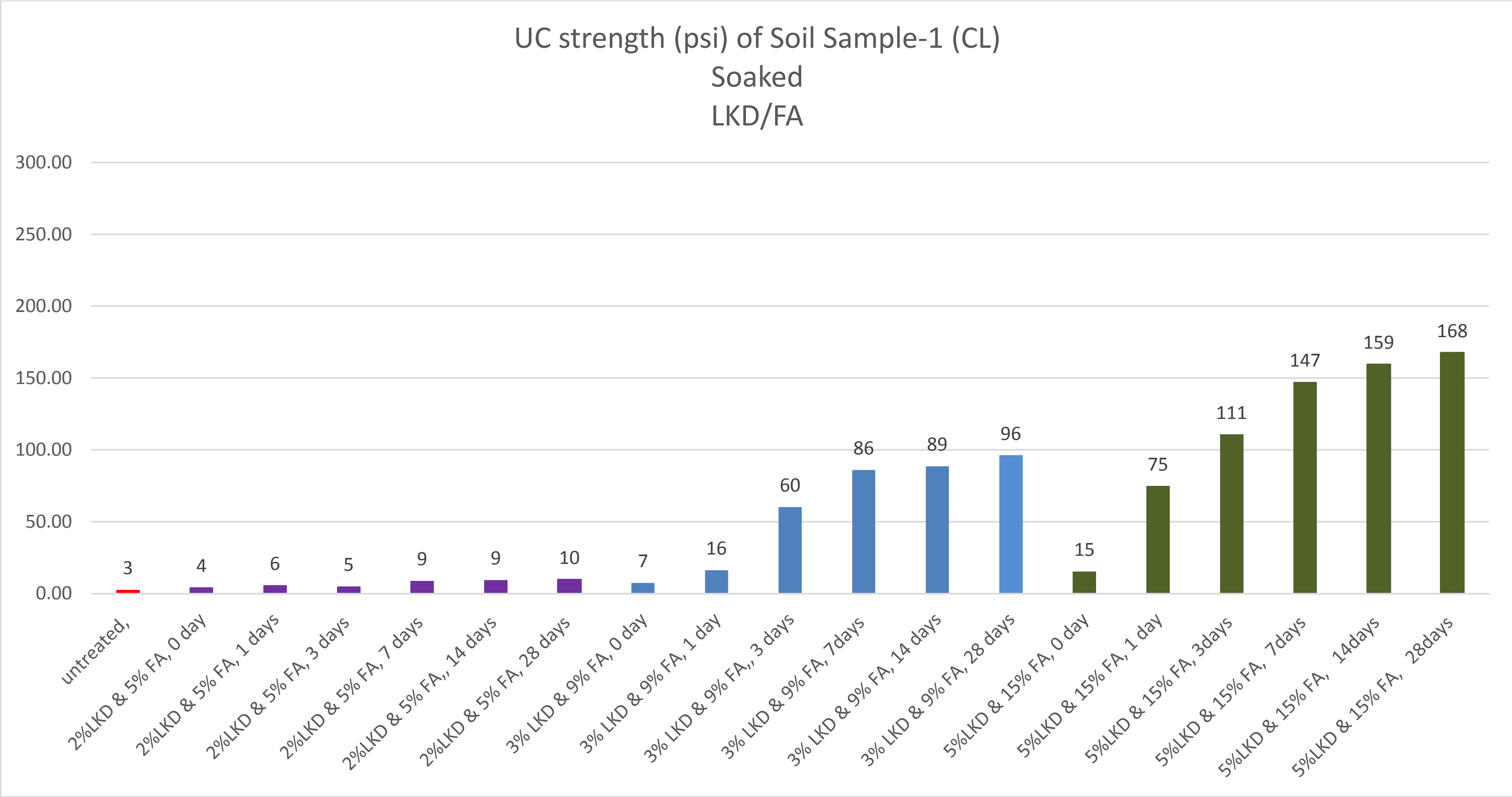


Figure 3.31: Comparison of Soaked UCS of Soil-1 (A-6) & LKD/FA Mixes

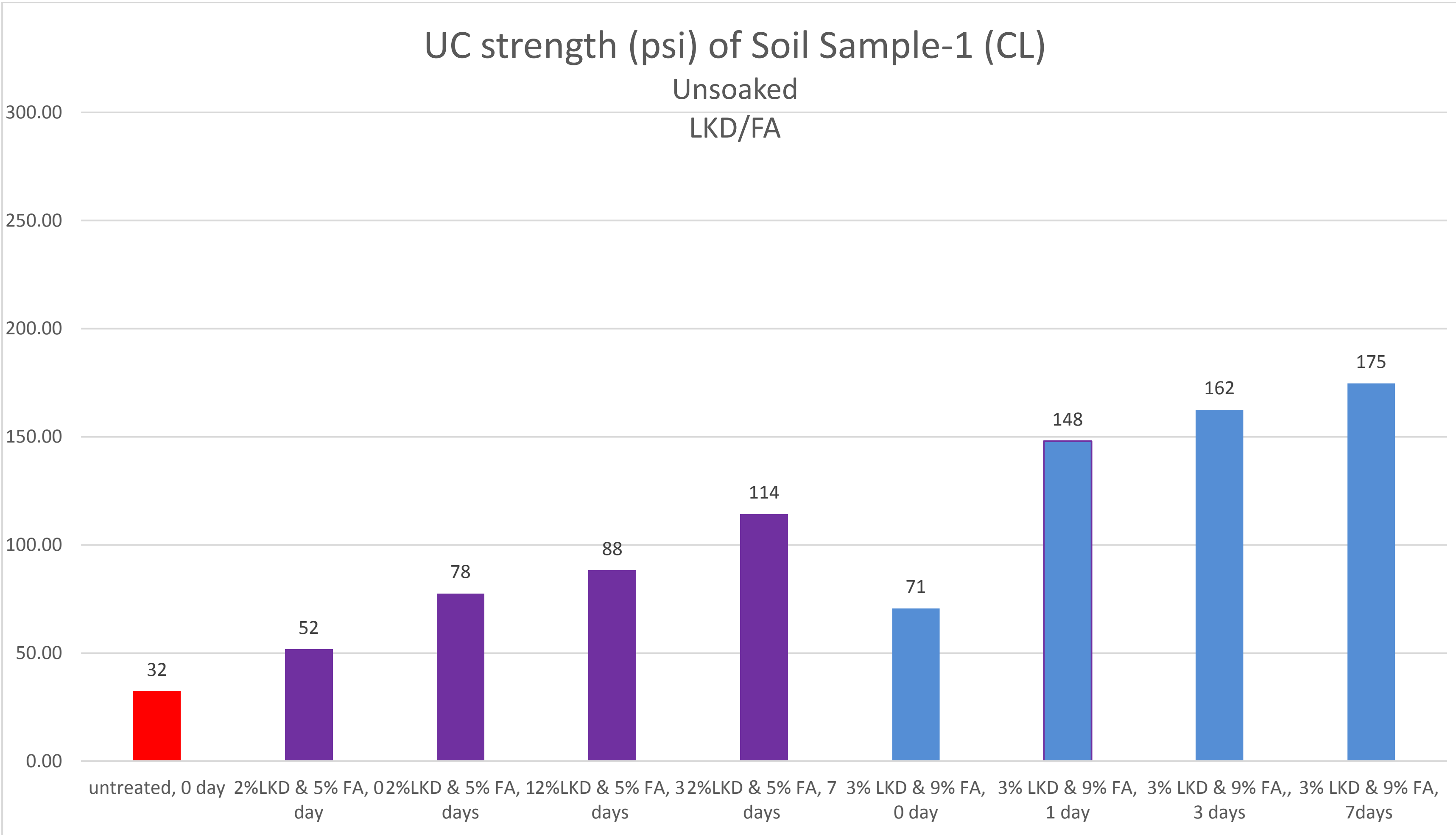


Figure 3.32: Comparison of Unsoaked UCS of Soil-1 (A-6) & LKD/FA Mixes

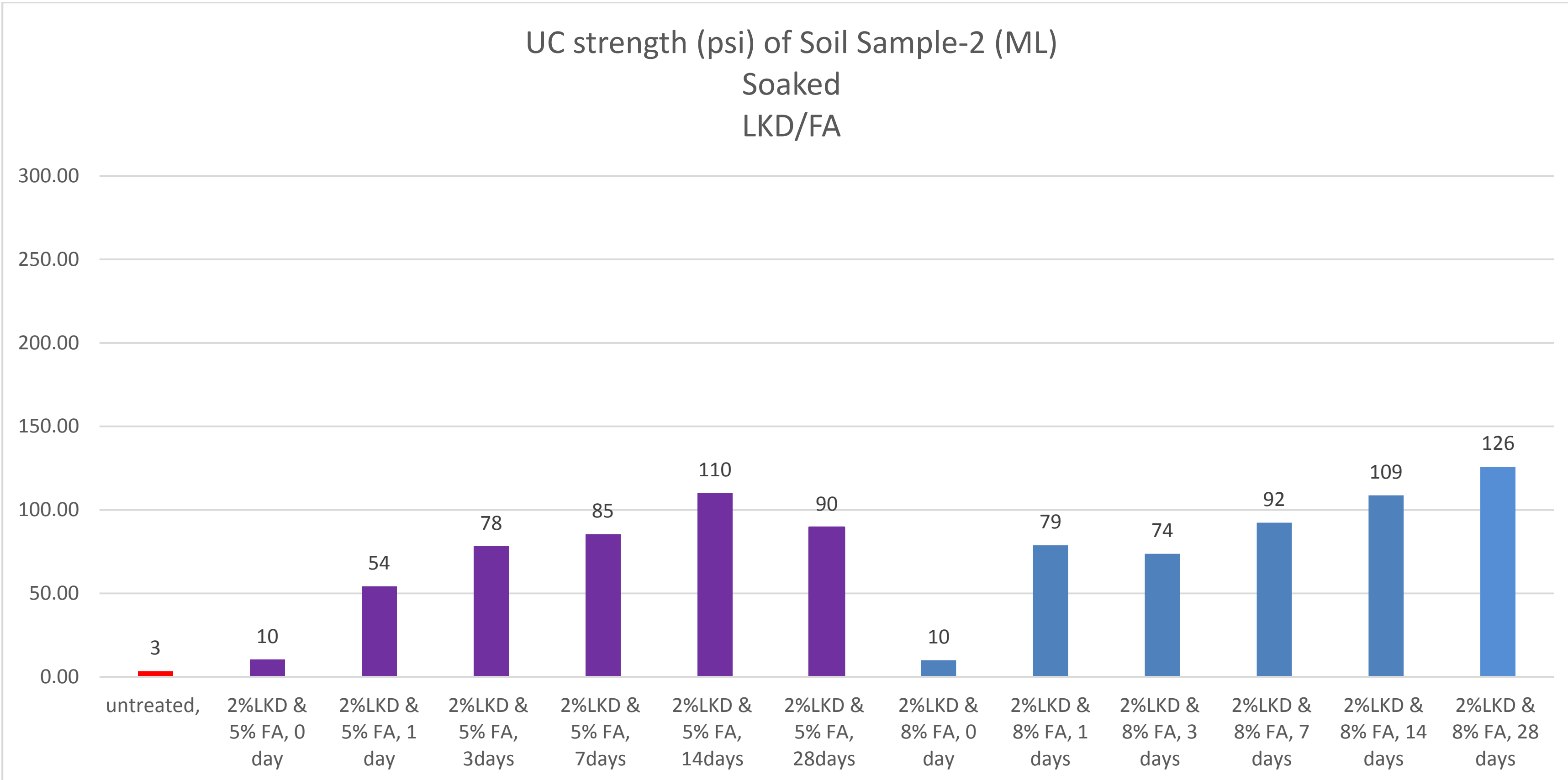


Figure 3.33: Comparison of Soaked UCS of Soil-2 (A-4) & LKD/FA Mixes

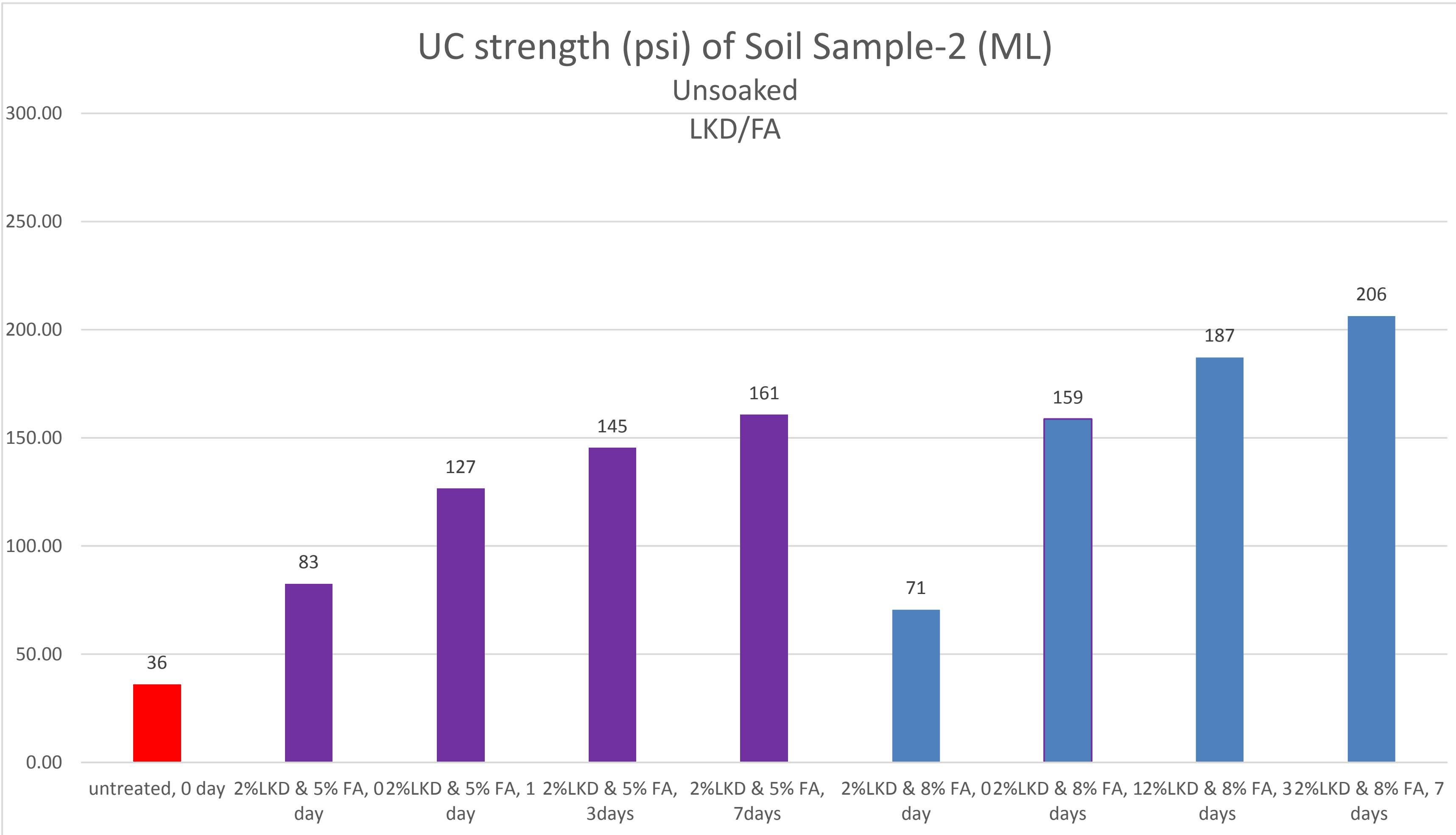


Figure 3.34: Comparison of Unsoaked UCS of Soil-2 (A-4) & LKD/FA Mixes

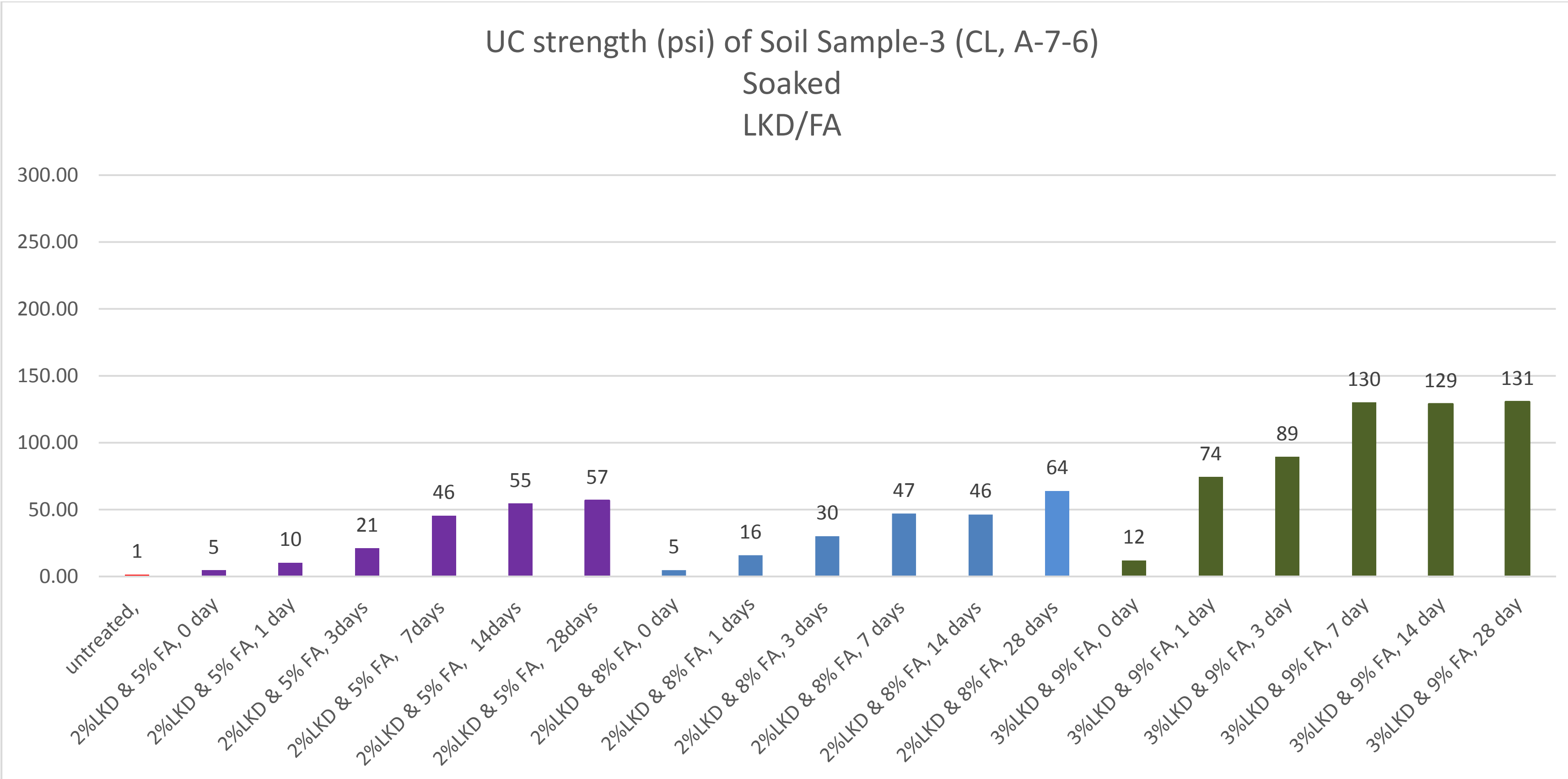


Figure 3.35: Comparison of Soaked UCS of Soil-3 (A-7-6) & LKD/FA Mixes

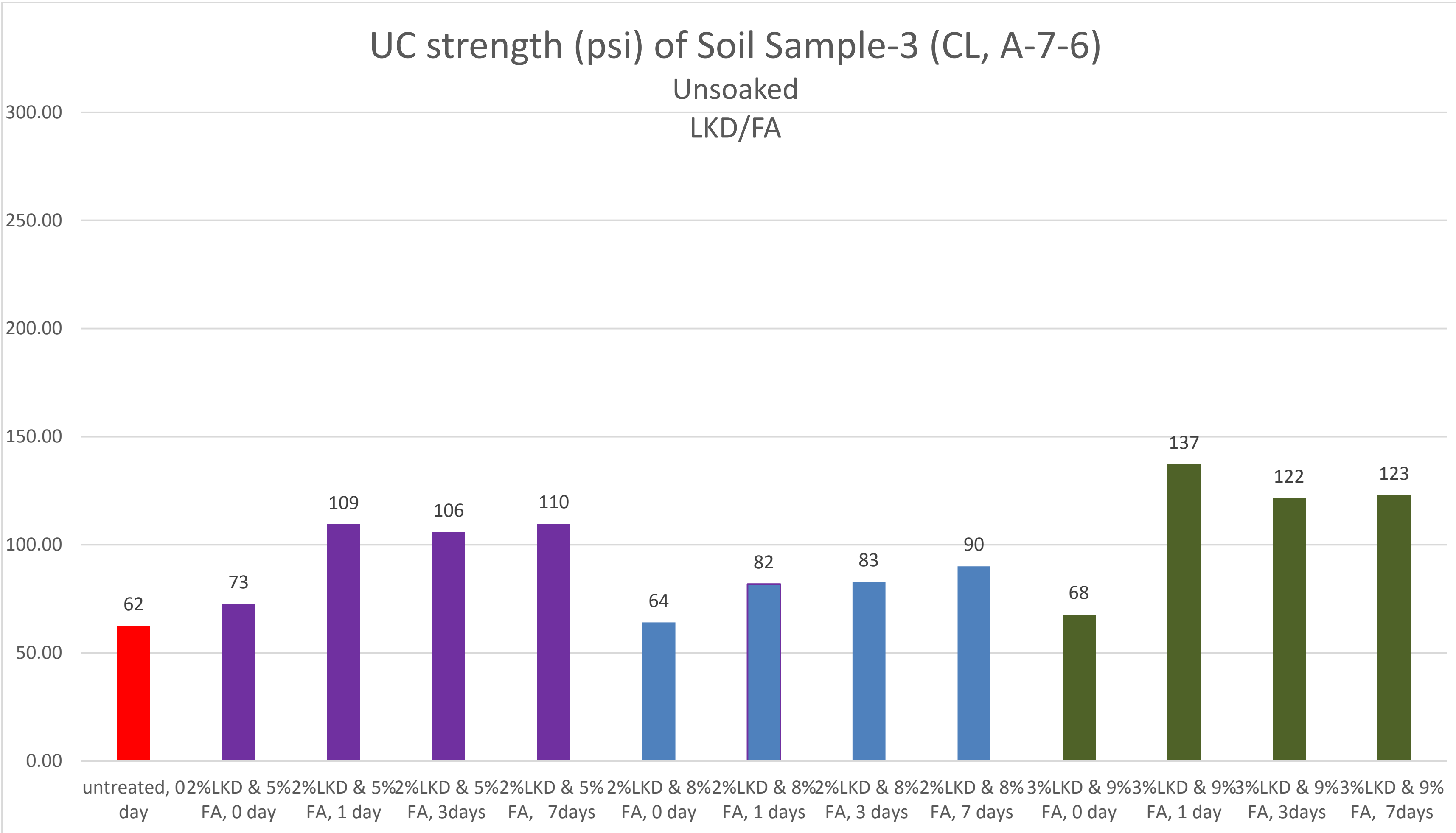


Figure 3.36: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & LKD/FA Mixes

3.5.5 Lime Kiln Dust (LKD)

Two types of Lime Kiln Dust were used in this research project: high-calcium Lime Kiln Dust (LKD) and Dolomite Lime Kiln Dust (DLKD). Shown in Table 3.47, the percentages of LKD and DLKD mixed with the different soils were determined using the pH test described in Section 3.5.5.1.

Table 3.47: Percentages of LKD and DLKD Mixed with Different Soil Types

Soil	LKD (%)	DLKD (%)
Soil-1 (A-6)	6	12
Soil-2 (A-4)	4	17
Soil-3 (A-7-6)	6	16

3.5.5.1 Laboratory pH test

The mix ratio of LKD and DLKD was determined by a laboratory pH test. According to ASTM D6276 (Eads-Grim Test), the standard maximum allowable lime content in soil is the percentage of LKD and/or DLKD that produces a pH of 12.4. A solution of 20g soil and different percentages of LKD and DLKD were mixed with 100 mL water. The treated soil samples were mixed every ten minutes. After an hour, the pH was measured. The pH of the soils, LKD, and DLKD was also determined. These results are shown in Tables 3.48 and 3.49. The pH results of the different soils mixed with LKD and DLKD are shown in Tables 3.50 to 3.55. From the pH test data, the percentages of LKD and DLKD that generated a pH of 12.4 were selected for further testing. The pH values of the different soil mixes were corrected for temperature using the following equation:

$$pH_{corrected} = pH_{reading} + 0.003 \times (pH_{reading} - 7) \times (T - 25) \qquad \text{(Equation 3.1)}$$

Where,

$pH_{corrected}$ = corrected pH

$pH_{reading}$ = pH at temperature, T

T = temperature in ^{O}C

Table 3.48: Soil pH Results

Soil Number	pH reading	Temperature (°C)	Corrected pH
Soil - 1	7.82	26.3	7.82
Soil - 2	7.66	23.8	7.66
Soil - 3	7.67	23.9	7.67

Table 3.49: LKD and DLKD pH Results

Stabilizer Type	pH Reading	Temperature (°C)	Corrected pH
LKD	12.62	26.1	12.64
DLKD	12.61	24.6	12.60

Table 3.50: Soil-1 (A-6) and LKD Mix pH Results

LKD %	pH Reading	Temperature (°C)	Corrected pH
2	11.98	26.6	12.00
3	12.19	26.7	12.22
4	12.30	26.4	12.32
5	12.36	26.5	12.38
6	**12.40**	**26.4**	**12.42**
7	12.48	26.5	12.50
8	12.50	26.5	12.52
9	12.51	26.5	12.53
10	12.51	26.6	12.54

Table 3.51: Soil-1 (A-6) and DLKD Mix pH Results

DLKD %	pH Reading	Temperature (°C)	Corrected pH
5	11.62	26.9	11.65
6	11.78	27.0	11.81
7	11.91	26.9	11.94
8	11.97	27.0	12.00
10	12.35	25.5	12.36
12	**12.40**	**25.5**	**12.41**
14	12.42	25.3	12.42
16	12.44	25.4	12.45

Table 3.52: Soil-2 (A-4) and LKD Mix pH Results

LKD %	pH Reading	Temperature (°C)	Corrected pH
1	11.62	24.2	11.61
2	12.08	23.9	12.06
3	12.22	23.9	12.20
4	**12.42**	**23.9**	**12.40**
5	12.48	24.0	12.46
6	12.52	23.9	12.50
7	12.54	24.0	12.52
8	12.57	24.1	12.55

Table 3.53: Soil-2 (A-4) and DLKD Mix pH Results

DLKD %	pH Reading	Temperature (°C)	Corrected pH
11	12.14	24.0	12.12
12	12.16	23.8	12.14
13	12.20	24.0	12.18
14	12.28	24.1	12.27
15	12.31	24.1	12.30
16	12.35	24.0	12.33
17	**12.41**	**24.0**	**12.39**
18	12.45	24.2	12.44

Table 3.54: Soil-3 (A-7-6) and LKD Mix pH Results

LKD %	pH Reading	Temperature (°C)	Corrected pH
1	11.04	24.0	11.03
2	11.59	23.8	11.57
3	12.06	23.7	12.04
4	12.22	23.8	12.20
5	12.34	24.3	12.33
6	**12.44**	**24.0**	**12.42**
7	12.49	24.3	12.48
8	12.56	24.1	12.54

Table 3.55: Soil-3 (A-7-6) and DLKD Mix pH Results

DLKD %	pH Reading	Temperature (°C)	Corrected pH
11	12.23	24.0	12.21
12	12.25	24.1	12.24
13	12.29	24.2	12.28
14	12.35	24.1	12.34
15	12.39	24.2	12.38
16	**12.42**	**24.2**	**12.41**
17	12.44	24.2	12.43
18	12.48	24.1	12.47

3.5.5.2 Atterberg Limit Tests

The Liquid Limit, Plastic Limit, and Plasticity Index results of LKD and DLKD-stabilized Soil-1, Soil-2 and Soil-3 are shown in Tables 3.56, 3.57 and 3.58 respectively. The tables also show the soil classification of the stabilized soils.

Changes in Atterberg limits were insignificant in most cases after adding LKD. The classification remains unchanged in most cases from the untreated soil except for the 12% DLKD – Soil-1 (A-6) mix. AASHTO classification of A-7-6 was applied to this mix.

Table 3.56: Atterberg Limit Test Results of LKD and Soil-1 (A-6) Mix

Percentage Stabilizer	Test	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
6% LKD	Plastic Limit, PL	26.0	27.1	28.0	27.0	A-6
	Liquid Limit, LL	37.4	40.5	39.9	39.3	
	Plasticity Index, PI	11.3	13.4	11.9	12.2	
12% DLKD	Plastic Limit, PL	29.2	27.7	26.9	28.0	A-7-6
	Liquid Limit, LL	43.4	42.8	42.1	42.7	
	Plasticity Index, PI	14.1	15.0	15.1	14.8	

Table 3.57: Atterberg Limit Test Results of LKD and Soil-2 (A-4) Mix

Percentage Stabilizer	Test	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
4% LKD	Plastic Limit, PL	18.3	18.2	18.9	18.5	A-4
	Liquid Limit, LL	21.5	22.0	22.2	21.9	
	Plasticity Index, PI	3.2	3.8	3.2	3.4	
17% DLKD	Plastic Limit, PL	19.5	19.2	18.8	19.2	A-4
	Liquid Limit, LL	22.5	22.3	22.2	22.3	
	Plasticity Index, PI	3.0	3.1	3.3	3.1	

Table 3.58: Atterberg Limit Test Results of LKD and Soil-3 (A-7-6) Mix

Percentage Stabilizer	Test	Run 1	Run 2	Run 3	Average	Classification of Mixed Soils
6% LKD	Plastic Limit, PL	24.3	24.5	24.7	24.5	A-7-6
	Liquid Limit, LL	44.6	44.2	45.7	44.8	
	Plasticity Index, PI	20.2	19.7	20.9	20.3	
16% DLKD	Plastic Limit, PL	27.7	27.4	27.2	27.4	A-7-6
	Liquid Limit, LL	47.3	47.5	47.1	47.3	
	Plasticity Index, PI	19.6	20.2	19.9	19.9	

3.5.5.3 Standard Proctor Test

According to ASTM D3551, all soil/LKD mixes used for testing were prepared 24 hours prior to performing the Standard Proctor Test. In order to compensate for evaporation that occurs during mixing, 1% more water over the desired water content was added.

Results of the Standard Proctor Test of the LKD/DLKD and soil mixes are shown in Tables 3.59, 3.60 and 3.61.

Table 3.59: MDD and OMC of Soil-1 (A-6) Mixed with LKD

Test Number	6%LKD		12%DLKD	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	97.8	15.9	104.1	17.3
2	98.5	16.7	103.4	17.5
3	98.0	15.4	103.5	17.2
Average	98.1	16.0	103.7	17.3

Table 3.60: MDD and OMC of Soil-2 (A-4) Mixed with LKD

Test Number	4%LKD		17%DLKD	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	110.2	13.9	113.2	13.8
2	111.1	13.5	113.7	13.1
3	110.4	14.4	113.2	13.1
Average	110.6	14.0	113.4	13.3

Table 3.61: MDD and OMC of Soil-3 (A-7-6) Mixed with LKD

Test Number	6%LKD		16%DLKD	
	MDD, Υ_d (lb/ft^3)	OMC, ω (%)	MDD, Υ_d (lb/ft^3)	OMC, ω (%)
1	94.6	20.1	97.6	17.6
2	94.5	19.6	97.5	18.0
3	95.1	20.6	97.1	20.2
Average	94.7	20.1	97.4	18.6

3.5.5.4 Calibration of Harvard Miniature Compaction Apparatus

A summary the Harvard Miniature Compaction Apparatus calibration of LKD/DLKD mixed with Soil-1, Soil-2 and Soil-3 are shown in Tables 3.62, 3.63 and 3.64 respectively.

Table 3.62: Harvard Miniature Compactor Apparatus Calibration for Soil-1 (A-6) Mixed with LKD/DLKD

Percentage Stabilizer	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
6% LKD	37.5	5	15	16.27
12% DLKD	37.5	5	10	17.95

Table 3.63: Harvard Miniature Compactor Apparatus Calibration for Soil-2 (A-4) Mixed with LKD/DLKD

Percentage Stabilizer	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
4% LKD	37.5	5	15	14.39
17% DLKD	37.5	5	15	13.45

Table 3.64: Harvard Miniature Compactor Apparatus Calibration Apparatus for Soil-3 (A-7-6) Mixed with LKD/DLKD

Percentage Stabilizer	Spring Weight	Layers	Blows/Layer	Moisture Content (%)
6% LKD	37.5	5	15	18.02
16% DLKD	37.5	5	15	17.85

3.5.5.5 Unconfined Compressive Strength (UCS) Test

As with the Standard Proctor Test, all soil/LKD/DLKD mixes were prepared for testing with 1% more water to compensate for evaporation that occurs during mixing. After compaction and curing, UCS tests were performed on the soaked and unsoaked samples to determine the strength gain during curing. Changes in UCS for soaked samples with curing period are shown Figures 3.37 to 3.47. Changes in UCS of unsoaked samples are shown in Figure 3.38 to 3.48.

Changes in UCS of soaked LKD or DLKD-treated soils were insignificant in all soils. The change in UCS of the unsoaked samples was less than 50 psi for Soil-2 and Soil-3. However, the unsoaked UCS of LKD and Soil-1 mix gained 50 psi over the untreated soil after three days of curing. Therefore, 6% LKD is recommended for modification of Soil-1.

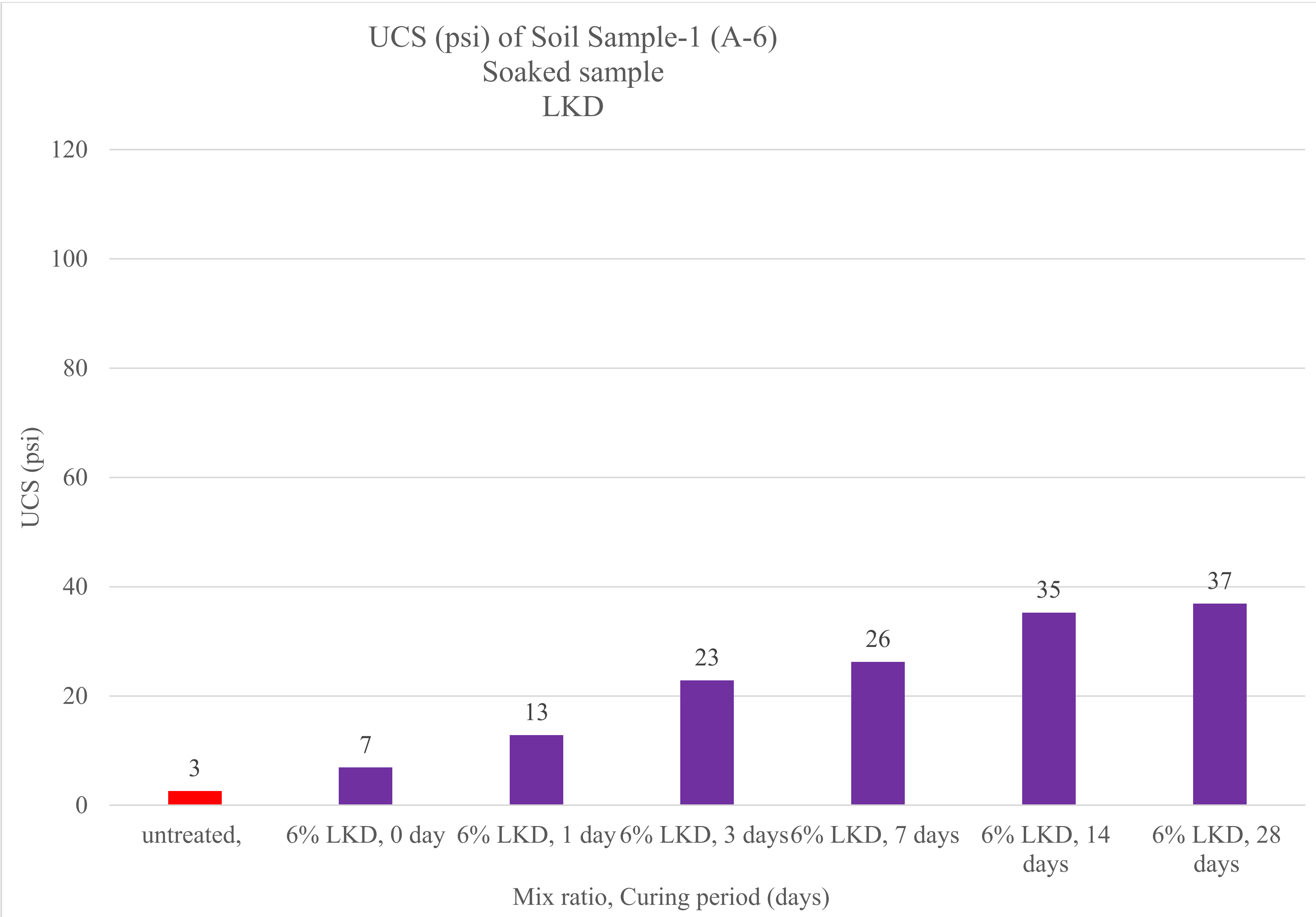


Figure 3.37: Comparison of Soaked UCS of Soil-1 (A-6) & 6% LKD Mix

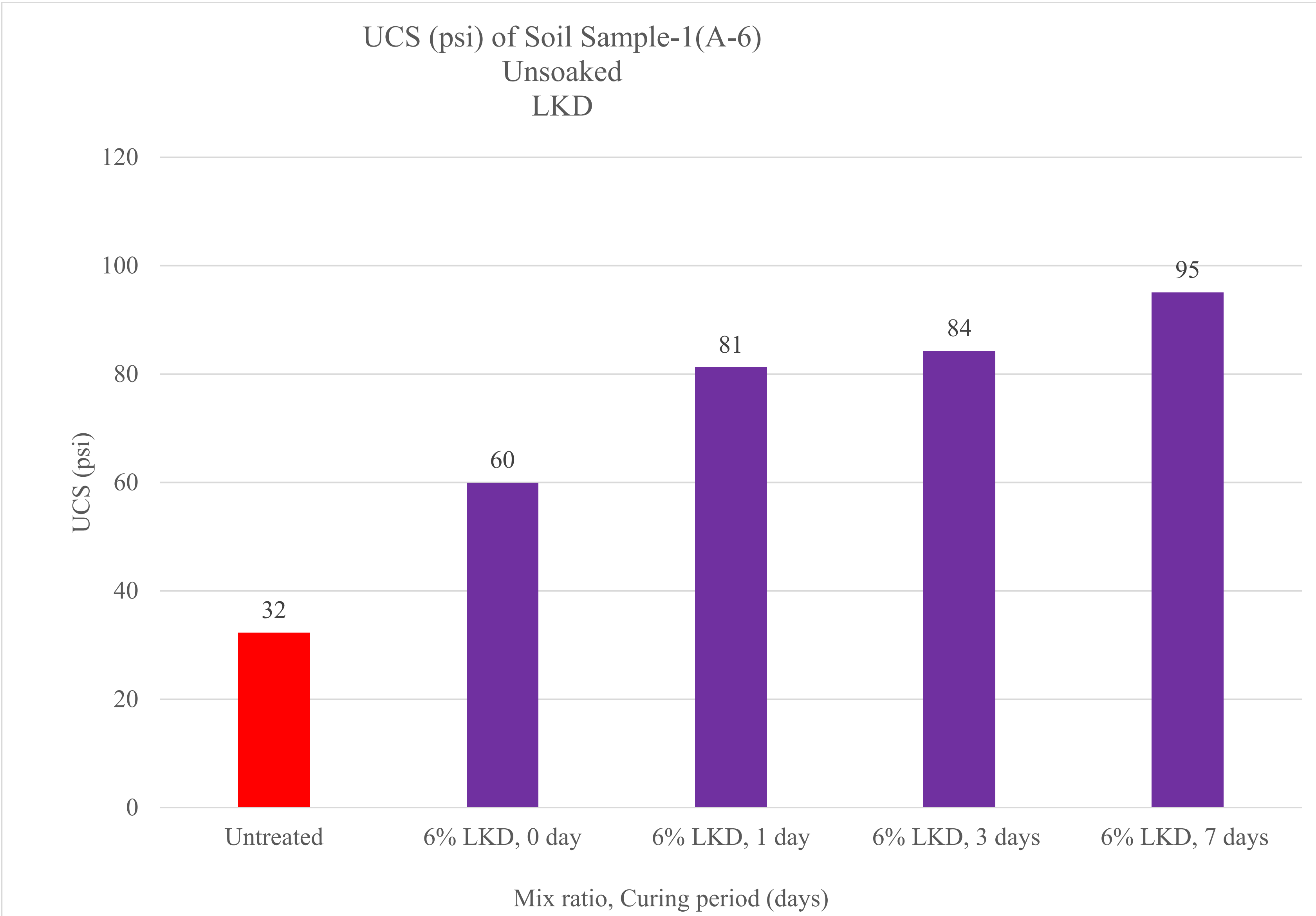


Figure 3.38: Comparison of Unsoaked UCS of Soil-1 (A-6) & 6% LKD Mix

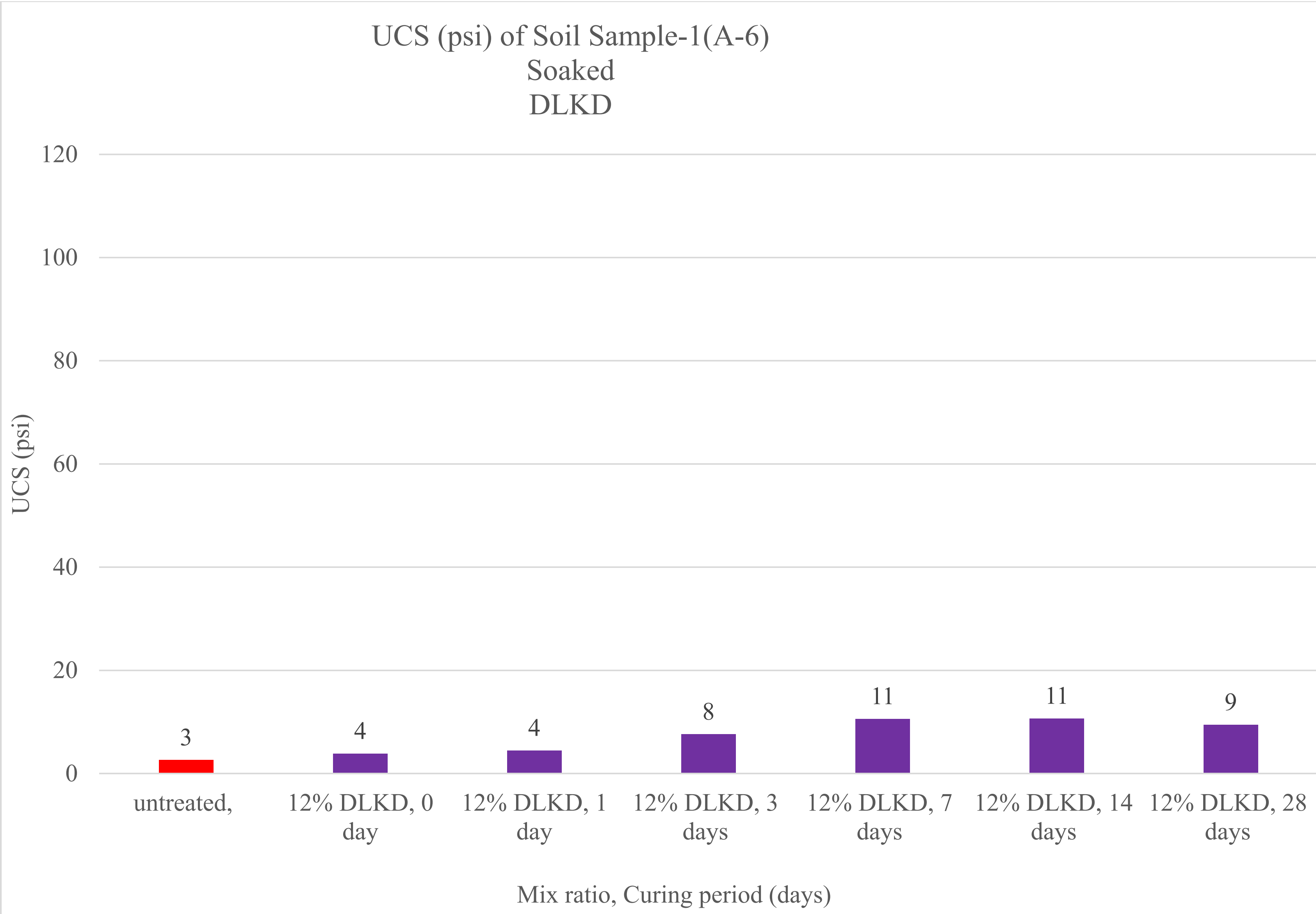


Figure 3.39: Comparison of Soaked UCS of Soil-1 (A-6) & 12% DLKD Mix

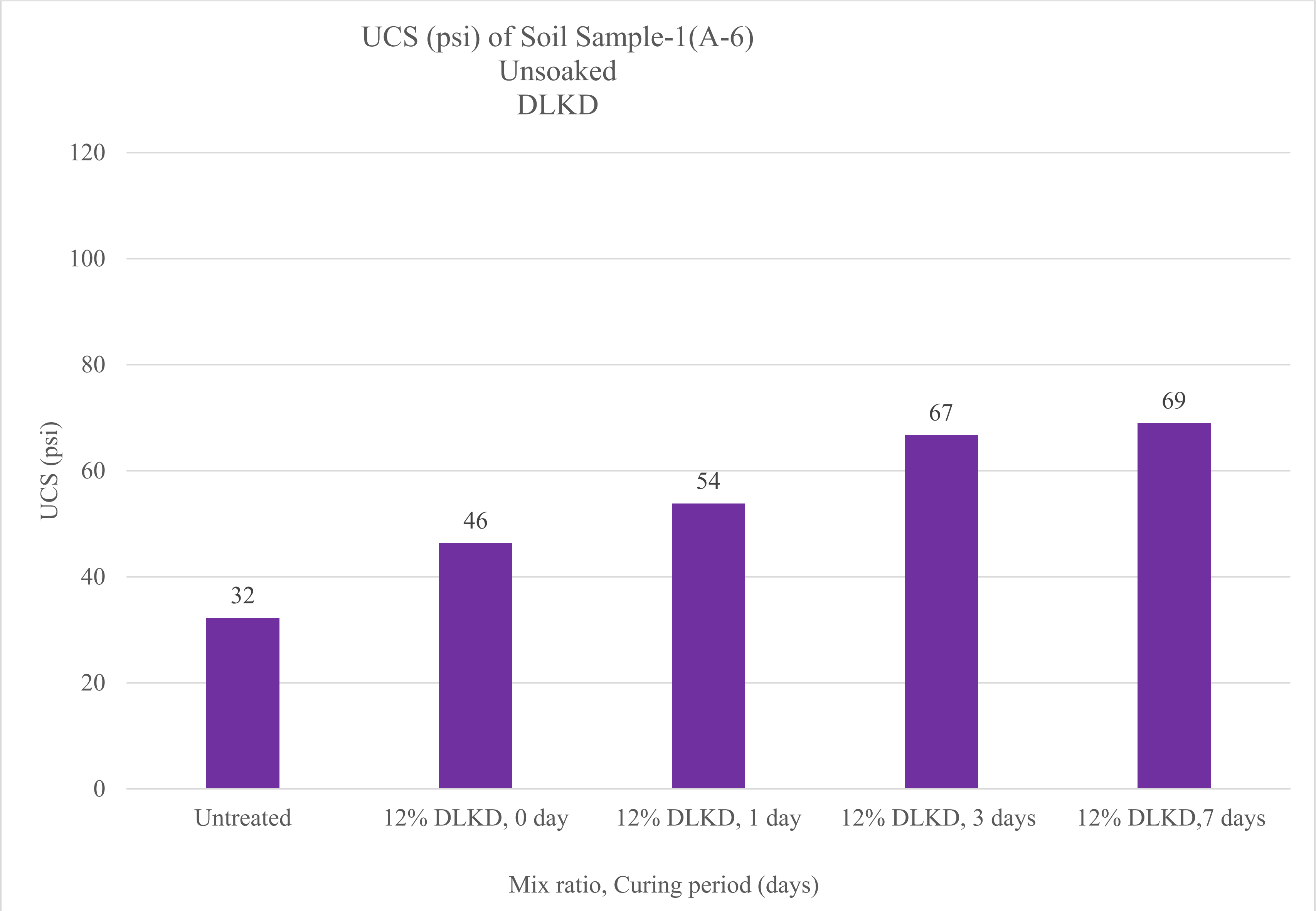


Figure 3.40: Comparison of Unsoaked UCS of Soil-1 (A-6) & 12% DLKD Mix

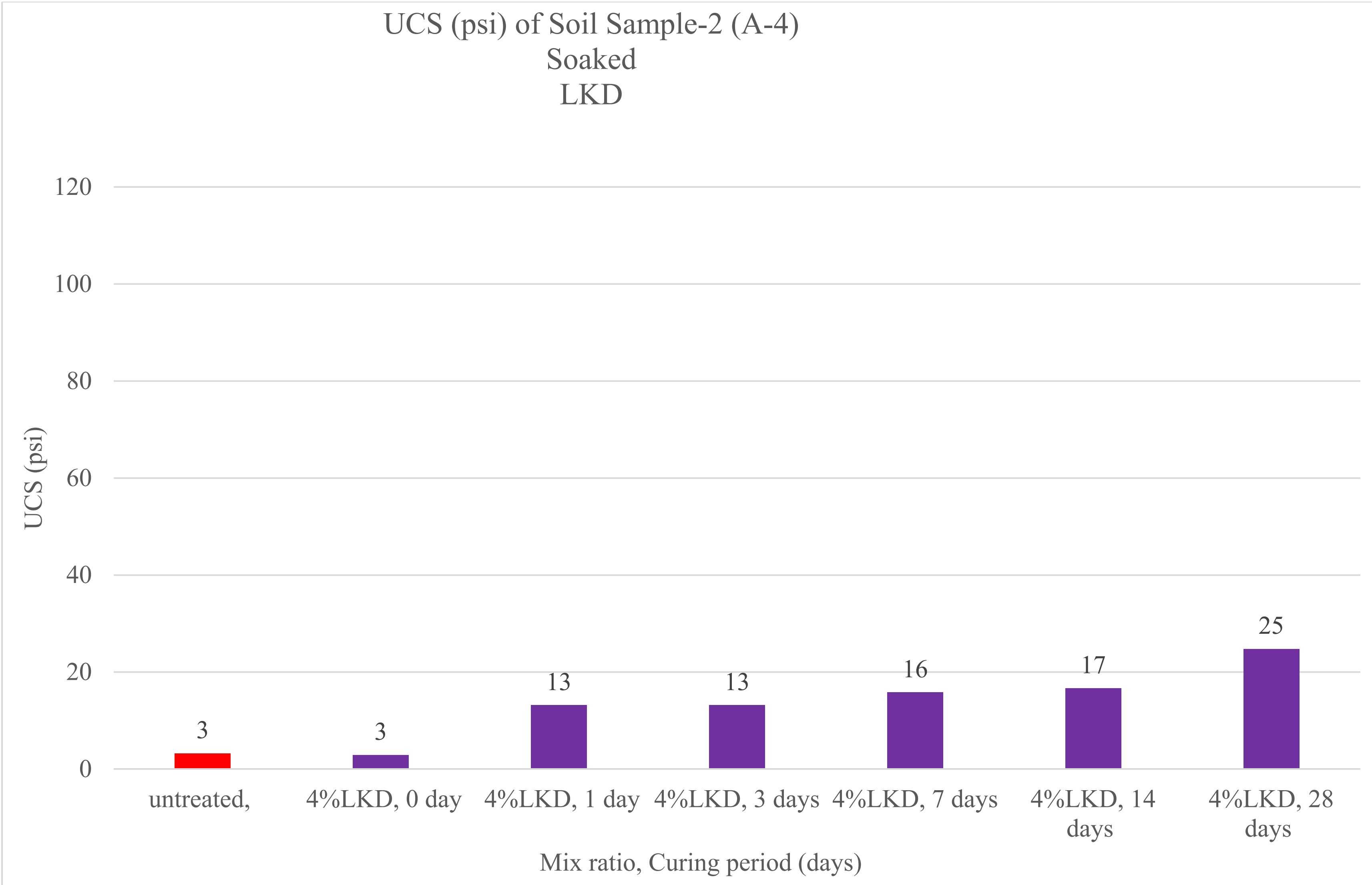


Figure 3.41: Comparison of Soaked UCS of Soil-2 (A-4) & 4% LKD Mix

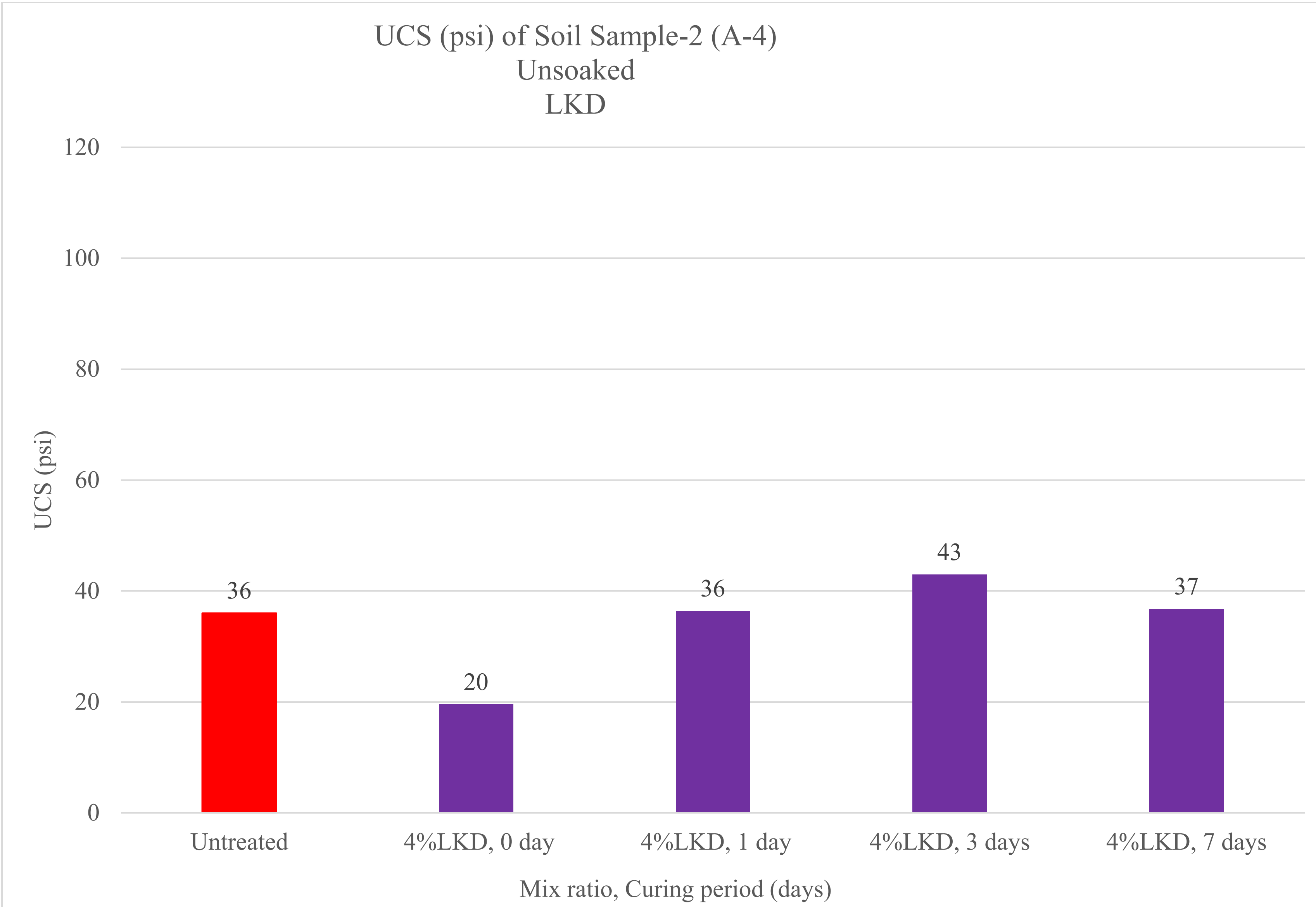


Figure 3.42: Comparison of Unsoaked UCS of Soil-2 (A-4) & 4% LKD Mix

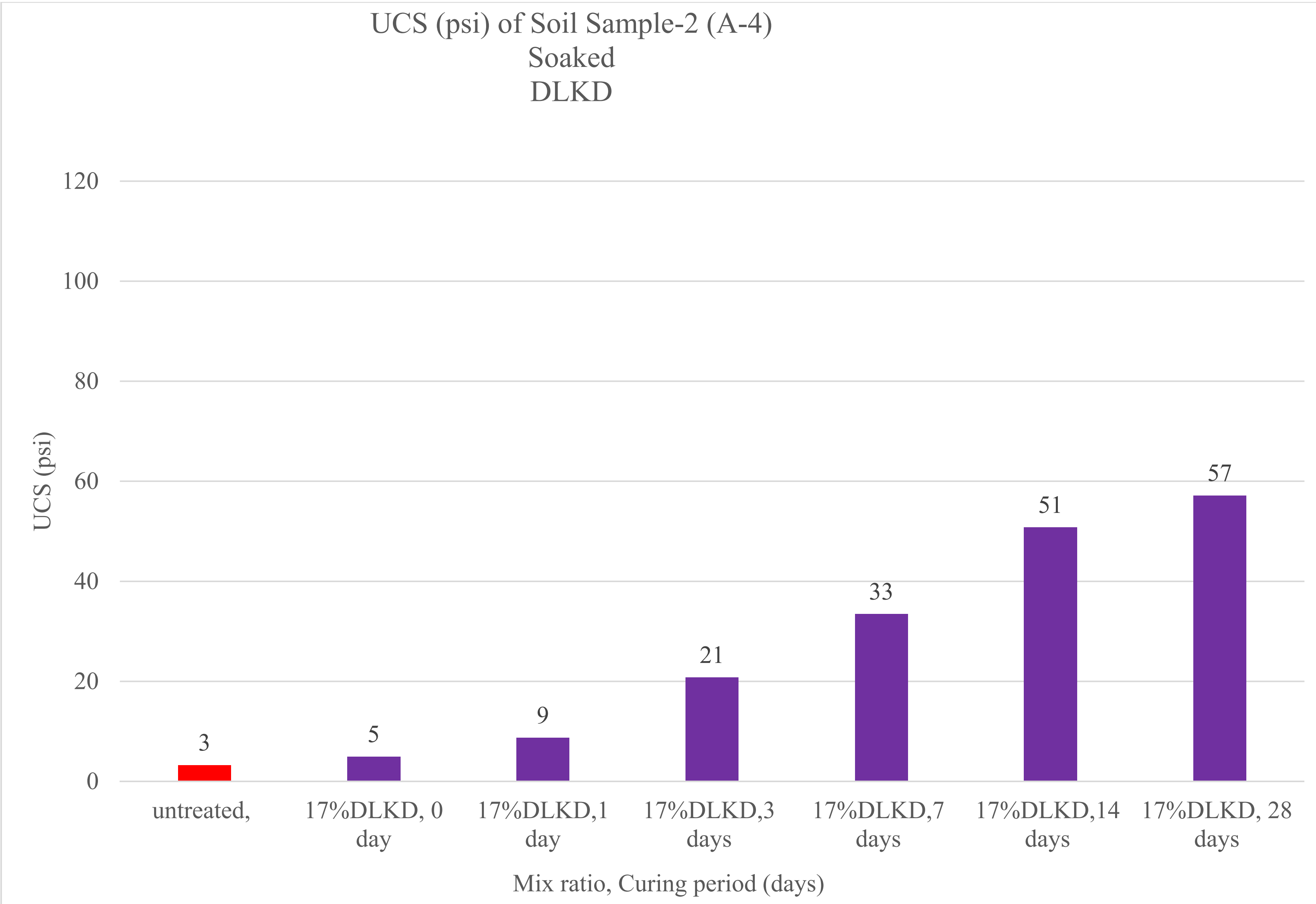


Figure 3.43: Comparison of Soaked UCS of Soil-2 (A-4) & 17% DLKD Mix

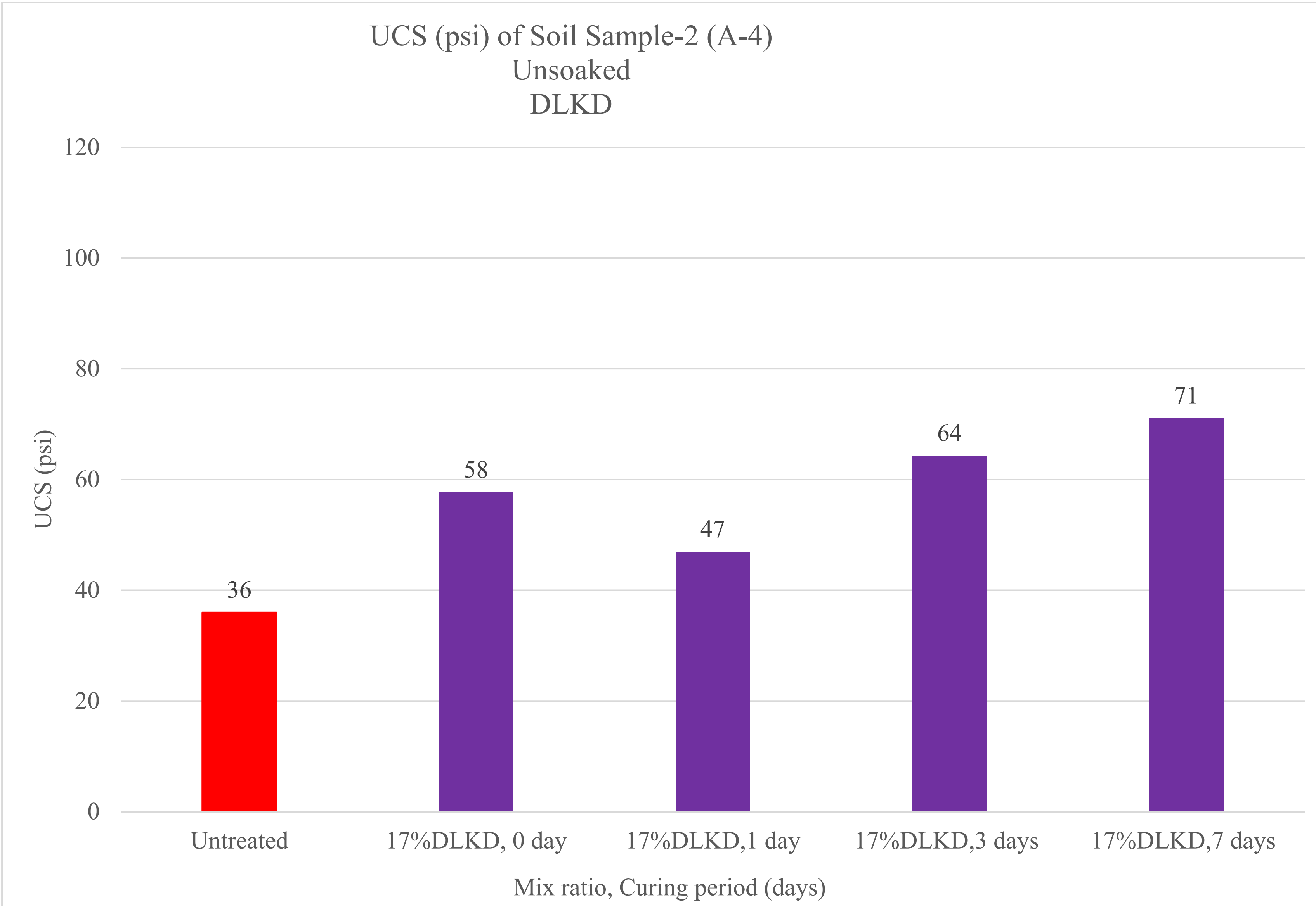


Figure 3.44: Comparison of Unsoaked UCS of Soil-2 (A-4) & 17% DLKD Mix

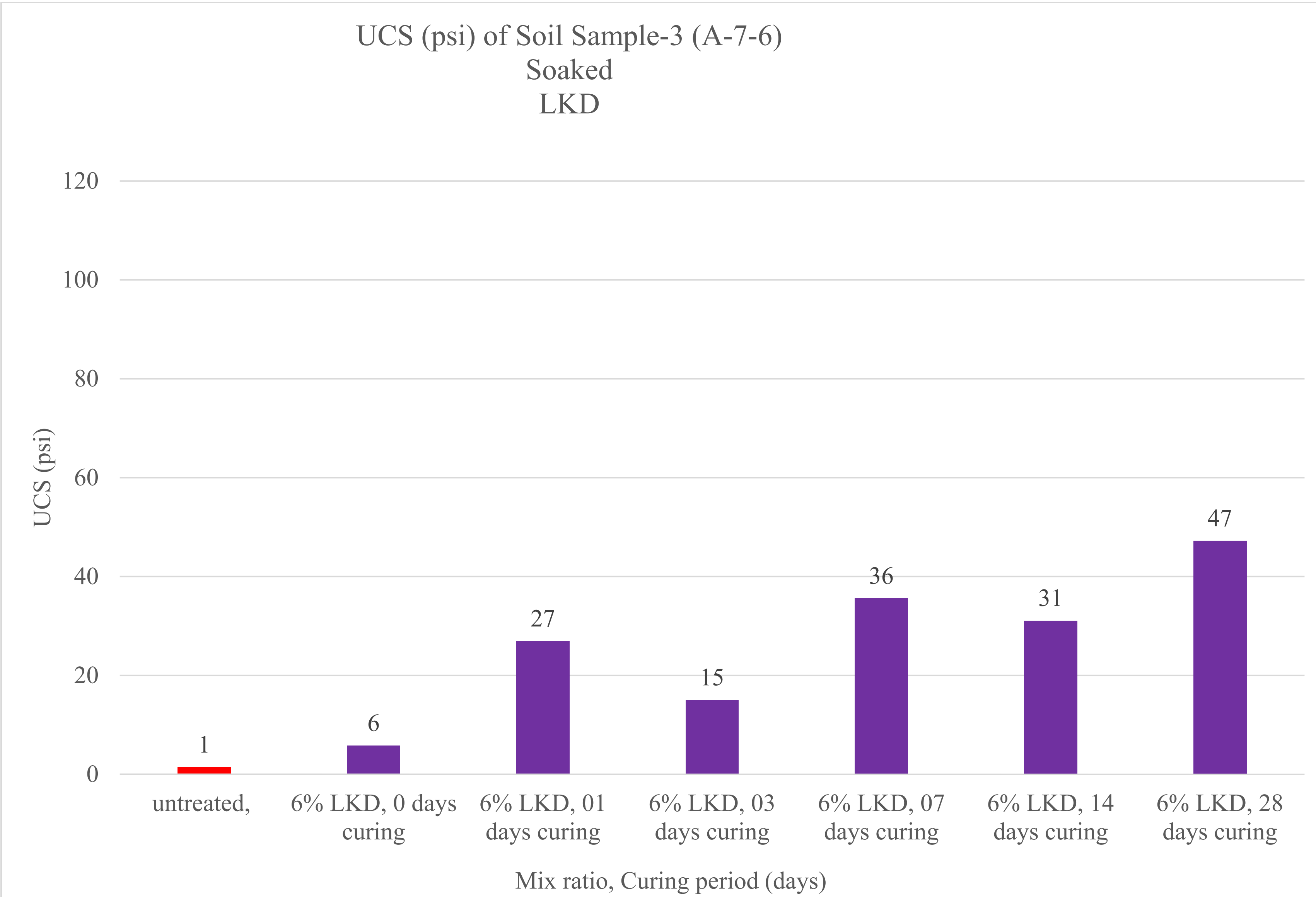


Figure 3.45: Comparison of Soaked UCS of Soil-3 (A-7-6) & LKD Mix

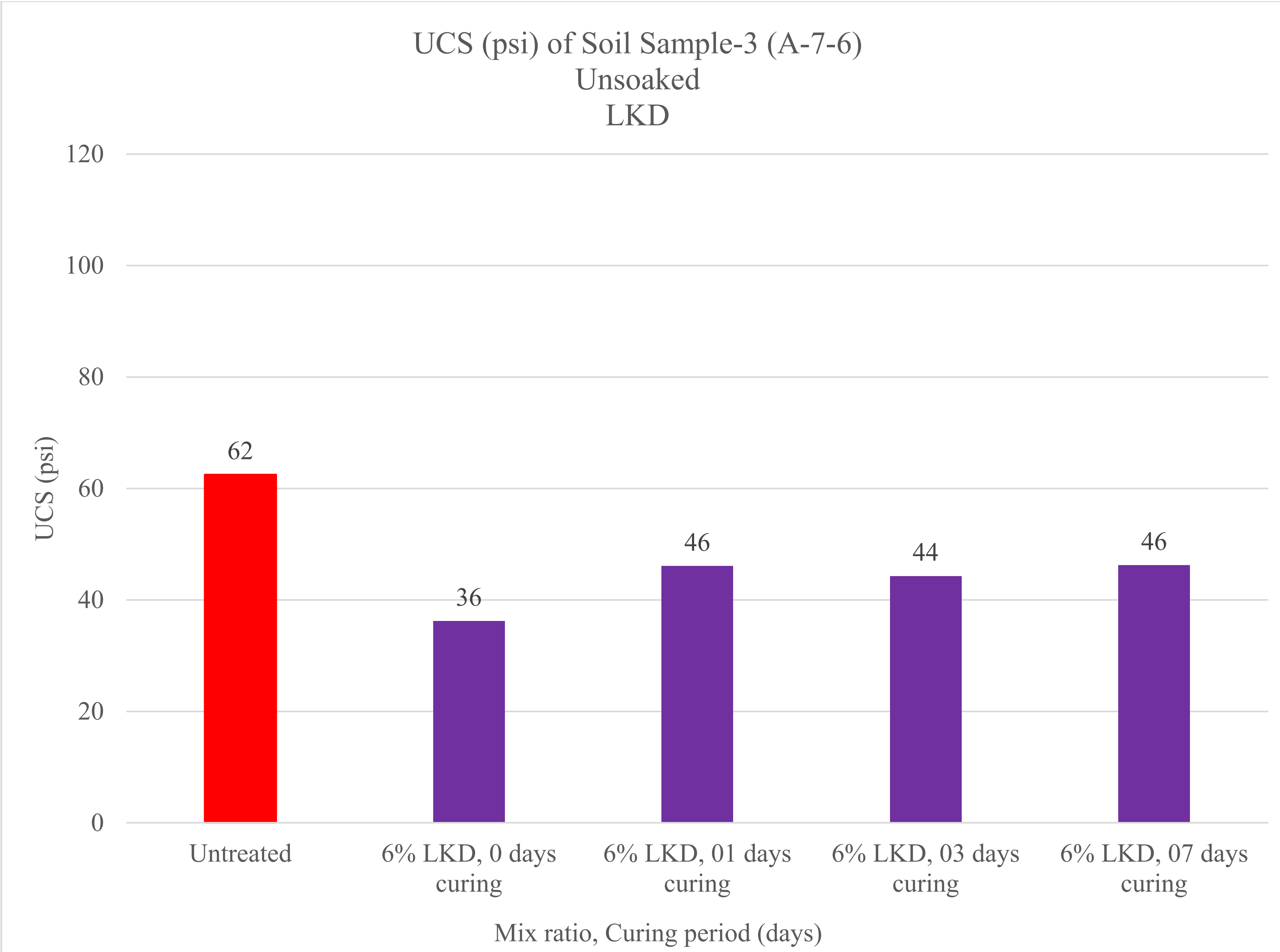


Figure 3.46: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & LKD Mix

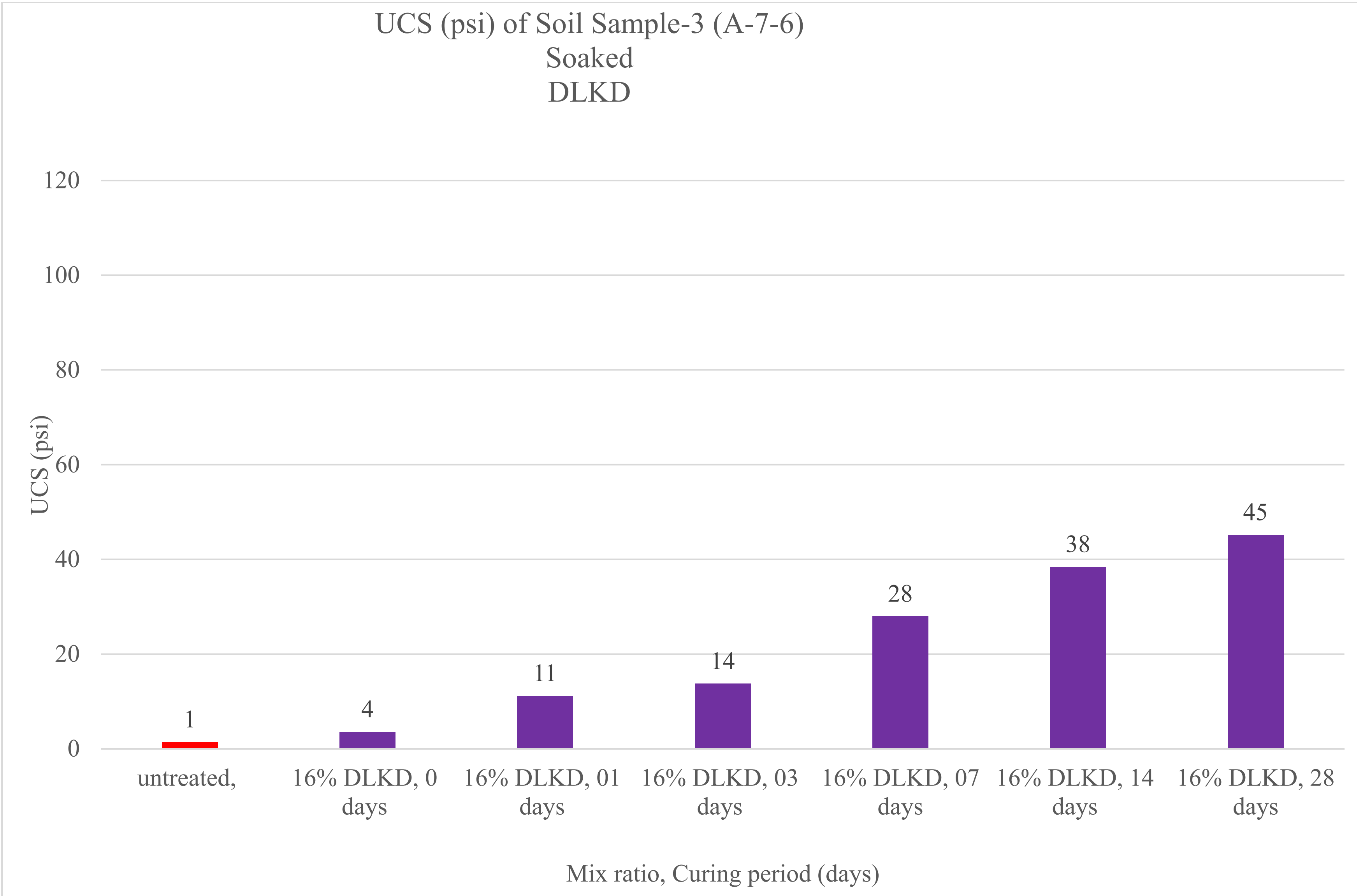


Figure 3.47: Comparison of Soaked UCS of Soil-3 (A-7-6) & DLKD Mix

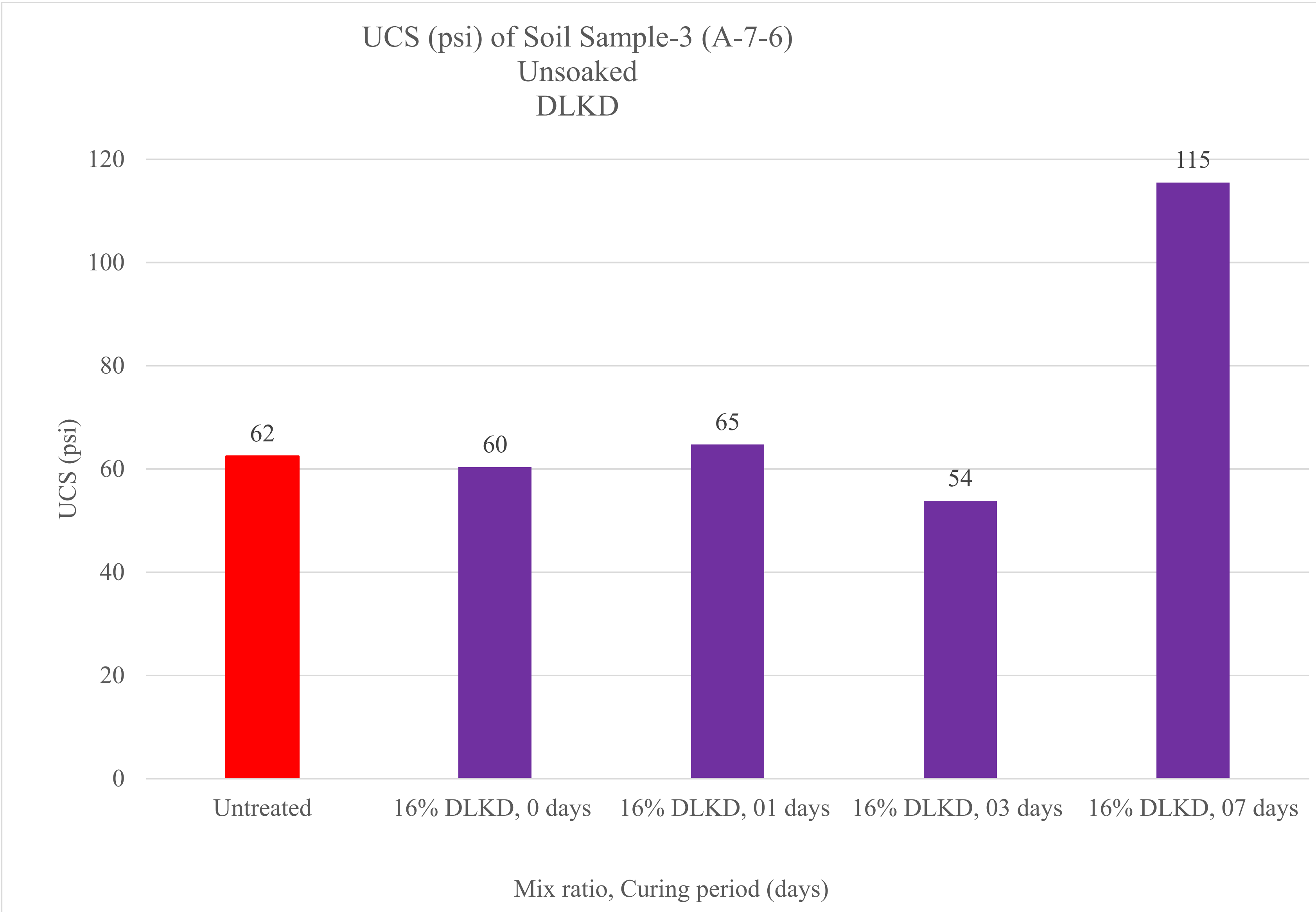


Figure 3.48: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & DLKD Mix

3.6 Mix Ratio Selection

Tables 3.65, 3.66, and 3.67 list the UCS results obtained for the soaked samples cured for seven days and the unsoaked samples cured for three days for Soil-1, Soil-2, and Soil-3. Pursuant to the short-term and long-term recommendations set forth in Section 3.4.2, if a treated soaked sample UCS increased more than 50 psi over the untreated soil after seven days of curing, the treatment is recommended for long-term stabilization. If a treated unsoaked sample realized a USC gain over the untreated soil after three days of curing, the treatment is recommended for short-term modification.

Table 3.65: UCS Test Results & Selection of Stabilizer for Soil-1 (A-6)

Treatment	Soaked UCS (psi)*	Increase (psi)	Unsoaked UCS (psi)+	Increase (psi)	Comments
Untreated	2.61	-	32.26	-	
6% CKD	30.33	28	61.72	29	
8% CKD	71.91	69	70.71	38	Stabilization
12% CKD	77.77	75	153.51	121	
4% CF	4.29	2	55.86	24	
12% CF	18.40	16	48.43	16	
25% CF	19.91	17	57.60	25	
10% FA	10.94	8	63.81	32	
15% FA	4.71	2	92.81	61	Modification
25% FA	4.94	2	79.57	47	
2% LKD/5% FA	8.70	6	88.14	56	
3% LKD/9% FA	85.95	83	162.48	130	Stabilization
5% LKD/15% FA	147.15	145	192.55	160	
6% LKD	26.27	24	84.27	52	Modification
12% DLKD	10.59	8	66.75	34	

*Seven days of curing

+Three days of curing

Table 3.66: UCS Test Results & Selection of Stabilizer for Soil-2 (A-4)

Treatment	Soaked UCS (psi)*	Increase (psi)	Unsoaked UCS (psi)+	Increase (psi)	Comments
Untreated	3.25	-	36.00	-	
4% CKD	81.73	78	117.97	82	Stabilization
6% CKD	114.3	111	158.01	122	
8% CKD	104.21	101	206.67	171	
4% CF	6.82	4	26.88	-9	
12% CF	5.47	2	49.54	14	
25% CF	13.83	11	47.08	11	
10% FA	4.10	1	59.37	23	
15% FA	21.65	18	80.73	45	
25% FA	14.15	11	92.00	56	Modification
2% LKD/5% FA	85.38	82	145.40	109	Stabilization
2% LKD/8% FA	92.33	89	187.18	151	
4% LKD	15.82	13	42.93	7	
17% DLKD	33.43	30	64.33	28	

*Seven days of curing

+Three days of curing

Table 3.67: UCS Test Results & Selection of Stabilizer for Soil-3 (A-7-6)

Treatment	Soaked UCS (psi)*	Soaked UCS Increase (psi)	Unsoaked UCS (psi)+	Unsoaked UCS Increase (psi)	Comments
Untreated	1.43	-	62.49	-	
4% CKD	81.42	80	176.23	114	Stabilization
6% CKD	105.05	104	223.26	161	
8% CKD	133.43	132	220.46	158	
4% CF	4.25	3	71.77	9	
15% CF	6.58	5	54.51	-8	
25% CF	13.30	12	58.31	-4	
10% FA	24.26	23	102.48	40	
15% FA	67.99	67	91.12	29	Stabilization
25% FA	63.90	62	105.36	43	
2% LKD/5% FA	45.51	44	105.74	43	
2% LKD/8% FA	47.11	46	82.83	20	
3% LKD/9% FA	130.12	129	121.54	59	Stabilization
6% LKD	35.57	34	44.29	-18	
16% DLKD	27.96	27	53.78	-9	

*Seven days of curing

+Three days of curing

A summary of the selected treatments and their required percentages needed to stabilize or modify the different types of soils is shown in Table 3.68.

Table 3.68: Recommended Stabilizer Percentages

Soil Type	CKD (%)	LKD (%)/ FA (%)	FA (%)	CF (%)	LKD (%)	DLKD (%)
CL, A-6	8*	3/9*	15**	-	6 **	-
ML, A-4	4*	2/5*	25**	-	-	-
CL, A-7-6	4*	3/9*	15*	-	-	-

- Not recommended to use at any percentage.
* Percentage of required treatment for stabilization.
**Percentage of required treatment for modification, only.

3.7 CBR Test Results

California Bearing Ratio (CBR) tests were performed according to ASTM D1883 on the mix ratios selected for stabilization. The method used for preparation and compaction of soil specimens was ASTM D698 Method C. Fifty-six blows were applied to each of the three layers. The soil and treatments/recycled materials were mixed with water to achieve an OMC as determined by the Standard Proctor Test. A 2-inch diameter penetration piston was used to penetrate the soil during the test. A load was applied on the penetration piston so that the rate of penetration was approximately 0.05 inch/min (1.27mm/min). A 10-lbf surcharge load was applied on the specimen to prevent heaving of the soil. The same surcharge was used during 96 hours of specimen soaking in preparation for the soaked CBR test. All tests were performed in triplicate. The average results of the soaked CBR, the increase in CBR when compared to untreated soil, and the Resilient Modulus (M_R) calculated from CBR are shown in Table 3.69. Table 3.70 shows the results of the unsoaked CBR values. A stress-penetration graph of untreated Soil-1 (A-6) tested after 96 hours of soaking is shown is Figure 3.49. Bearing ratios were calculated at 0.1 inches (2.54 mm) and 0.2 inches (5.08 mm) of penetration. Stress values taken from the stress penetration curve for 0.1 inches and 0.2 inches penetrations were used to calculate the bearing ratios for each penetration by dividing the corrected stresses by the standard stresses of 1000 psi (6.9 MPa) and 1500 psi (10.3 MPa) respectively, and then multiplying by 100. The bearing ratio, reported for the soil, is normally the one at 0.1 in. (2.54 mm) penetration. If the ratio at 0.2 in. (5.08 mm) penetration is greater the bearing ratio at 0.2 in. (5.08mm) penetration is reported as CBR value. Hence, for Figure 3.49, the CBR at 0.1 inches and 0.2 inches are $39 \times \frac{100}{1000}$ or 3.9 and $65 \times \frac{100}{1500}6$ or 4.33 respectively. The CBR value was then used to calculate resilient modulus using the following equation:

$$M_r = 2555 \times CBR^{0.64} \qquad \text{(Equation 3.2)}$$

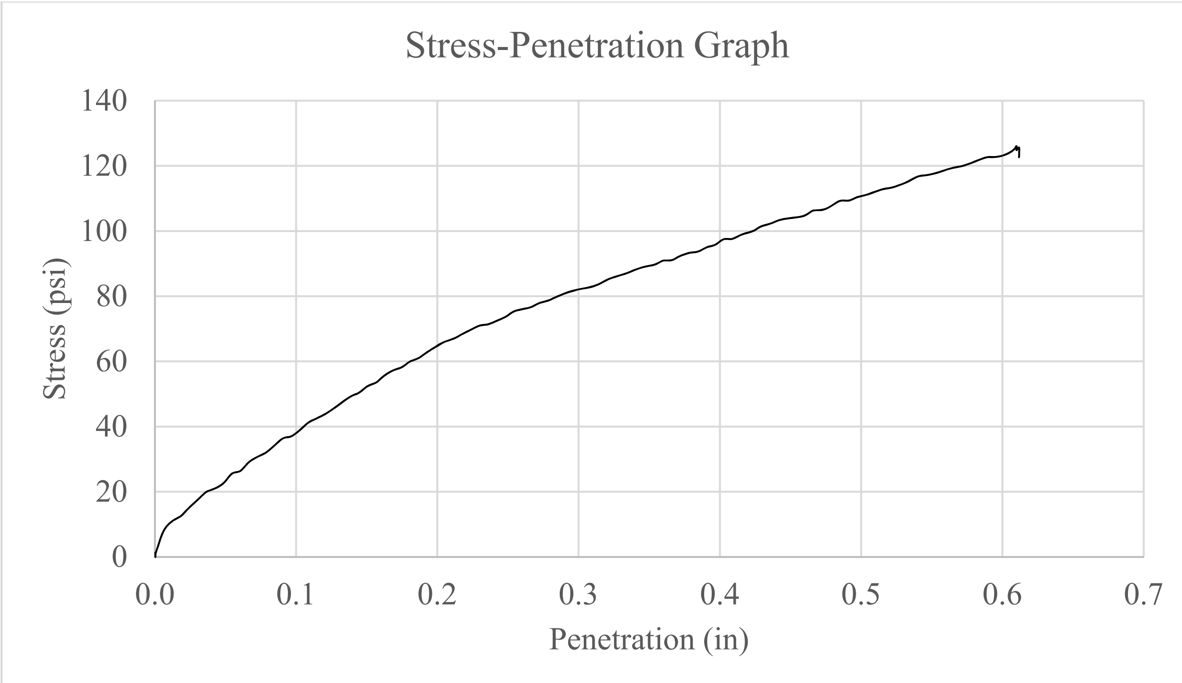


Figure 3.49: Example Stress-Penetration Graph of CBR Test (Soaked CBR of Untreated Soil-1, Specimen-1)

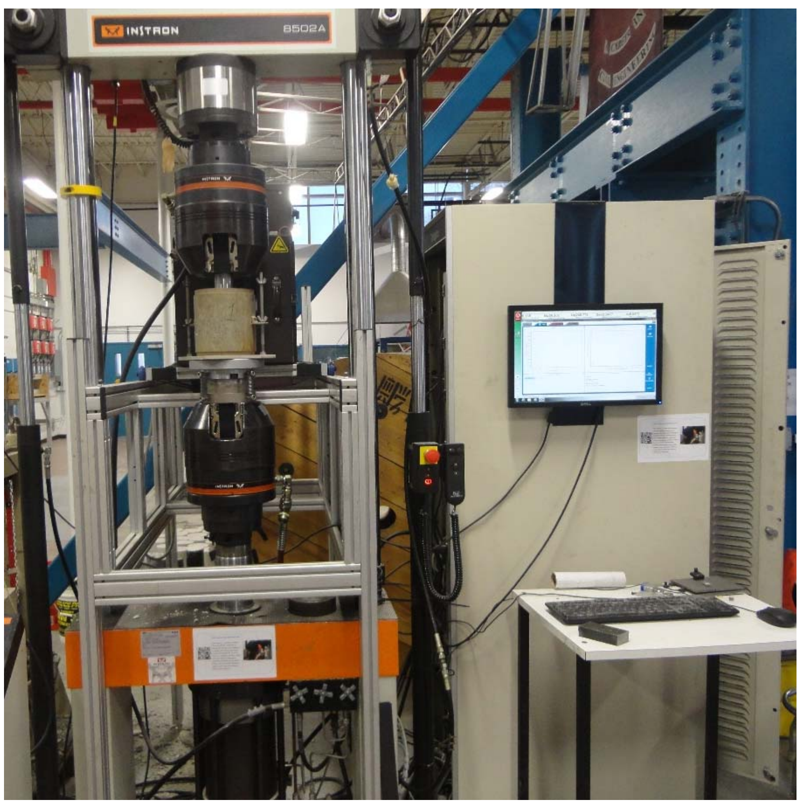


Figure 3.50: CBR Test Using Instron

Table 3.69: Soaked CBR & Resilient Modulus

Soil	Treatment	CBR	M_R Increase (%)	M_R (psi)
Soil-1 (CL, A-6)	Untreated	3.5	-	5,600
	8% CKD	8.2	75	9,800
	3% LKD/9% FA	33.4	328	24,000
Soil-2 (ML, A-4)	Untreated	2.5	-	4,500
	4% CKD	56.3	633	33,000
	2% LKD/5% FA	44.9	544	29,000
Soil-3 (ML, A-7-6)	Untreated	6.7	-	8,600
	4% CKD	55.3	283	33,000
	3% LKD/9% FA	49.8	260	31,000
	15% FA	35.7	190	25,000

Table 3.70: Unsoaked CBR & Resilient Modulus

Soil	Treatment	CBR	M_R Increase (%)	M_R (psi)
Soil-1 (CL, A-6)	Untreated	19.6	-	17,000
	8% CKD	26.9	23	21,000
	3% LKD/9% FA	34.4	43	24,500
Soil-2 (ML, A-4)	Untreated	17.5	-	15,900
	4% CKD	26.4	30	20,700
	2% LKD/5% FA	36.2	59	25,400
Soil-3 (ML, A-7-6)	Untreated	25.0	-	20,000
	4% CKD	42.0	40	27,900
	3% LKD/9% FA	35.9	26	25,200
	15% FA	28.4	9	21,700

3.8 Freeze/Thaw Durability Test Results

3.8.1 Laboratory Freeze/Thaw Test

A laboratory freeze/thaw test was performed on stabilized soils. ASTM D560 was utilized as a reference. The soil samples were prepared using the Harvard Miniature Compaction Apparatus and compacted to achieve the optimal MDD. The freeze/thaw cycles were initiated after compaction and 28 days of curing. One freeze/thaw cycle included 24 hours of freezing at −10°F (−23°C) followed by 24 hours of thawing at 70°F (21°C) Unconfined Compressive Strength tests were performed after a predetermined number of freeze/thaw cycles (1, 3, 7 and 12 cycles) and 24 hours of capillary soaking. Figure 3.50 shows the visual condition of a CKD-stabilized soil sample after seven freeze/thaw cycles and 24 hour capillary soaking period. Figure 3.52 shows the significant reduction in UCS after each freeze/thaw cycle.

Another laboratory freeze/thaw test was performed on samples using similar compaction, moisture content and freeze/thaw cycle conditions. One difference was implemented for this round of freeze/thaw testing. Instead of soaking the samples for 24 hours after the final freeze/thaw cycle, the samples were soaked after every thaw interval. Loss of strength was even more severe in this case (Figure 3.53).

Figure 3.51: Condition of Specimen after Seven Freeze/Thaw Cycles and 24 Hours of Capillary Soaking (Soil-1 stabilized with 8% CKD)

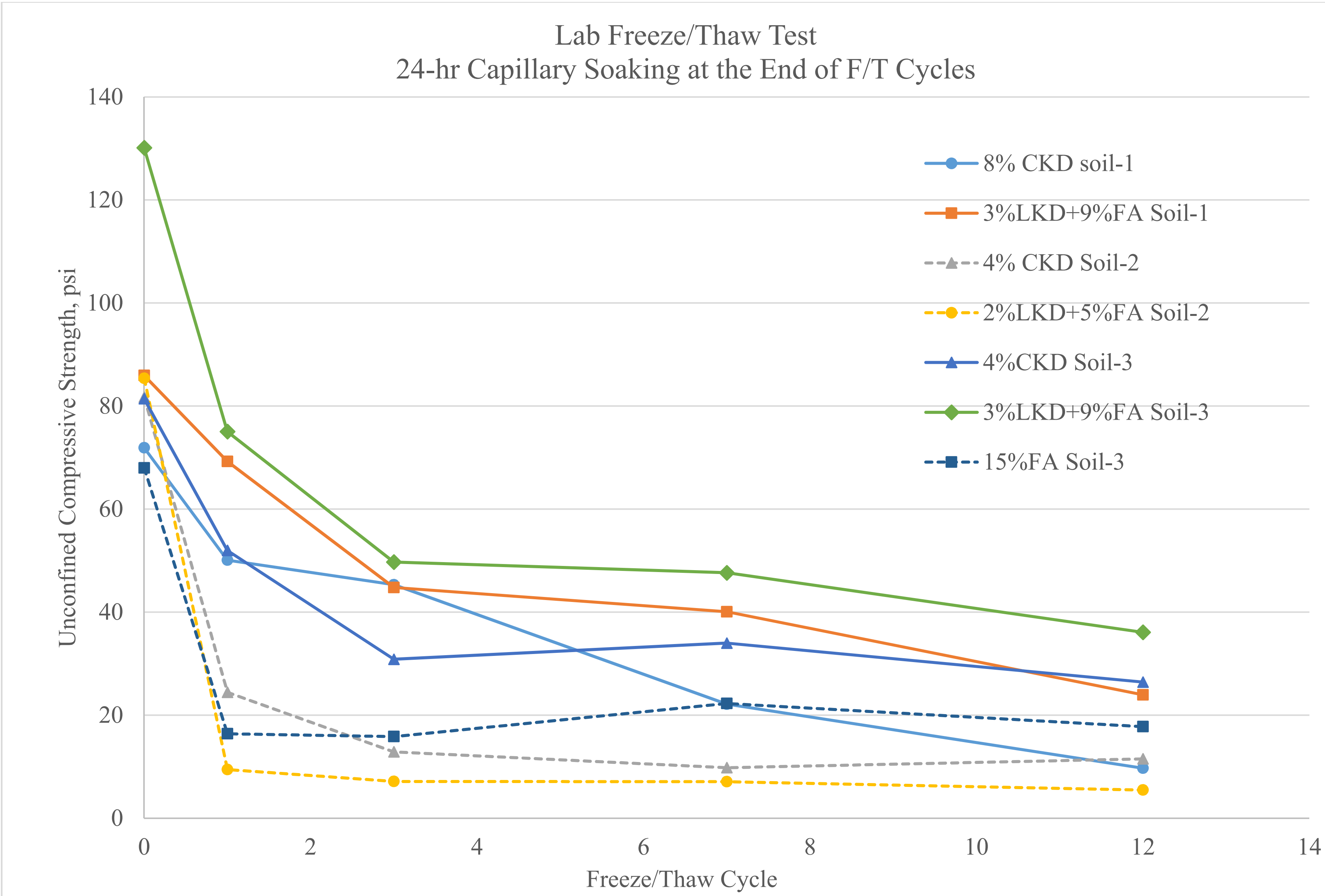


Figure 3.52: Reduction of UCS with Freeze/Thaw Cycles (24-hr Capillary Soaking at the End of Cycles)

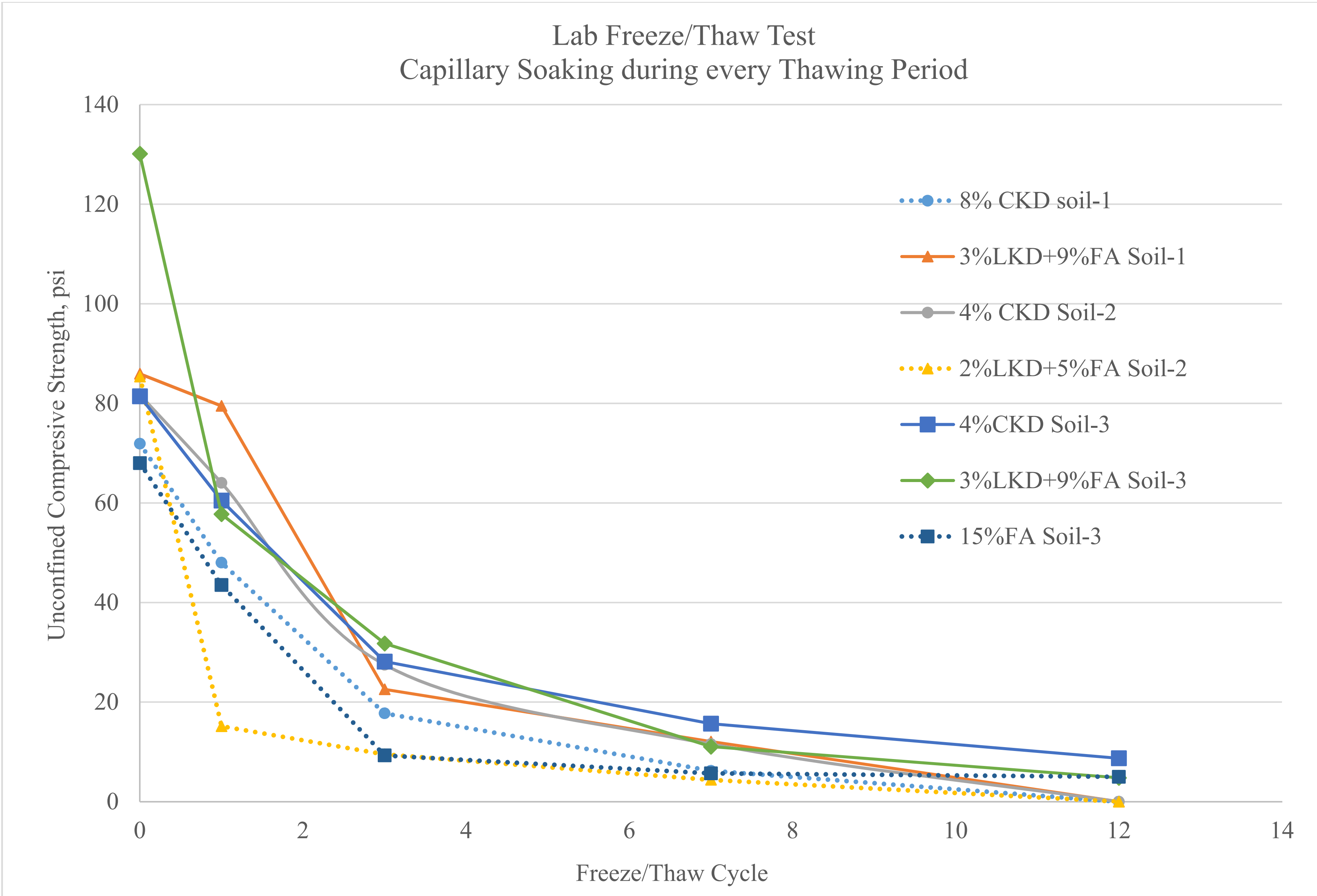


Figure 3.53: Reduction of UCS with Freeze/Thaw Cycles (24-hr Capillary Soaking during every Thawing Period)

3.8.2 Large-Scale Freeze/Thaw Test

As shown in the Section 3.7.1, a significant UCS loss was observed after few laboratory freeze/thaw cycles. Laboratory freeze/thaw tests are extremely harsh and can be considered overly conservative when compared to actual field conditions. A large-scale freeze/thaw testing program was designed to simulate the actual field conditions in a controlled laboratory environment. The tests were performed on compacted stabilized soil in a 3-foot by 7-foot container. The compacted soil depth was eight (8) inches (Figure 3.54).

Figure 3.54: Full Scale Freeze/Thaw Test Sample

Four containers were available for freeze/thaw testing. The following compacted soil mixes were used as the subgrade soils for small soil samples used for the testing.

1. Soil-1 (A-6) stabilized with 8%CKD
2. Soil-1 (A-6) stabilized with 3%LKD/9%FA
3. Soil-2 (A-4) stabilized with 4%CKD
4. Soil-2 (A-4) stabilized with 2%LKD/5%FA

The soil and recycled materials were mixed together using a mechanical mixer as shown in Figure 3.55. To obtain the OMC, they were compacted using plate compactor in order to achieve at least 95% dry density.

Figure 3.55: Mixing Soil-Recycled Materials (Down) and Compactor (Up)

Figure 3.55 shows the compactor used to compact soil in the containers. The compactor had 2275 lbs. of centrifugal force, weighed 132 lbs., and was powered with a 4HP engine. The plate size of the compactor was 13.5 inches x 2 inches. After mixing, the soil was placed into the container and compacted in four lifts. In situ density was measured using the sand cone method as shown in Figure 3.56. The expected and achieved density is shown in Table 3.71.

Figure 3.56: In situ Density Test Using Sand Cone Method

Table 3.71: Expected and Achieved Density of Compacted Soils in the Container

Soil	Treatment	MDD (pcf)	In situ Density (pcf)	Achieved Density %	OMC %	In situ Moisture Content %
Soil-1 (A-6)	8% CKD	103.10	82.45	79.97	15.73	22.62
	3% LKD/9% FA	103.26	83.34	80.70	16.89	20.04
Soil-2 (A-4)	4% CKD	112.36	90.08	80.17	13.78	16.35
	2% LKD/5% FA	114.27	98.00	85.76	13.76	16.98

The containers with compacted soil were kept on a 4-foot × 8-foot × 6-inch plastic tray. These trays were used to provide water for capillary soaking of the soil slab during the thawing period. An automated submersible pump was used to fill the trays with water during thawing cycle and a valve control outlet was used to drain the water out of the tray before the beginning of freezing cycle. A float valve was used to prevent overflow of water and maintain the water level so that it was always in contact with the soil. A timer was used to control the pump and the outlet valve. A schematic figure of the system is shown in Figure 3.57. After compaction, the soil in the container was kept in a humid room for seven days for curing. After the curing period, several drill holes were made in the soil slabs. Each hole was two inches in diameter and five inches depth. The holes were placed approximately eight inches from hole center to hole center.

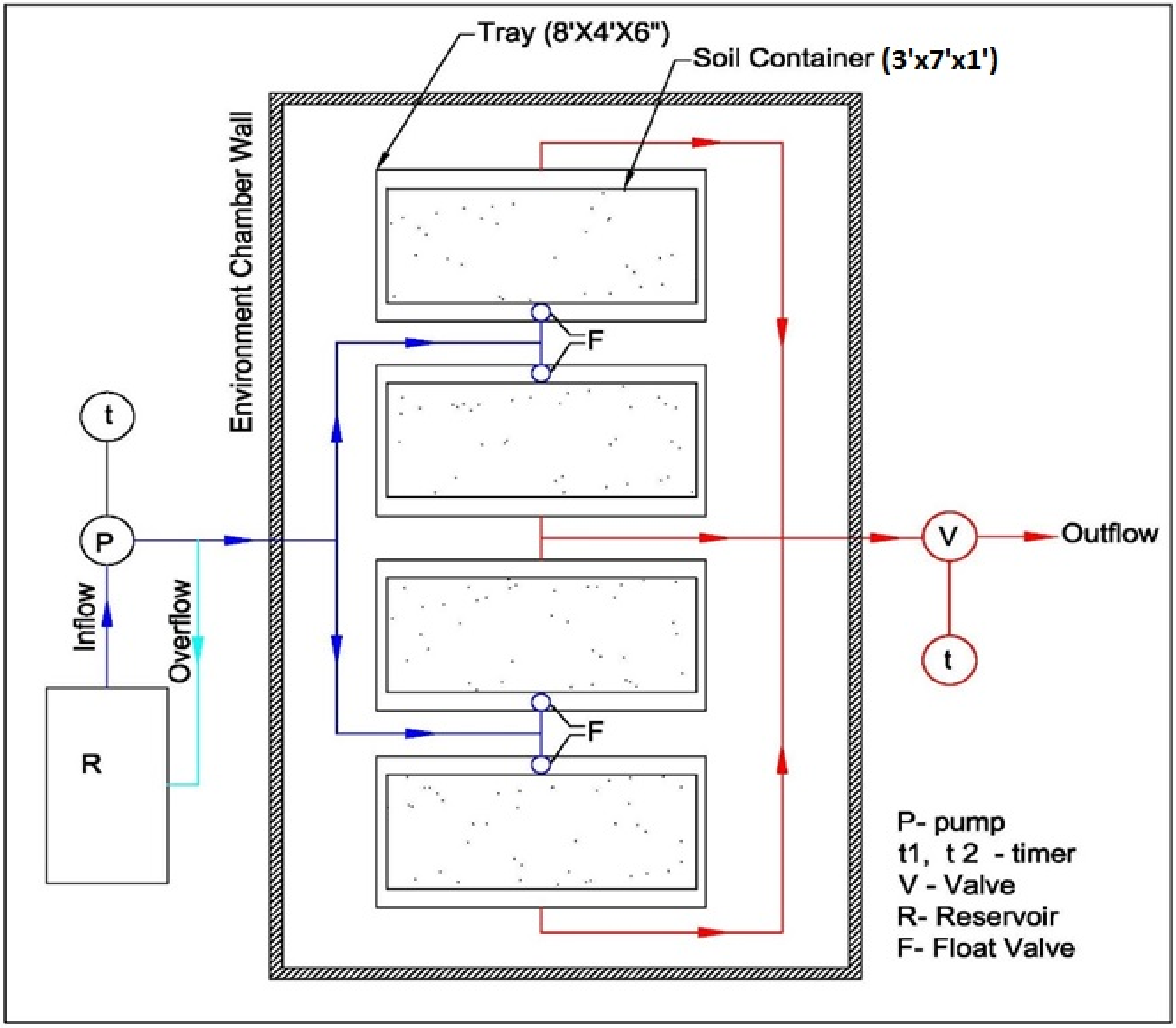


Figure 3.57: Schematic Diagram of Water Flow

Meanwhile, small soil samples (1.3125 inches in diameter and 2.816 inches in height) were compacted in the laboratory using the Harvard Miniature Compaction Apparatus. A total of 105 samples was prepared in the lab. Samples were compacted to achieve MDD. Dry density of all samples was calculated after compaction and the density of all samples was at least 95% of MDD. These samples were also kept in an open plastic bag and stored in an airtight, moisture proof bag with half-filled water at room temperature for curing. The curing process of these samples was described in Section 3.3.6. After curing period, the samples were wrapped with gauge fabric and placed into the pre-drilled holes in the soil slab and covered with approximately three inch soil on top. Each soil sample had approximately three inches of soil cover on top and bottom. Figure shows the placement of soil samples in the pre-drilled holes.

Figure 3.58: Wrapping & Placing Samples

Freeze/thaw cycles were started after the placement of soil into the pre-drilled holes in the soil slab. During compaction of the soil in the container, T-type thermocouples were placed at different depths into the soil slab to measure the temperature. The T-type thermocouple is a Copper versus Copper-Nickel wire with a temperature measuring range of 350°C to - 200°C (662°F to - 328°F) and 1.0°C or 1.5% limit of error below 0°C. An automated data acquisition system, as shown in Figure 3.59, was used to measure and save temperature readings every five minutes. The thermocouples were placed in the center and the edge of each container at mid depth. One container was equipped with thermocouples at every 2-inch depth at the center. Measured temperature readings are shown in Figure 3.60 and Figure for the stabilized soils at the optimum moisture content and prior to introduction of additional moisture through soaking. One freeze/thaw cycle included 24 hours of freezing at −10°F (−23°C) followed by 24 hours of thawing at 70°F (21°C). Unconfined Compressive Strength tests were performed on these samples after a number of freeze/thaw cycles. Results of these tests are shown in Figure 3.62 to 3.64.

Figure 3.59: Data Storing System

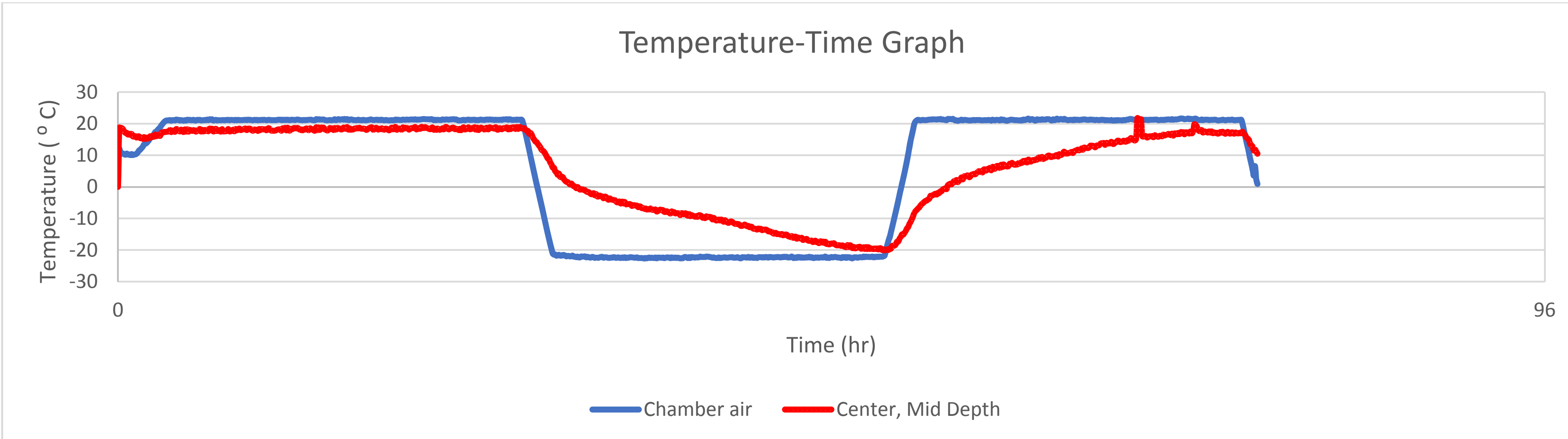


Figure 3.60: Soil Slab and Air Temperature

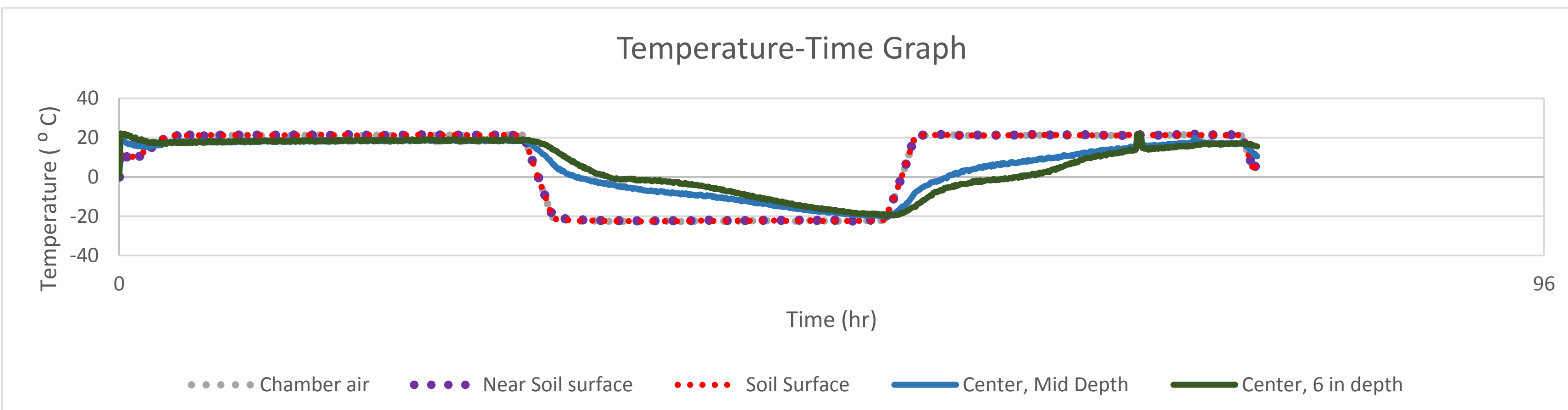


Figure 3.61: Soil Slab Temperature Variation with Depth

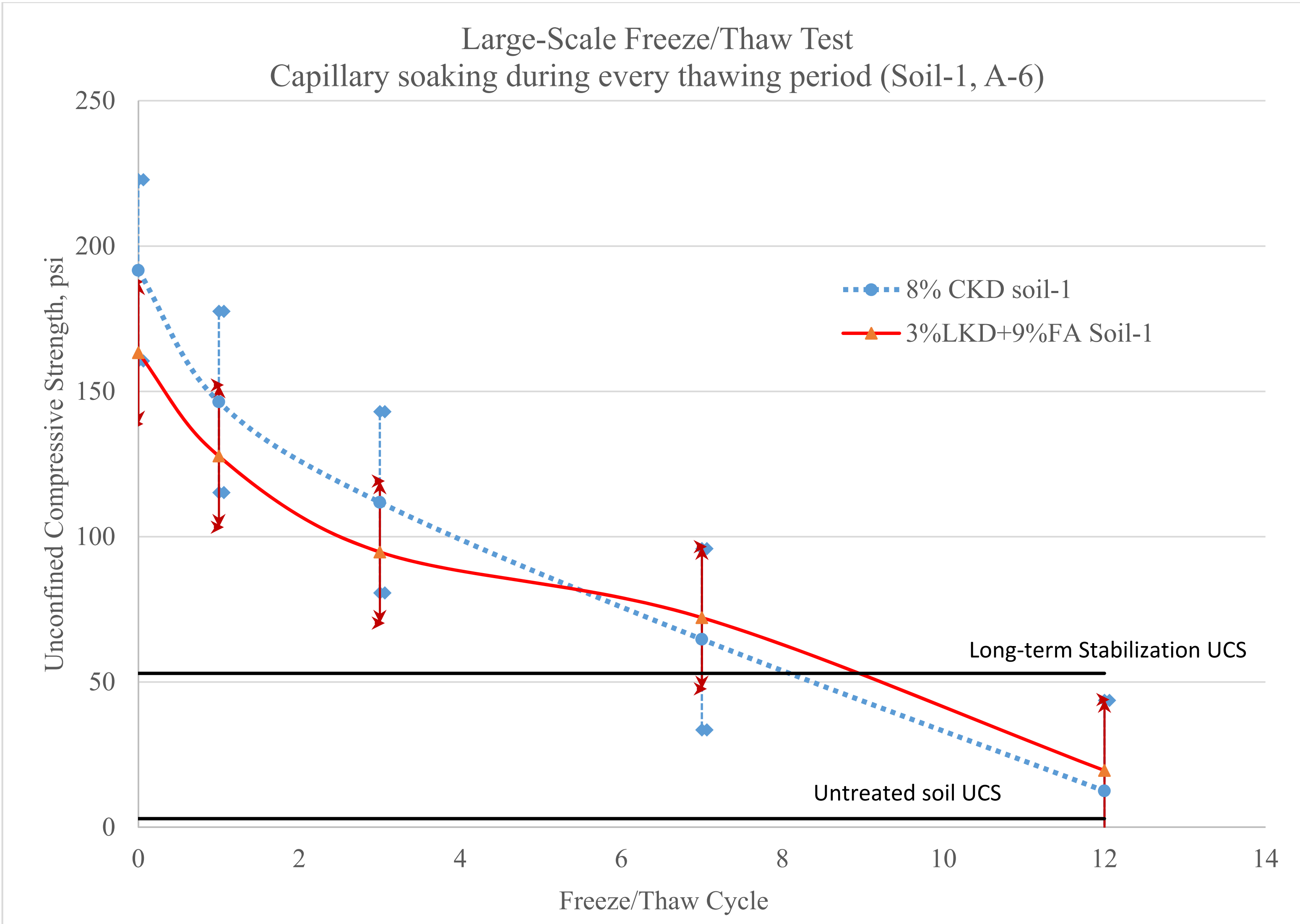


Figure 3.62: Reduction of UCS with Large-Scale Freeze/Thaw Cycles (Soil-1, A-6)

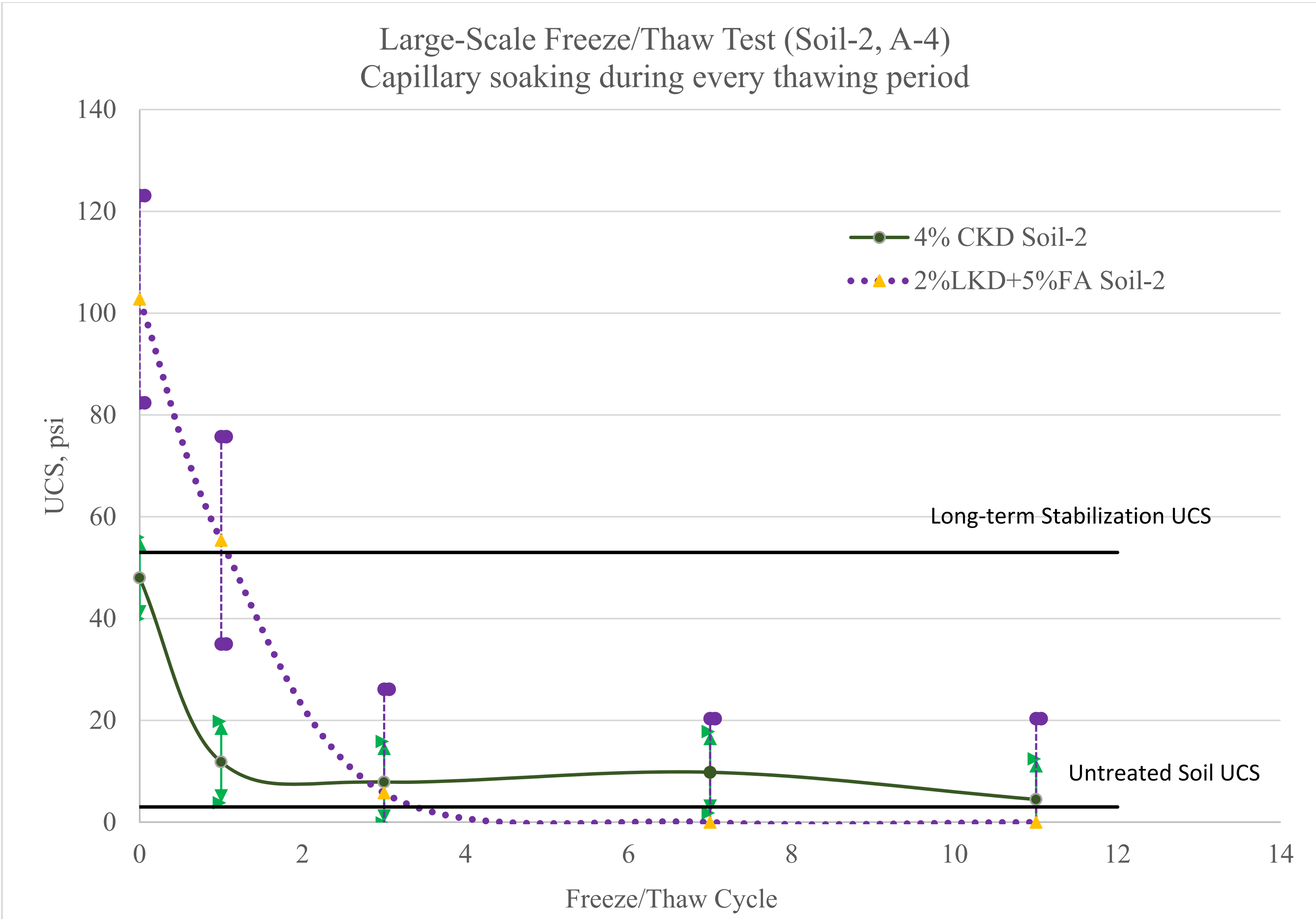


Figure 3.63: Reduction of UCS with Large Scale Freeze/Thaw Cycles (Soil-2, A-4)

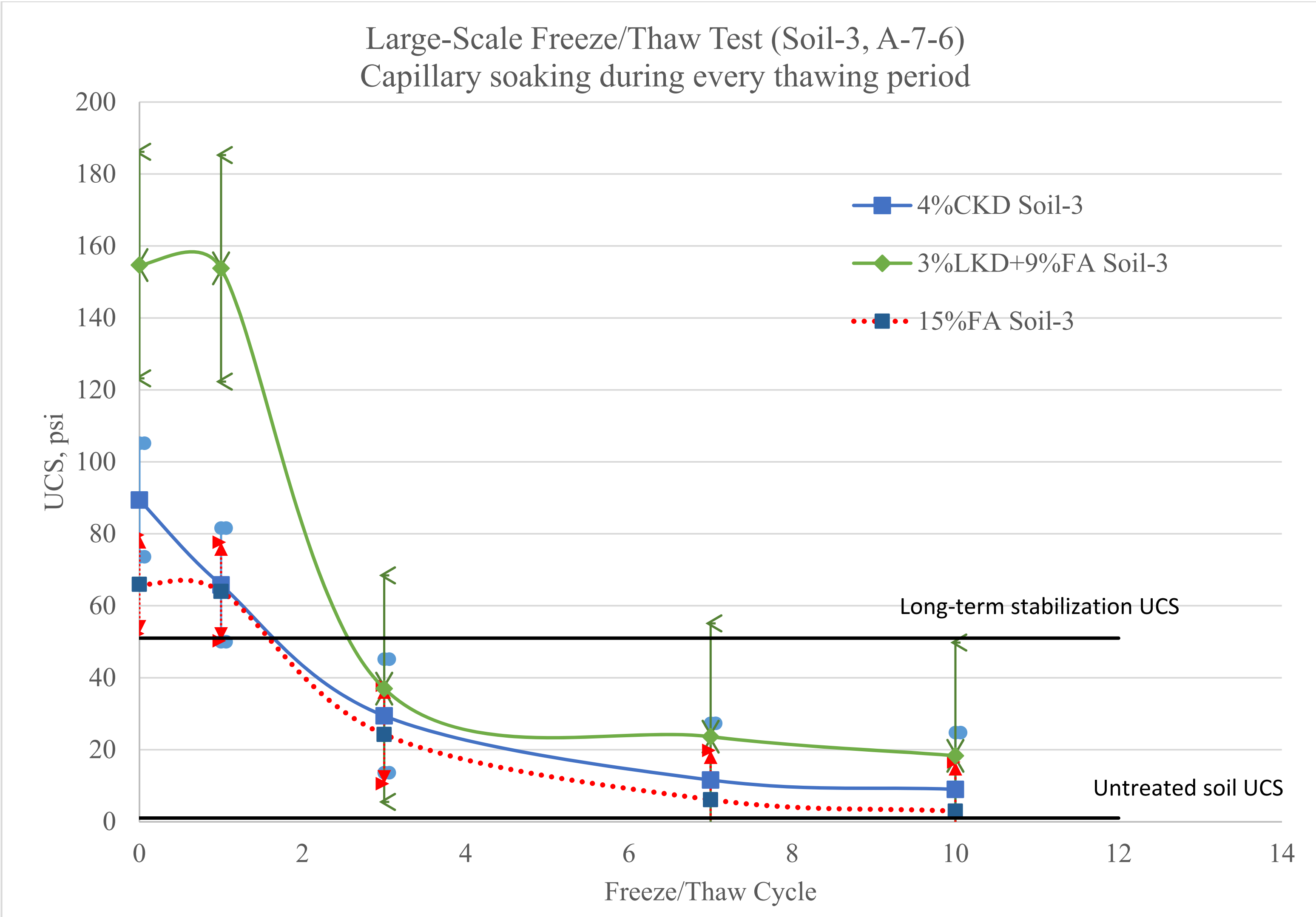


Figure 3.64: Reduction of UCS with Large-Scale Freeze/Thaw Cycles (Soil-3, A-7-6)

Large-scale freeze/thaw test data for Soil-1 (A-6), as shown in Figure 3.62, show that soil strength reduces significantly with continued freeze/thaw cycles. Even though the strength loss was significant, the treated samples retained more strength than the untreated soils after 12 freeze/thaw cycles. The required UCS needed for Soil-1 (A-6) and these recycled materials was 53 psi. This strength was observed after approximately eight cycles for the 8% CKD-treated soil and nine cycles for the 3% LKD/9% FA-treated soil.

Figure 3.63 shows the Soil-2 UCS test results obtained after the freeze/thaw cycles. In this case, 3% LKD/9% FA-stabilized Soil-2 (A-4) reached its effective strength of 53 psi after one freeze/thaw cycle. The strength of 4% CKD-treated soil without any freeze/thaw cycle was lower than the value determined previously in lab tests. It is possible that human error or sample variations caused this variation.

Large-scale freeze/thaw test results for Soil-3 (A-7-6), in Figure 3.64, show that the soil strength reduces to its effective limit of 51 psi within two to three cycles. It also does not reach its original strength of one psi even after 10 FT cycles.

Even though the strength loss was again significant, the treated samples retained more strength than the untreated soils after 10 freeze/thaw cycles.

Strength loss due to freeze/thaw was very acute. The capillary soaking water in the soil slab had a negative effect on this strength. Water introduced during the capillary soaking did not drain as expected with gravity. As such, additional water was introduced during every subsequent thawing cycle since the soil slab was soaked by capillary soaking for 24 hours until the cooling process started again. The water stayed in the soil slab during the freezing cycles and, therefore, turned into ice. The change in volume during the freezing and thawing of this water introduced a stress which was not considered in this experiment. Moreover, the moisture content kept increasing with every thawing cycle as the moisture from the previous cycle did not drain. Hence, every freeze/thaw cycle became harsher than the previous cycle.

The temperature range used in the large-scale freeze/thaw test (−10°F) was more extreme than the usual temperature range of subgrade soil seen in Michigan. Although, subgrade soil is technically covered with a few layers of pavement in the field, there was no cover used during testing. Moreover, heat transfer occurred from all directions as there was no insulation on the sides and the bottom of the soil slabs. The temperature change was also very fast compared to natural conditions (40°F/hour). Therefore, it can be said that the soil would retain more strength in the field. Nonetheless, all construction should be completed before the beginning of fall when temperatures start to decrease. This would prevent the stabilized subgrade from losing strength due to freeze/thaw conditions.

CHAPTER 4: REVIEW LONG-TERM PERFORMANCE OF STABILIZED SECTIONS

Long-term performance of stabilized pavement sections in Michigan and neighboring states provided field performance details of different stabilized materials under realistic moisture, environment, and traffic conditions. However, only a handful of projects were completed by MDOT on state highways. During the course of this study, the following MDOT projects with stabilized subgrades were identified (Table 4.1).

Table 4.1: MDOT Subgrade Stabilization Projects

Project	Material(s) Used	Construction Year
I-96 from Schaefer to M-39, Wayne County, MI	Lime	2005
I-75/I-96 from Vernor to Michigan, Wayne County, MI	Lime, Lime/fly ash, CKD	2008
M-84, Bay and Saginaw County, MI	Lime, Lime/fly ash	2010

Other soil stabilization projects conducted by MDOT, as well as county, city or commercial entities, were identified using the contacts made during the interview portion of this study. Wadel Stabilization, Inc. provided a list with city, county, and commercial projects for consideration. Project selection was based on input from MDOT Project Manager (PM) and Research Advisory Panel (RAP) members.

The neighboring states of Ohio, Indiana, Wisconsin, and Minnesota have completed hundreds of soil stabilization projects. The research team had access to some of these projects since the sub-contractor, Soil and Materials Engineers, Inc. (SME) completed the majority of mix designs and construction inspections for these projects. Based on discussions with Carmeuse Lime and Stone personnel, the Indiana Department of Transportation (INDOT) completed 155 LKD stabilization projects in 2012 alone. Additionally, recent studies conducted on the performance of stabilized subgrades in Ohio, Wisconsin, and Minnesota by local Departments of Transportation (DOT) were summarized in the literature review section of this report.

Representative in-situ pavement sections were selected after identifying suitable projects and reviewing respective construction details. The in-situ pavement layer properties were then assessed on each section using a Field Evaluation Program.

4.1 Field Evaluation Program

A Field Evaluation Program consists of coring, DCP testing, and FWD testing on selected stabilized projects within a representative pavement section. More details on each field testing task are given in the following sections.

4.1.1 Coring and Dynamic Cone Penetrometer (DCP) Testing

Subgrade strength improvement due to stabilization was measured by DCP testing pursuant to *ASTM D 6951*. DCP measures the resistance to penetration due to an impact load applied via a rod. The penetration per blow value was used to estimate the in-situ CBR using a correlation developed by the U.S. Army Corps of Engineers. DCP measurements also generated a thickness log of the stabilized layer and in-situ soil stiffness results based on the resistance to penetration values.

Pavement coring was performed using a truck-mounted hydraulic drill. At each location, a 4- inch diameter pavement core was removed and a hand auger was used to remove the underlying aggregate base layers. These removal techniques allowed the DCP testing to initiate at the top of the stabilized subgrade.

After the pavement coring and hand auger borings were completed, DCP tests were conducted at each test location. The tests were performed at least two feet below the stabilized layer into the in situ subgrade material. In some cases though, the DCP rod could not advance past the stabilized layer due to extremely hard materials. If the DCP rod did not advance after a few drops, the test was stopped due to impenetrable conditions.

A typical DCP plot of a stabilized layer is shown in Figure 4.1. DCP plots were created for each test location to determine the stiffness in terms of CBR for the stabilized layer and the in-situ soil layer thickness. The stabilized layer thickness was estimated by equating the bottom of the stabilized layer to intersect with the depth where a drastic decrease in soil stiffness was observed as shown in Figure 4.1.

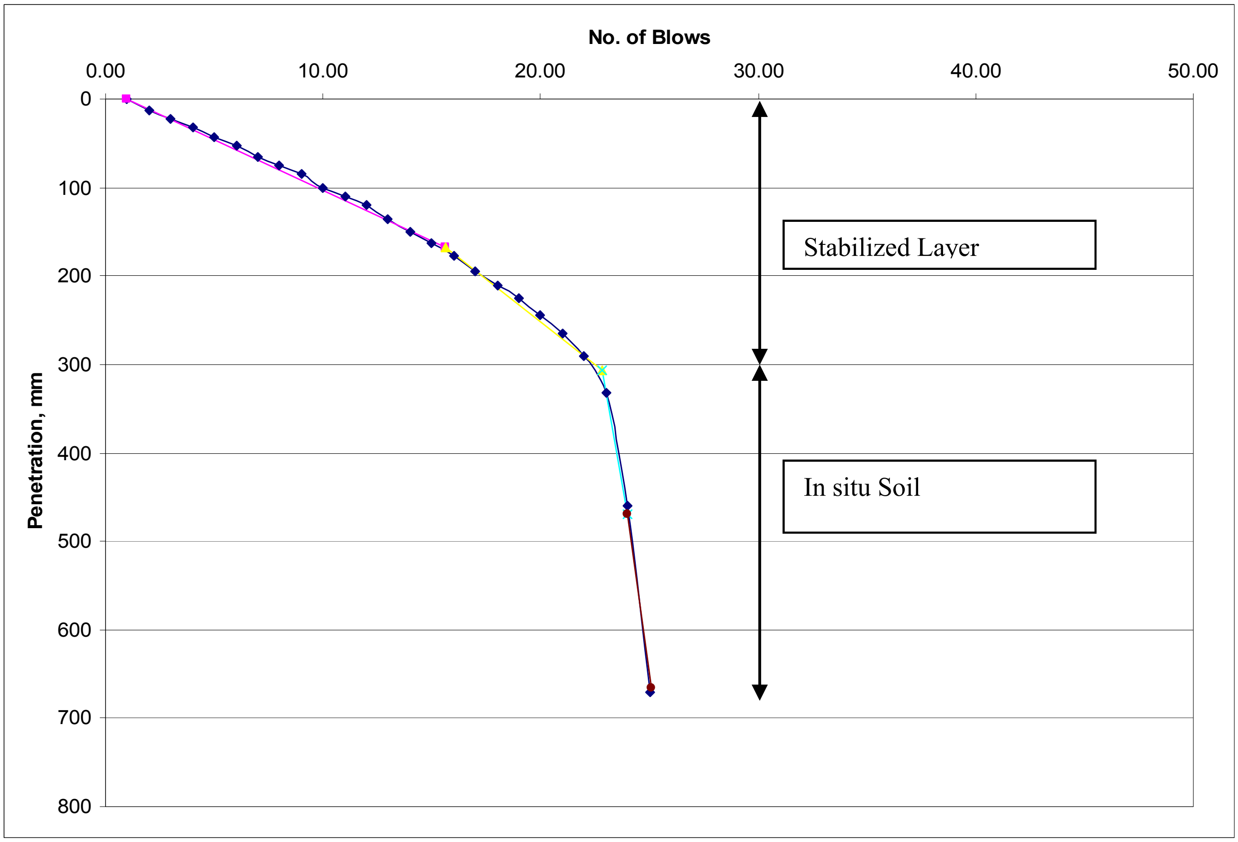


Figure 4.1: Typical DCP Results Plot for a Stabilized Subgrade

4.1.2 Falling Weight Deflectometer (FWD) Testing

FWD is one of the most reliable non-destructive test methods for determining the structural condition of in-service pavements. FWD data can be used to determine structural properties of pavement and stiffness of subgrade. A Dynatest FWD, owned and operated by Soil and Materials Engineers, Inc. (SME), was used throughout the field investigation. FWD data were collected on selected projects at 50-foot intervals along a 500- foot test section. Three load levels were initially used: 9,000 lbs (pounds), 12,000 lbs, and 15,000 lbs. These load levels were adjusted based on individual pavement structure thickness.

4.2 Identify Pavement Sections with Stabilized Subgrades in Michigan and Neighboring States

Based on the discussions with MDOT, stabilization contractors in Michigan, and stabilization engineering consultants, the following projects were identified for field data collection (Table 4.2).

Table 4.2: Pavement Sections Selected for Field Data Collection

Project	Stabilization Material Used	Construction Year
I-75/I-96 from Vernor to Michigan, Wayne County, MI	Lime, Lime/fly ash, CKD	2008
M-84, Bay and Saginaw Counties, MI	Lime, Lime/fly ash	2010
Waverly Road, Ingham County, MI	CKD	2010
SR310/US40 Licking County, OH	LKD	2008

4.3 Field Data Collection

4.3.1 I-75/I-96 in Wayne County, MI

I-75/I-96 in Wayne County, Michigan, was a concrete reconstruction project completed in 2008. Lime stabilization was included in this project due to extremely weak subgrade soil conditions. Two test sections with CKD-stabilized subgrade were constructed as a part of this project for side-by-side comparison with lime-stabilized subgrade. A research report published by MDOT (Bandara, 2009) showed substantial short term strength gain due to lime and CKD stabilization. On average, the CKD-stabilized areas showed 885% percent strength gain over untreated soil strength while lime-stabilized areas showed 531% strength gain. This report recommended performing a long term subgrade performance study to evaluate the effect of freeze/thaw cycles and subgrade moisture movement on subgrade stiffness.

This project site was divided into five different areas with different stabilization materials as shown in Table 4.3 and Figure 4.2. At the time of this study, the field samples were six years old.

Table 4.3: I-75/I-96 Test Areas

Test Area	Direction*	Start Station	End Station	Length (feet)	Stabilization Materials
1	NB	1250+32	1260+40	1008	CKD for 12 inches
2	NB	1263+00	1269+43	643	CKD for 12 inches
3	NB	1271+50	1278+00	650	Lime for 18 inches
4	SB	1222+47	1226+57	410	Lime/fly ash for 12 inches
5	SB	1258+68	1265+63	695	Lime for 12 inches
* NB – Northbound, SB – Southbound					

Figure 4.2: General Site Layout of I-75/I-96 Test Areas

Figure 4.3: General Site Overview of I-75/I-96 Site

DCP tests were performed in each test area along the outside shoulder. After coring the concrete pavement, hand auger borings were performed to reach the top of the stabilized subgrade layer. DCP tests were initiated from the top of the stabilized subgrade. Table 4.4 shows observed concrete (PCC) and base thicknesses from the cores and hand auger borings. The pavement thicknesses are for the shoulder pavement section. The mainline pavement of I-75 is 13 inches of Portland Cement Concrete (PCC) over 16 inches of aggregate base. The mainline pavement of I-96 is 12.5 inches of PCC over 16 inches of aggregate base.

Based on the collected DCP test data, the Penetration Rate (penetration per blow, DCP) was calculated for each depth. These values were then converted to CBR using Equation 4.1 established by U. S. Army Corps of Engineers (USACOE, 1992). The CBR test results are as shown in Table 4.5.

$$CBR = \frac{292}{DCP^{1.12}} \qquad \text{(Equation 4.1)}$$

Table 4.4: Pavement Core and Hand Augur Boring Results for I-75/I-96

Test Area	Test Hole Number	Direction	Distance (feet)	Offset (feet)	Thickness (inches)	
					PCC	Base
1	1	NB	50	5.0 R	11.4	17.5
	2	NB	409	5.0 R	11.2	18.8
	3	NB	853	5.6 R	10.9	14.3
2	4	NB	1363	5.0 R	12.0	16.8
	5	NB	1598	5.0 R	11.2	14.0
	6	NB	1850	4.7 R	11.4	16.0
3	7	NB	2193	5.8 R	10.9	16.9
	8	NB	2397	4.1 R	11.4	16.8
	9	NB	2692	4.9 R	9.7	19.1
4	10	SB	2108	5.1 L	10.9	17.9
	11	SB	2250	4.8 L	12.0	18.4
	12	SB	2392	4.9 L	12.1	18.5
5	13	SB	615	4.8 L	11.5	16.6
	14	SB	859	4.7 L	10.9	16.8
	15	SB	1102	4.9 L	9.8	17.9

Table 4.5: DCP Test Results for I-75/I-96 Site

Test Area	Test Hole Number	CBR (%)		Stabilized Depth (inches)	Average CBR (%)			Average Stabilized Depth (inches)
		Stabilized	In situ		Stabilized	In situ	% Increase	
1	1	39.0	32.8	11.4	46.7	23.3	200.7	11.4
	2	5.0	7.0	9.8				
	3	96.1	30.0	13.1				
2	4	73.2	21.1	12.0	68.4	60.6	112.9	12.5
	5	75.7	100.0	12.0				
	6	56.3	N/A	13.5				
3	7	94.2	8.0	10.6	92.5	26.0	355.9	9.8
	8	89.0	35.0	9.1				
	9	94.4	35.0	9.8				
4	10	90.8	25.0	16.3	94.1	41.7	225.8	14.6
	11	100.0	60.0	12.5				
	12	91.5	40.0	14.9				
5	13	7.0	40.0	12.7	55.8	27.5	202.9	10.5
	14	80.0	15.0	6.7				
	15	80.4	N/A	12.0				

FWD Testing was performed at the center of the slabs in the inside lane and the shoulder of each test area. Testing was performed in approximately 50-foot intervals (or every third slab). Lane 1 was designated as the inside lane, while Lane 2 was designated as the shoulder. The FWD sensors were placed at 0, 8, 12, 18, 24, 36, and 60 inches from the center of the load plate. The 11.8-inch diameter load plate was used to apply the load. Two 9,000-lb seating loads were applied at each test location before performing the test sequence. The test sequence at each test location consisted of recording deflections for 9,000, 15,000, and 32,000 lbf.

Figures 4.4 and 4.5 show the deflection plots for different areas at the center of the load plate (D0) and 60 inches away from the load plate (D60). A 32,000-lb load for the inside lane and the shoulder lane of I-75/I-96 was utilized.

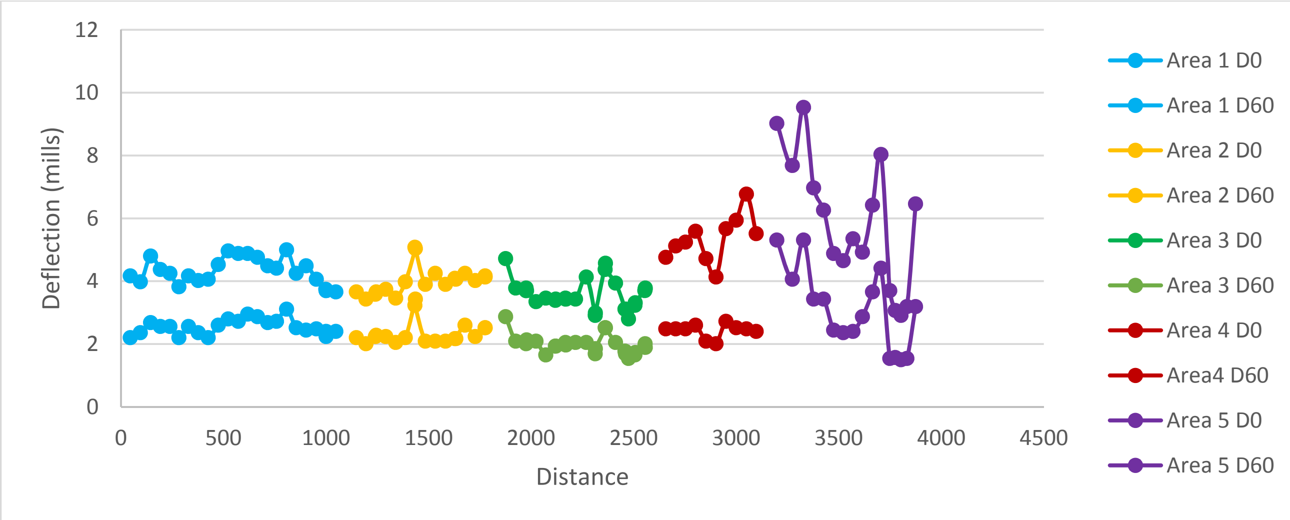


Figure 4.4: Deflection Plots for Inside Lane of I-75/I-96

Higher deflections at Area 5 were expected due to the difference in concrete pavement thickness values. Area 5 is located in I-96 where the concrete pavement thickness is 12.5 inches compared to 13 inches in other areas.

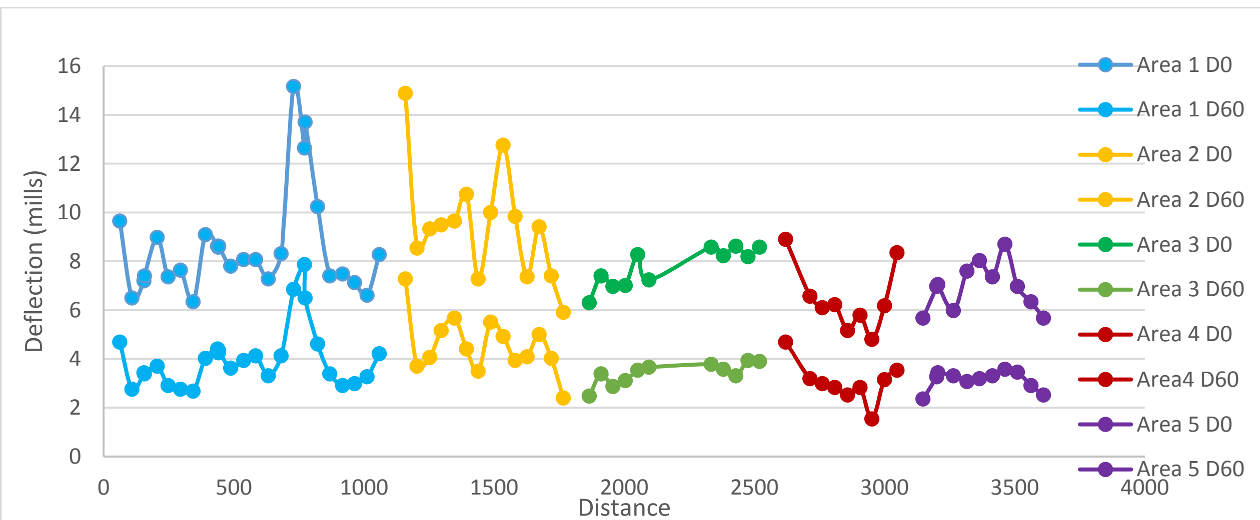


Figure 4.5: Deflection Plots for Shoulder Lane of I-75/I-96

Higher deflections were noticed for the shoulder lane when compared to the inside lane of I-75/I-96. This is most likely due to thickness differences in the pavement section. The I-75/I-96 mainline pavement consists of 13 inches of concrete pavement followed by 16 inches of open graded aggregate base. The I-75/I-96 shoulder pavement has a concrete pavement thickness varying from 10 inches to 13 inches at the valley gutter.

4.3.2 M-84, Bay and Saginaw Counties, MI

M-84 was constructed in 2010. Lime stabilization was included due to poor subgrade conditions. During construction, the stabilization of the northbound lanes of M-84 between Hotchkiss Road and Salzburg Road in Bay City was changed from lime to LKD. This section of M-84 was selected for a side-by-side evaluation to compare the lime-stabilized subgrade with the LKD-stabilized subgrade after five years of use since construction in 2010. The section consists of two lanes in each direction with a center turning lane. Coring, DCP testing, and FWD testing were performed along travel lanes of the northbound and southbound lanes. Figure 4.6 shows the general view of the test site. As shown, no visible pavement distress is present after five years of use.

Figure 4.6: General Site Overview, M-84 in Bay City, Michigan

According to the construction documents, this pavement section consists of 7.75 inches of Asphalt Pavement (5E3 at 165 lb, 1.5 inches; 4E3 at 275 lb, 2.5 inches, 3E3 at 410 lb, 3.75 inches) followed by six inches of aggregate base and 18 inches of sand subbase.

DCP tests were performed on the paved shoulder between the white edge strip and the concrete gutter. DCP started approximately below the bottom of the subbase. The following table shows the core and DCP locations with associated test depths.

Table 4.6: Core and DCP locations of M-84 Site

Probe	Station	Direction	Offset (feet)	Orientation*	Depth at Start of DCP (inches)	Test Depth (inches)
C1	711+49	NB	13	E of CL	29	77
C2	716+49	NB	14	E of CL	30	77
C3	721+49	NB	14	E of CL	30	77
C4	726+50	SB	16	W of CL	30	78
C5	721+50	SB	14	W of CL	30	73
C6	716+50	SB	16	W of CL	30	77
* East (E), West (W), Center Line (CL)						

Three DCP tests were performed in each test area (southbound and northbound lanes) after coring of the asphalt pavement surface and aggregate base to the top of the stabilized base. Table 4.7 shows the DCP test results for this site.

Table 4.7: DCP Test Results for M-84 Site

Test Area (Direction , Material)	Test Hole Number	CBR (%)		Stabilized Depth (inches)	Average CBR (%)			Average Stabilized Depth (inches)
		Stabilized	In situ		Stabilized	In situ	% Increase	
1 (NB M-84, LKD)	1	29.6	9.3	17.7	23.2	15.6	148.7	13.3
	2	21.1	16.0	13.6				
	3	18.9	21.5	8.7				
2 (SB M-84, lime)	4	50.6	16.3	18.9	39.6	29.8	132.9	18.6
	5	28.4	56.2	16.5				
	6	39.8	16.8	20.5				

FWD testing was conducted over a distance of approximately 1,950 feet in the northbound and southbound directions along the right wheel path of each lane. Northbound FWD tests were started at 1,641 feet north of centerline of Red Feather Drive and southbound FWD tests were started at 1,678 feet south of centerline of Christopher Court. At each test location, two seating drops were first performed by applying a target load of 9,000 lb. Thereafter, testing at each test location was performed at target load levels of 9,000, 12,000, and 24,000 lb. The deflections were measured at distances of 0, 8, 12, 18, 24, 36, and 60 inches from the center of the load plate.

Figures 4.7 shows the deflection plots for the different test areas at center of the load plate (D0) and 60 inches away from the load plate (D60). A 9,000-lb load for both the northbound and southbound lanes was utilized

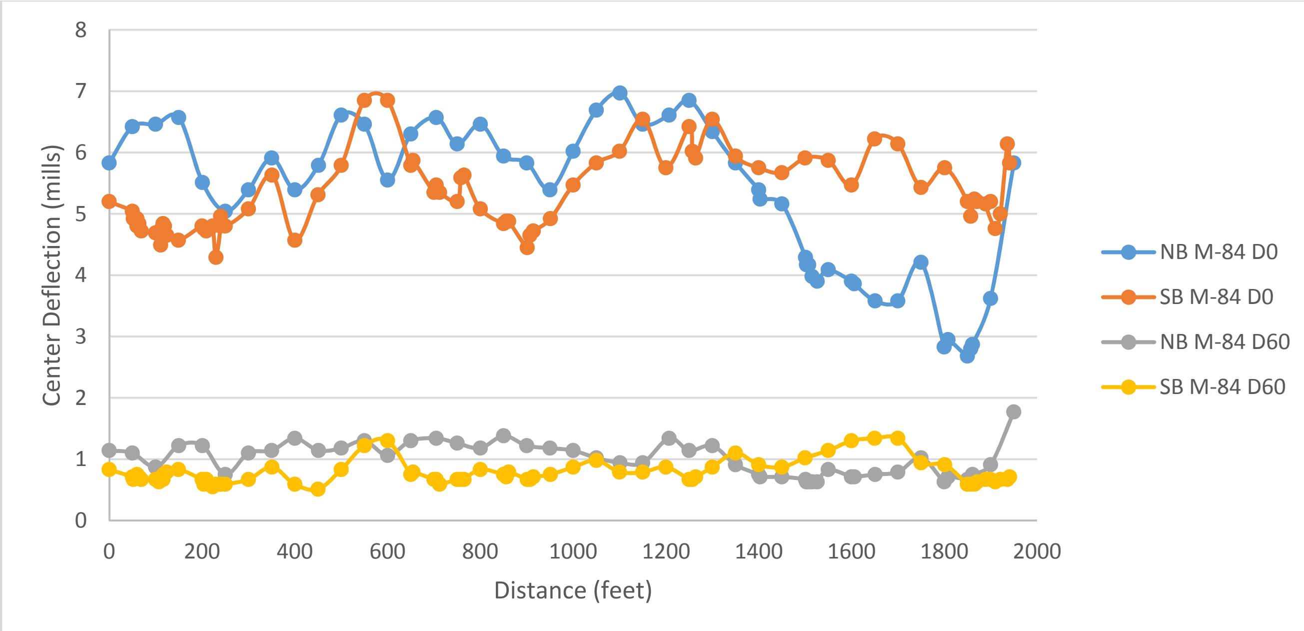


Figure 4.7: Deflection Plots for M-84

4.3.3 Waverly Road, Ingham County, MI

Waverly Road was constructed in 2010 and CKD stabilization was included in a section of the road due to poor subgrade conditions. Based on construction records, CKD stabilization was performed for a depth of 12 inches with a CKD application rate of 5% by weight of soil. Like M-84, Waverly Road has been in use for five years since construction. The selected section of Waverly Road consists of two lanes in each direction and a center turning lane. Coring, DCP testing, and FWD testing were performed along the northbound and southbound travel lanes.

Based on the construction documents, the pavement section consists of three to four inches of Hot Mix Asphalt (HMA), five inches of asphalt-stabilized base, six inches of aggregate base, nine inches of subbase and 12 inches of CKD-stabilized subgrade.

DCP tests were performed at four locations on the shoulder as detailed in Table 4.8.

Table 4.8: Core and DCP Locations of Waverly Road

Probe	Station	Direction	Offset (feet)	Orientation	Depth at Start of DCP (inches)	Test Depth (inches)
TH1	55+00	NB	15	E of CL	3.5	76
TH2	57+00	NB	14	E of CL	4.5	79
TH3	57+50	SB	14	W of CL	5.5	78
TH4	56+00	SB	15	W of CL	4.0	76

A general view of the Waverly Road test site is shown in Figure 4.8. Constructed five years ago, the pavement shows no signs of visible distress.

Figure 4.8: General Site Overview, Waverly Road, Ingham County, Michigan

DCP testing started at depths ranging from 3.5 to 5.5 inches. As stated in the construction documents, the pavement includes three to four inches of HMA and five inches of asphalt-stabilized base. However, during the DCP testing, the asphalt-stabilized layer was not detected.

Based on the specified pavement thickness, an aggregate base layer should have been present from nine inches to 15 inches and a subbase layer should have been detected from 15 inches to 24 inches. The CBR results of the layer from nine inches to 15 inches ranged from 18 to 26. The CBR results of the layer from 19 inches to 24 inches ranged from 1.1 to 41.3 with an average value of 13.5. Only one test hole, Test Hole Number 2 (TH2), showed evidence of stabilization. At TH2, a CBR of 100 was recorded from 23 inches to 30 inches while a CBR of 75 was recorded from 30 inches to 36 inches.

Average DCP test results for Waverly Road are shown in Table 4.9. As stated earlier and as seen in Table 4.9, only TH2 shows evidence of stabilization. Based on the very poor sand subbase CBR values, it can be concluded moisture could be present in the subbase and the stabilized layers of subgrade. The high moisture contents may have degraded the stiffness values of CKD stabilized layer showing poor subgrade conditions.

Table 4.9: DCP Test Results for Waverly Road

Test Hole Number	Base CBR %	Subbase CBR %	Stabilized Subgrade CBR %	In situ Subgrade CBR %	Average CBR		
					Stabilized %	In situ %	% Increase over In situ
1	17.9	2.2	3.2	45.4	87.5*	23.4	373.9
2	25.5	41.3	87.5	17.0			
3	20.2	09.4	1.7	2.5			
4	22.7	1.1	12.7	28.9			
*Only Test Hole Number 2 is considered for stabilized CBR value.							

FWD testing on a 350 foot long section of Waverly Road was performed on November 17, 2014. Testing commenced at Station 54+50 which is 100 feet north of Wilbur Highway. Testing was performed at an approximate spacing of 10-foot intervals. Tests on the southbound lanes were offset 5 feet from the tests that were performed on the northbound lanes. At each test location, two seating drops were first performed by applying a target load of 9,000 lb. Thereafter, testing at each test location was performed by using target load levels of, 9,000, 12,000, and 24,000 lb. The deflections were measured at distances of 0, 8, 12, 18, 24, 36, and 60 inches from the center of the load plate.

Figure 4.9 shows the deflection plots for different areas at the center of the load plate (D0) and 60 inches away from the load plate (D60) at 9,000 lb load.

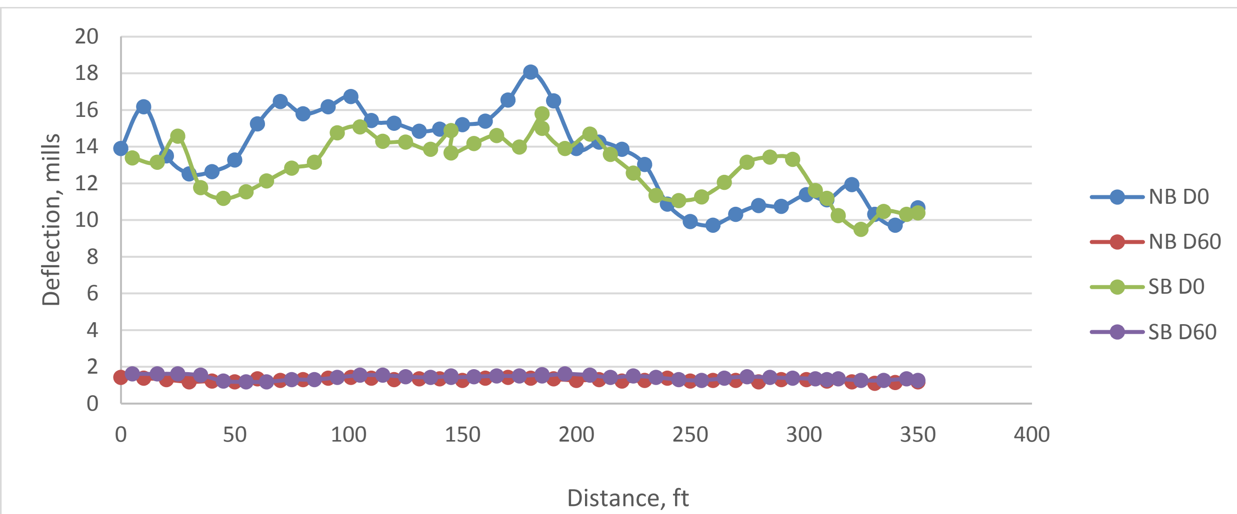


Figure 4.9: Deflection Plots for Waverly Road

4.3.4 SR 310, Licking County, OH

Constructed in 2008, this two lane road is 36 feet wide with a center turning lane. Shoulders are six feet wide. Based on the construction documents, the pavement section consists of 9.25 inches of HMA surface (1.5 inches HMA surface course, 1.75 inches HMA intermediate course, six inches HMA base) followed by 14 inches of LKD-stabilized subgrade. In the test area, 8% LKD stabilization was utilized. Based on the construction plan boring sheets, the existing subgrade generally consisted of sandy silts and silty clays.

A general view of the test site is shown in Figure 4.10. In use for seven years, the road has evidence of a few longitudinal cracks.

Figure 4.10: General Site Overview SR 310

FWD testing started approximately 285 feet north of the US 41 centerline. This location was designated as Distance “zero” location. The distance where the DCP test was performed is given with a reference to this Distance “zero” location. DCP testing was performed from six inches to 12 inches outside of the paved shoulder. Plans indicated that the stabilization was utilized to a distance of 18 inches beyond the asphalt base. The unpaved shoulder area consisted of compacted soil up to a depth of 9.25 inches followed by approximately 14 inches of LKD-stabilized soil. DCP tests were performed at four locations on the shoulder as shown in Table 4.10.

Table 4.10: Core and DCP Locations of SR 310

Probe	Station*	Direction	Offset (inches)	Orientation	Depth at Start of DCP (inches)	Test Depth (inches)
TH1	115	NB	6	E of E Edge of Shoulder	0	77
TH2	415	NB	6	E of E Edge of Shoulder	0	72
TH3	415	NB	6	W of W Edge of Shoulder	0	79
TH4	215	SB	6	W of W Edge of Shoulder	0	79
*Distance: Distance Relative to FWD Start Location						

Table 4.11: DCP Test Results for SR 310 Site

Test Hole Number	Compacted Soil Shoulder CBR (%)	Stabilized Subgrade CBR (%)	Stabilized Layer Thickness (inches)	In situ Subgrade CBR (%)	Average CBR (%)		
					Stabilized	In situ	% Increase
TH1	2.1	50.1	16.6	9.9	49.8	19.4	256.7
TH2	7.6	80.2	16.9	23.7			
TH3	2.6	73.0	9.7	31.7			
TH4	0.4	61.8	15.0	12.3			

FWD testing on a 530-foot long section of SR-310 located north of US 40 in Etna, Ohio, was performed on October 21, 2014. The beginning of this 530-foot long section was approximately 285 feet north of US-41. This location was designated as Distance "zero" location (D0). Testing was performed along the right wheel path in both directions at a spacing of 10 foot intervals. Testing at each test location was performed at target load levels of 9,000, 12,000, and 24,000 lb. The deflections were measured at distances of 0, 8, 12, 18, 24, 36, and 60 inches from the center of the load plate.

Figure 4.11 below shows the deflection plots for different areas at center of the load plate (D0) and 60 inches away from the load plate (D60) at 9,000 lb load for SR 310.

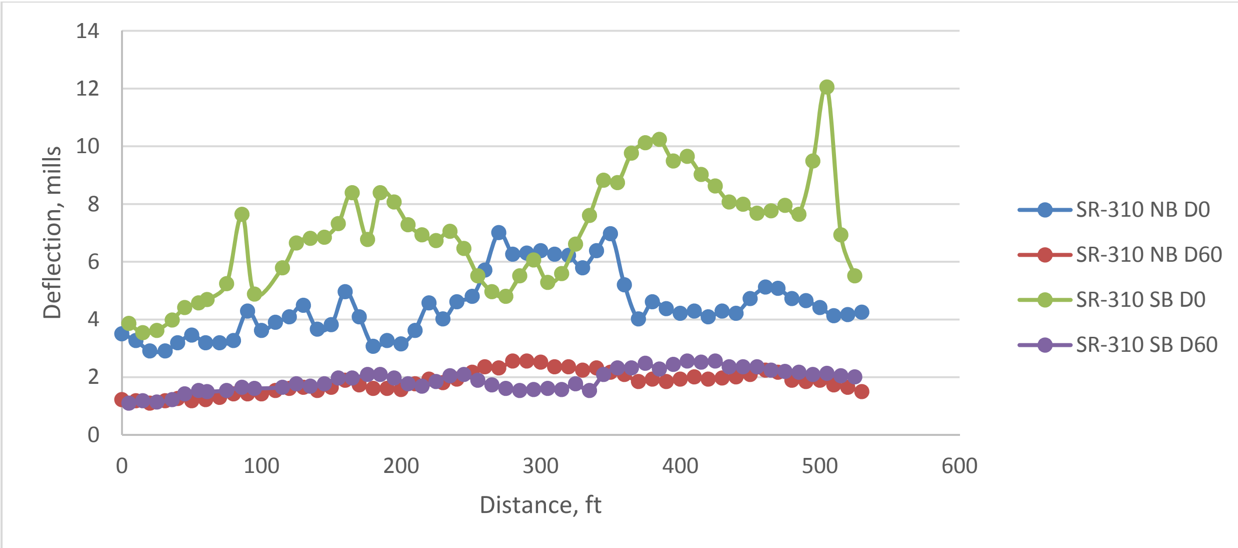


Figure 4.11: FWD Deflection Plots for SR 310

4.4 FWD Data Analysis

FWD data analyses were performed to estimate the elastic modulus of pavement layers for flexible pavements and effective modulus of subgrade reaction for rigid pavements. ILLI-BACK back calculation was used for rigid pavements and the method given in *AASHTO 1993 Guide for Design of Pavement Structures* (AASHTO, 1993) was used to calculate layer coefficients for the stabilized layer. The following sections present results of the FWD data analysis for each test pavement section.

4.4.1 FWD Data Back Calculation

4.4.2 Structural Layer Coefficient Calculations using FWD Data

The *AASHTO 1993 Guide for the Design of Pavement Structures* (AASHTO, 1993) provides a method to calculate structural layer coefficients from FWD data. This method was developed for evaluating pavement structures for rehabilitation planning. The first step of this calculation process is to estimate the subgrade resilient modulus (M_R) using the following equation:

$$M_R = \frac{0.24P}{d_r r} \qquad \textbf{(Equation 4.2)}$$

Where,

M_R = subgrade resilient modulus (psi)

P = applied load (lbs)

d_r = deflection at a distance r from the center of the load (inches)

r = distance from the center of load (inches)

The deflections used in the back calculation of M_R must be measured from a minimum distance from the center of the load plate to be independent from the effects of the pavement layers. The minimum distance (*r*) can be determined from the following equation:

$$r \geq 0.7a_e \qquad \text{(Equation 4.3)}$$

Where,

$$a_e = \sqrt{\left[a^2 + \left(D\sqrt[3]{\frac{E_p}{M_r}}\right)^2\right]} \qquad \text{(Equation 4.4)}$$

a_e = radius of the stress bulb at the subgrade-pavement interface (inches)

a = FWD load plate radius (inches)

D = total thickness of pavement layers above the subgrade (inches)

E_p = effective modulus of all pavement layers above the subgrade as estimated below (psi)

$$d_0 = 1.5pa\left\{\frac{1}{M_R\sqrt{1+\left(\frac{D}{a}\sqrt[3]{\frac{E_p}{M_R}}\right)^2}}\right\} + \frac{\left[1-\frac{1}{\sqrt{1+\left(\frac{D}{a}\right)^2}}\right]}{E_p} \qquad \text{(Equation 4.5)}$$

Where,

d_0 = deflection measured at the center of the load plate and adjusted to a standard temperature of 68°F (inches)

p = FWD load plate pressure (psi)

Once the subgrade modulus and effective modulus are estimated, the following AASHTO equation was used to calculate the effective structural number (SN_{eff}) of the pavement:

$$SN_{eff} = 0.0045D\sqrt[3]{E_p} \qquad \text{(Equation 4.6)}$$

After the SN_{eff} of the pavement is estimated, the following equation was used to calculate the structural layer coefficient of the stabilized subgrade (a_3):

$$SN_{eff} = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3$$

(Equation 4.7)

And solving for a_3:

$$a_3 = \frac{SN_{eff} - a_1 D_1 + a_2 D_2 m_2}{D_3 m_3}$$

(Equation 4.8)

Where,

a_1 = structural layer coefficient of the asphalt layer (assumed 0.42)

a_2 = structural layer coefficient of the base and subbase layer (assumed 0.10)

m_2 = drainage coefficient of the base and subbase layer (assumed 1.0)

a_3 = structural layer coefficient of the stabilized subgrade layer

m_3 = drainage coefficient of the stabilized subgrade layer (assumed 1.0)

Based on the above procedure, the pavement subgrade modulus, effective modulus of pavement layers, effective structural number of the pavement section, and the structural layer coefficient of the stabilized subgrade at each FWD test point were calculated.

4.4.3 FWD Data Analysis for I-75/I-96 Site in Wayne County, MI

FWD back calculation was performed using a concrete thickness of 13 inches for the inside lane and 11.5 inches for the shoulder as given in the construction documents. The ILLI-BACK back calculation program only calculates a concrete modulus and a composite modulus[3] of the subgrade reaction. Figures 4.12 and 4.13 show the back-calculated modulus of subgrade reaction for Areas 1-3 and 4-3 respectively for the inside and shoulder lanes.

[3] The composite modulus value includes all pavement layers below the concrete surface layer and the subgrade.

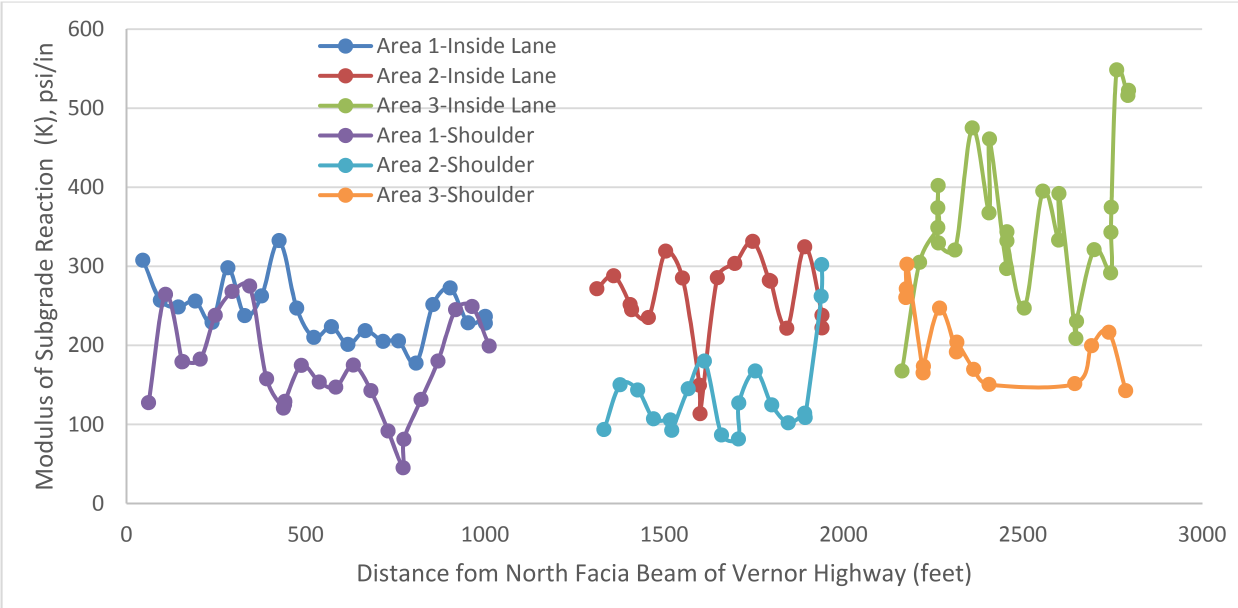


Figure 4.12: Modulus of Subgrade Reaction Values for Areas 1-3

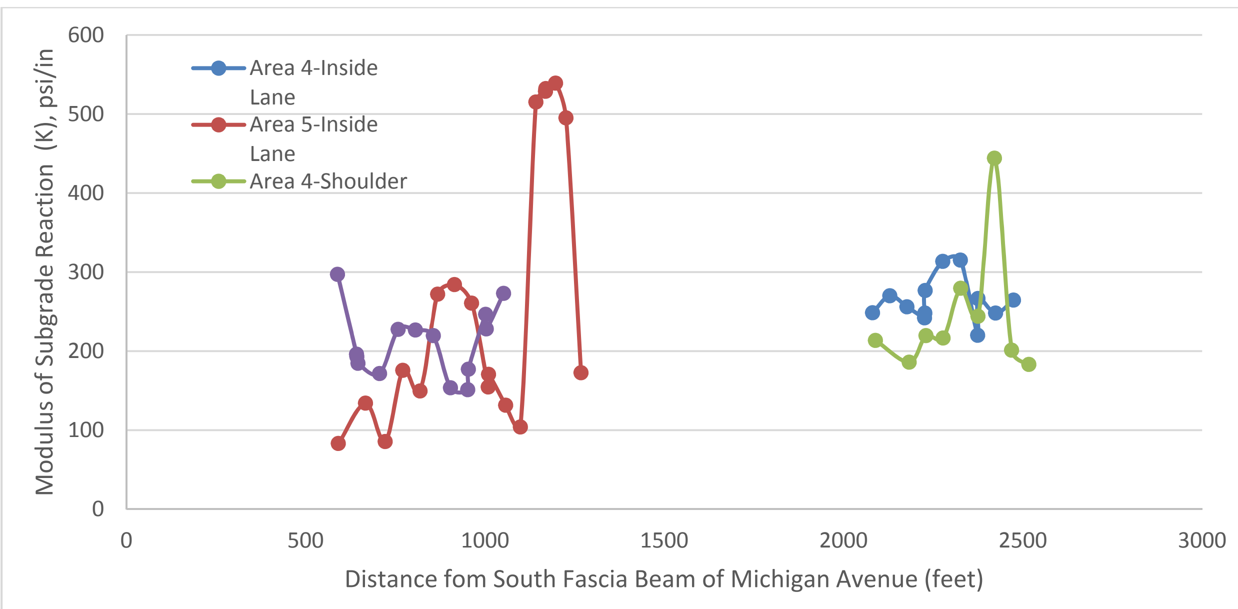


Figure 4.13: Modulus of Subgrade Reaction Values for Areas 4-3

The back calculated average modulus of subgrade reaction for different areas of the I-75/I-96 site is presented in Table 4.12 below.

The modulus of subgrade reaction for shoulder areas shows considerably lower values when compared to inside lane values. Excessive deflections along the shoulder lanes due to a lack of lateral support and variations in pavement thickness values may be the reason for these differences. Using inside lane values for further analyses is recommended.

Table 4.12: Back Calculated Average Modulus of Subgrade Reaction Values for I-75/I-96

Direction	Test Area	Area	Treatment	Thickness (inches)	Average k-Value (psi/inch)	Std. Dev. Of k (psi/inch)
NB	Pavement	1	CKD	12	242	35
		2	CKD	12	258	56
		3	Lime	18	356	92
	Shoulder	1	CKD	12	170	59
		2	CKD	12	138	58
		3	Lime	18	203	48
SB	Pavement	4	Lime/fly ash	12	264	27
		5	Lime	12	266	168
	Shoulder	4	Lime/fly ash	12	243	76
		5	Lime	12	210	41

4.4.4 FWD Data Analysis for M-84, Bay and Saginaw County, MI

The results of the pavement structural number calculation, based on the AASHTO method, for NB M-84 and SB M-84 are shown in Tables 4.13 and 4.14, respectively.

Table 4.13: AASHTO Pavement Structural Number Evaluation for NB M-84 FWD Data

Parameter	Average	Minimum	Maximum	Standard Deviation
Design M_R (0.35 of calculated M_R)	12,894.2	9,281.0	18,857.0	3,068.0
SN_{eff}	11.4	10.2	14.4	0.9
SN of stabilized layer	5.8	4.5	8.8	0.9
a_3 (structural layer coefficient) of 12-inch stabilized layer	0.48	0.38	0.73	0.08

Table 4.14: AASHTO Pavement Structural Number Evaluation for SB M-84 FWD Data

Parameter	Average	Minimum	Maximum	Standard Deviation
Design M_R (0.35 of calculated M_R)	16,558.6	9,659.0	21,600.0	2,958.6
SN_{eff}	11.4	10.4	12.4	0.5
SN of stabilized layer	5.8	4.8	6.7	0.5
a_3 (structural layer coefficient) of 12-inch stabilized layer	0.48	0.40	0.56	0.04

4.4.5 FWD Data Analysis for Waverly Road, Ingham County, MI

The results of the pavement structural number calculation, based on the AASHTO method, for Waverly Road are shown in Table 4.15.

Table 4.15: AASHTO Pavement Structural Number Evaluation for Waverly Road FWD Data

Parameter	Average	Minimum	Maximum	Standard Deviation
Design M_R (0.35 of calculated M_R)	11,929	9,474	15,349	1263
SN_{eff}	5.1	4.5	5.7	0.3
SN of stabilized layer	1.9	1.3	2.5	0.3
a_3 (structural layer coefficient) of 12-inch stabilized layer	0.16	0.11	0.21	0.02

4.4.6 FWD Data Analysis for SR310, Licking County, OH

Using the AASHTO method, the results of the pavement structural number calculation for NB and SB SR310 are shown in Table 4.16 and Table 4.17. Separate analyses were performed on the northbound and southbound lanes due to differences in calculated structural coefficient values.

Table 4.16: AASHTO Pavement Structural Number Evaluation for NB SR 310

Parameter	Average	Minimum	Maximum	Standard Deviation
Design M_R (0.35 of Calculated M_R)	7,506	5,188	12,857	1,722
SN_{eff}	7.3	5.9	8.8	0.64
SN of Stabilized layer	3.4	2.0	4.9	0.64
a_3 (structural layer coefficient) of 12-inch stabilized layer	0.24	0.14	0.35	0.05

Table 4.17: AASHTO Pavement Structural Number Evaluation for SB SR 310

Parameter	Average	Minimum	Maximum	Standard Deviation
Design M_R (0.35 of Calculated M_R)	6,144	3,976	10,939	1,708
SN_{eff}	5.8	4.7	7.5	0.66
SN of Stabilized layer	1.9	0.78	3.6	0.66
a_3 (structural layer coefficient) of 12" stabilized layer	0.14	0.06	0.26	0.05

4.4.7 AASHTO Layer Coefficients from DCP Data

AASHTO layer coefficients for stabilized layers were calculated from the methodology developed by B. K. Roy, (Roy, B.K., 2007). The methodology is based on the average DCP penetration rate (PR -inches/blow) and the thickness of the stabilized layer as shown below:

$$DCPN_i = BR_i \times T_i \qquad \text{(Equation 4.9)}$$

Where,

$DCPN_i$ = DCP number for the ith layer

BR_i = DCP blow rate for the ith layer (blows/inch)

T_i = Thickness of the ith layer

The structural number (SN) of the ith layer was calculated from the following equation:

$$SN_i = \frac{DCPN_i}{38.98} \qquad \text{(Equation 4.10)}$$

Once the SN of the layer was established, the structural layer coefficient of the stabilized layer was obtained by dividing the SN by the layer thickness.

Table 4.18: Structural Layer Coefficients for I-75/I-96

Test Area (Material)	Test Hole Number	Average PR (inches/blow)	BR (blows/inch)	DCPN	SN	a_i	Average a_i
1 (CKD) – clay subgrade	1	0.30	3.29	37.52	0.96	0.08	0.17
	2	1.66	0.60	5.89	0.15	0.02	
	3	0.06	16.04	194.09	4.98	0.41	
2 (CKD) sand subgrade	4	0.14	7.01	84.12	2.16	0.18	0.17
	5	0.12	8.08	83.99	2.15	0.21	
	6	0.21	4.66	62.95	1.62	0.12	
3 (Lime)	7	0.11	9.18	97.28	2.50	0.24	0.24
	8	0.11	9.01	82.03	2.10	0.23	
	9	0.10	10.00	97.93	2.51	0.26	
4 (Lime/fly ash)	10	0.10	10.01	163.17	4.19	0.26	0.26
	11	0.09	11.33	141.61	3.63	0.29	
	12	0.11	9.34	139.16	3.57	0.24	
5 (Lime)	13	0.69	1.45	18.44	0.47	0.04	0.10
	14	0.38	2.62	17.56	0.45	0.07	
	15	0.15	6.88	14.46	0.37	0.18	

Table 4.19: Structural Layer Coefficients for M-84

Test Area (Direction, Material)	Test Hole Number	Average PR (inches/blow)	BR (blows/inch)	DCPN	SN	a_i	Average a_i
1 (NB, LKD)	1	8.4	3.02	53.52	1.37	0.08	0.06
	2	10.5	2.42	32.90	0.84	0.06	
	3	13.0	1.95	17.00	0.44	0.05	
2 (SB, Lime)	4	4.8	5.29	100.01	2.57	0.14	0.10
	5	9.6	2.65	43.66	1.12	0.07	
	6	7.6	3.34	68.51	1.76	0.09	

Table 4.20: Structural Layer Coefficients for Waverly Road

Test Hole Number*	Average PR (inches/blow)	BR (blows/inch)	DCPN	SN	a_i	Average a_i
1	N/A	N/A	N/A	N/A	N/A	0.27
2	2.40	10.58	110.07	2.82	0.27	
3	N/A	N/A	N/A	N/A	N/A	
4	N/A	N/A	N/A	N/A	N/A	
*Only Test Hole Number 2 is considered a stabilized layer						

Table 4.21: Structural Layer Coefficients for SR310

Test Area (Direction, Material)	Test Hole Number	Average PR (inches/blow)	BR (blows/inch)	DCPN	SN	a_i	Average a_i
1 (NB, CKD)	1	5.88	4.32	66.58	1.71	0.11	0.15
	2	3.38	7.53	127.19	3.26	0.19	
2 SB, CKD)	3	3.00	8.47	82.13	2.11	0.22	0.19
	4	3.83	6.63	90.12	2.31	0.17	

4.4.8 Summary of Field Investigation Data

The field data collected through DCP testing and FWD testing were analyzed to obtain AASHTO structural layer coefficients and the modulus of stabilized layers for pavement design. Table 4.22 shows the summary of these analyses using different methods.

Table 4.22: Summary of Field Data Results

Test Site	Year Built (Age in Years)	Stabilized Material	Using DCP			Using FWD	
			CBR (%)	M_R* (psi)	a_i	a_i	k (psi/inch)
I-75 Area 1	2008 (7)	CKD – clay subgrade	46.7	29,900	0.17	N/A	242
I-75 Area 2	2008 (7)	CKD – sand subgrade	68.4	38,100	0.17	N/A	258
I-75 Area 3	2008 (7)	Lime	92.5	46,300	0.24	N/A	356
I-75 Area 4	2008 (7)	Lime/fly ash	94.1	46,800	0.26	N/A	264
I-75 Area 5	2008 (7)	Lime	55.8	33,500	0.10	N/A	266
M-84 NB	2010 (5)	LKD	23.2	19,100	0.06	0.48	N/A
M-84 SB	2010 (5)	Lime	39.6	26,900	0.10	0.48	N/A
Waverly Road	2010 (5)	CKD	87.5	44,600	0.27	0.16	N/A
SR 310	2008 (7)	CKD	49.8	31,100	0.17	0.14	N/A

* $M_R=2555\times(CBR)^{0.64}$
N/A = Not Applicable

The above results show, the stabilized layers were intact five to seven years after construction. These layers may have gone through several freeze/thaw cycles per year and still show higher moduli values than underlying subgrade soil.

It should be noted that the above calculated moduli and layer coefficient values represents the in situ site conditions at the time of testing. These values may change due to moisture levels, freeze/thaw conditions and other factors. Therefore, the above summary results should not be used without adjustments in design.

CHAPTER 5: INCORPORATING SUBGRADE STABILIZATION INTO PAVEMENT DESIGN

Short-term and long-term performances of stabilized subgrades with recycled materials were evaluated during this study using a series of laboratory experiments and by evaluating field performance of stabilized pavement sections. Both laboratory and field studies showed significantly higher modulus values for the stabilized subgrade layer when compared to the original subgrade material. Also, if proper mix designs were used, these studies show that the stabilized layer was durable. In one project few areas on the shoulder did not show the expected stiffness increase. .

In this chapter, the impact of soil stabilization were evaluated in terms of surface layer thickness and expected service life as determined by pavement analyses and design. Two software applications were used. WESLEA uses a linear elastic multi-layer analysis. AASHTOWare Pavement ME Design is a revision of the National Cooperative Highway Research Program mechanistic-empirical (ME) pavement design guide.

5.1 Pavement Sections for WESLEA Analysis and AASHTOWare Pavement ME Design

Two subgrade stabilized reference pavement sections were selected for comparative analysis with one being a rigid pavement and one being a flexible pavement: I-75 in Detroit, Wayne County, Michigan, and M-84 in Bay and Saginaw Counties, Michigan. The analysis was conducted assuming that the pavement structures were placed on the subgrades (Soil-1, Soil-2, and Soil-3) investigated in this study and the subgrade was stabilized using the suitable mix designs presented in Chapter 3. Furthermore, the thickness and properties of the base and subbase was maintained as constructed for the analysis. In brief, the project details are listed below.

I-75, extending from Vernor Street to Michigan Avenue, was a concrete reconstruction project completed in 2008. Lime stabilization was included in this project due to the extremely weak subgrade soil conditions. As a part of this project for a future side-by-side comparison to the lime-stabilized subgrade, two test sections with CKD-stabilized subgrade were constructed. The mainline pavement of I-75 included 13 inches of PCC over 16 inches of aggregate base.

The M-84 road section was constructed in 2010. As with the I-75 section, lime stabilization was included because of the poor subgrade conditions. During construction, lime stabilization of the northbound lanes of M-84 between Hotchkiss Road and Salzburg Road in Bay City was changed from lime to LKD. In use for five years, this section of M-84 was selected to compare the lime-stabilized subgrade against the LKD-stabilized subgrade. According to the construction documents, this pavement section consisted of 7.75 inches of Asphalt Pavement (5E3 at 165 lbs, 1.5 in; 4E3 at 275 lbs, 2.5 inches; 3E3 at 410 lbs, 3.75 inches) followed by six inches of aggregate base and 18 inches of sand subbase.

The overall objective of this comparative analysis is to investigate the effect of subgrade stabilization on pavement response.

5.2 Design Traffic

Annual average daily traffic on the selected section of I-75 was 41,800 vehicles per day during the year of construction (2008) with 13,742 commercial vehicles. Annual average daily traffic on the selected section of M-84 was 11, 515 vehicles per day during the year of construction (2010) with 265 commercial vehicles.

5.3 Pavement Layer Properties for WESLEA and AASHTOWare Analyses

The in situ subgrade in both of the selected sections of I-75 and M-84 was clay (CL/A-6). However, different subgrade soil types (CL/A-6, ML/A-4 and ML/A-7-6) were analyzed under the pavement structure to compare the effect of these different subgrades and the effect of stabilization upon them.

For the WESLEA analysis, I-75 pavement was considered as an equivalent flexible pavement section with 13 inches of Hot Mix Asphalt (HMA) paved on the as constructed 6 inches of base layer over eight inches of subbase layer. For the M-84 analysis, a constructed flexible pavement section with 7.75 inches of HMA and six inches of aggregate base over 18 inches of subbase was used. For the stabilized pavement designs, a 12- inch thick stabilized layer with modulus values shown in Tables 5.1 and 5.2 were used directly beneath the subbase layer. Default Poisson ratios (0.35 for HMA, 0.4 for granular materials, 0.35 for other material including stabilized layers and 0.45 for subgrade soils) were used.

For the Pavement ME analyses, the Poisson's ratio for the PCC layer was considered as 0.2. As suggested by the MDOT ME pavement design guideline, a Poisson's ratio of 0.35 was implemented for all other layers including HMA. Asphalt binder grade PG 64-22 was used for M-84 while 70-22 was used for I-75 for pavement ME design. The default modulus of elasticity recommended by the software application was used.

Table 5.1: Layer Properties of I-75 Flexible Pavement Sections

Soil Number (Subgrade Soil)	Treatment*	HMA		Aggregate Base		Sand subbase		Stabilized Subgrade		Subgrade	
		Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)
Soil-1 (CL, A-6)	Untreated	13	400,000	16	33,000	8	20,000	0	-	Semi-Infinite	5,000
	8% CKD	13	400,000	16	33,000	8	20,000	12	9,800	Semi-Infinite	5,000
	3% LKD/9% FA	13	400,000	16	33,000	8	20,000	12	24,000	Semi-Infinite	5,000
Soil-2 (ML, A-4)	Untreated	13	400,000	16	33,000	8	20,000	0	-	Semi-Infinite	5,000
	4% CKD	13	400,000	16	33,000	8	20,000	12	33,000	Semi-Infinite	5,000
	2% LKD/5% FA	13	400,000	16	33,000	8	20,000	12	29,000	Semi-Infinite	5,000
Soil-3 (CL, A-7-6)	Untreated	13	400,000	16	33,000	8	20,000	0	-	Semi-Infinite	5,000
	4% CKD	13	400,000	16	33,000	8	20,000	12	33,000	Semi-Infinite	5,000
	3% LKD/9% FA	13	400,000	16	33,000	8	20,000	12	31,000	Semi-Infinite	5,000

*FA – Fly ash

Table 5.2: Layer Properties of M-84 Flexible Pavement Sections

Soil Number (Subgrade Soil)	Treatment	HMA		Aggregate Base		Sand subbase		Stabilized Subgrade		Subgrade	
		Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)	Thickness (inches)	Layer Modulus (psi)
Soil-1 (CL, A-6)	Untreated	7.75	400,000	6	33,000	18	20,000	0	-	Semi-Infinite	5,000
	8% CKD	7.75	400,000	6	33,000	18	20,000	12	9,800	Semi-Infinite	5,000
	3% LKD/9% FA	7.75	400,000	6	33,000	18	20,000	12	24,000	Semi-Infinite	5,000
Soil-2 (ML, A-4)	Untreated	7.75	400,000	6	33,000	18	20,000	0	-	Semi-Infinite	5,000
	4% CKD	7.75	400,000	6	33,000	18	20,000	12	33,000	Semi-Infinite	5,000
	2% LKD/5% FA	7.75	400,000	6	33,000	18	20,000	12	29,000	Semi-Infinite	5,000
Soil-3 (CL, A-7-6)	Untreated	7.75	400,000	6	33,000	18	20,000	0	-	Semi-Infinite	5,000
	4% CKD	7.75	400,000	6	33,000	18	20,000	12	33,000	Semi-Infinite	5,000
	3% LKD/9%FA	7.75	40,0000	6	33,000	18	20,000	12	31,000	Semi-Infinite	5,000

5.4 Flexible Pavement Design Analysis using WESLEA

WESLEA is a linear elastic multi-layer program that enables the response analysis of a pavement structure including the effects of complex load systems. It was designed for layered elastic analysis of flexible pavement structures. All layers are assumed to be isotropic in all directions and infinite in the horizontal direction. The fifth layer is assumed to be semi-infinite in the vertical direction. Material inputs include layer thickness, modulus, Poisson's ratio, and an index indicating the degree of slip between the layers. Loads are characterized by pressure and radius. The WESLEA program calculates normal and shear stresses, normal strain, and displacement at specified locations. Figures 5.1, 5.2, 5.3and 5.4 show typical layer properties input, load assignment, locations of strain calculation and calculated stress-strain at one of the locations respectively.

WESLEA analysis provides an effective method of comparing different pavement sections in terms of their structural response under standard loads. The performance of these pavement sections were compared using following equations developed by the Asphalt Institute.

$$N_f = 0.0796 \times (\varepsilon_t)^{-3.291} \times (E_1)^{-0.854} \qquad \text{(Equation 5.1)}$$

$$N_d = 1.365 \times 10^{-9} \times (\varepsilon_c)^{-4.477} \qquad \text{(Equation 5.2)}$$

Where,

N_f = load cycles to failure due to fatigue cracking

N_d = load cycles to failure due to rutting

ε_t = maximum horizontal strain at the bottom of the asphalt layer

ε_c = maximum vertical strain on the surface of the subgrade

E_1 = elastic modulus of the asphalt mixture

Once the N_f and N_d were determined from the above equations, the critical pavement response was determined by comparing the number of load cycles to failure. If $N_f < N_d$, the pavement structure failed due to fatigue cracking. Alternately, if $N_d<N_f$, the pavement structure failed due to rutting.

It should be noted that these predicted load cycles should be used for comparison only. The predicted number of cycles to failure were calculated considering the horizontal strain at the bottom of the asphalt layer when considering fatigue cracking and vertical strain the surface of the subgrade when considering rutting.

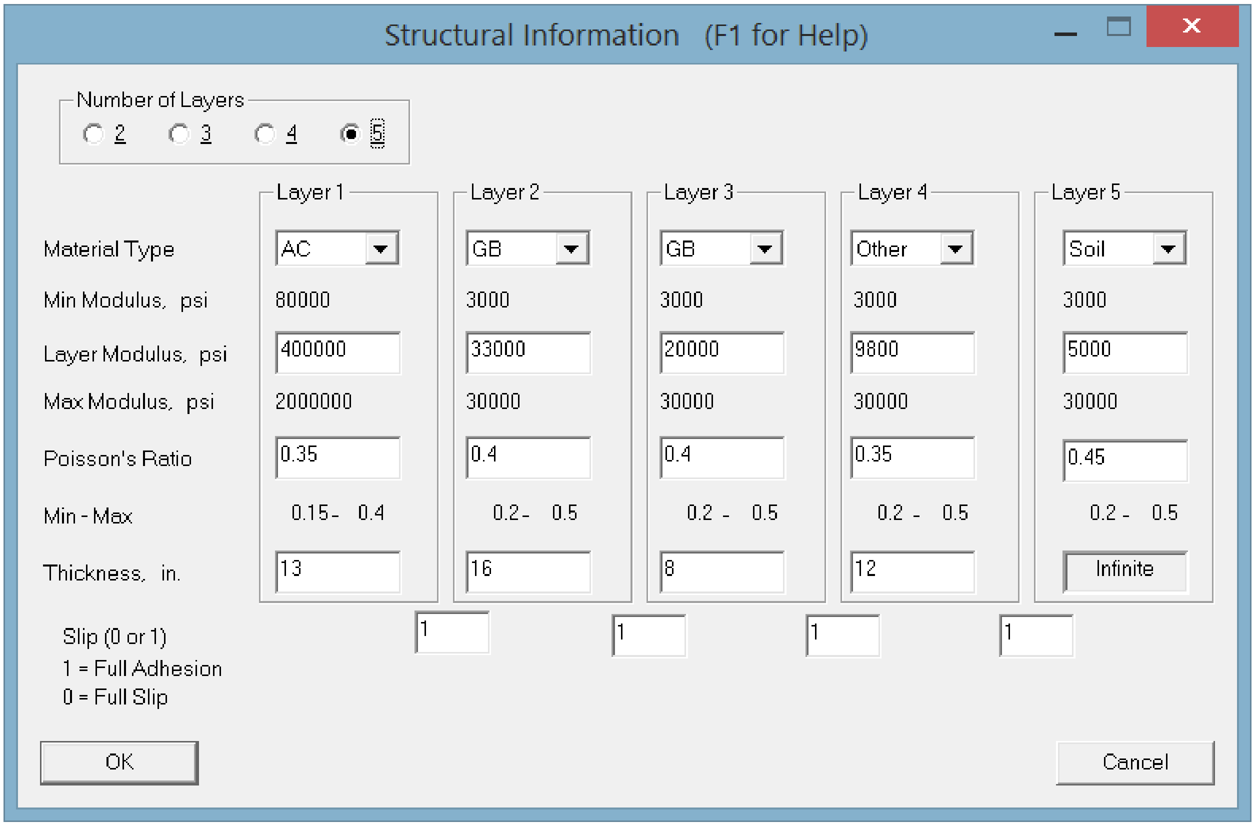


Figure 5.1: WESLEA Pavement Layer Properties Input Example

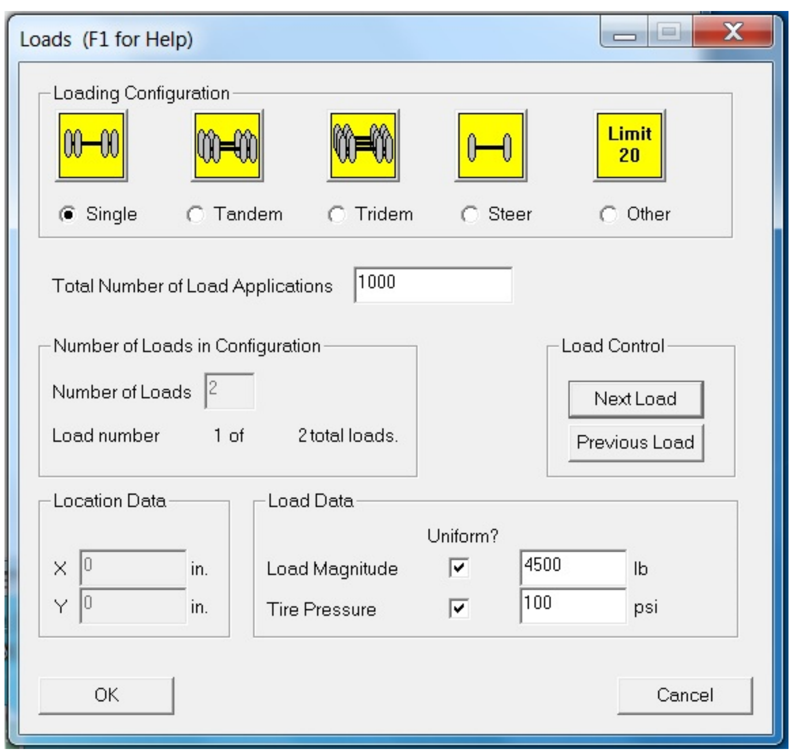


Figure 5.2: WESLEA Load Assignment Input Example

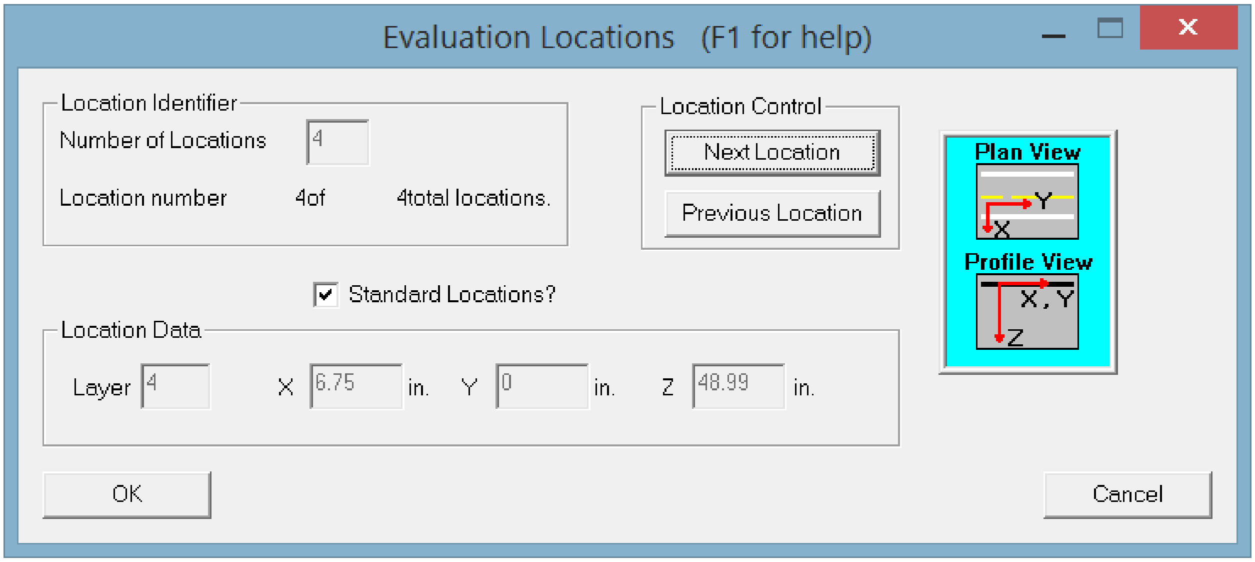


Figure 5.3: WESLEA Evaluation Locations Input Example

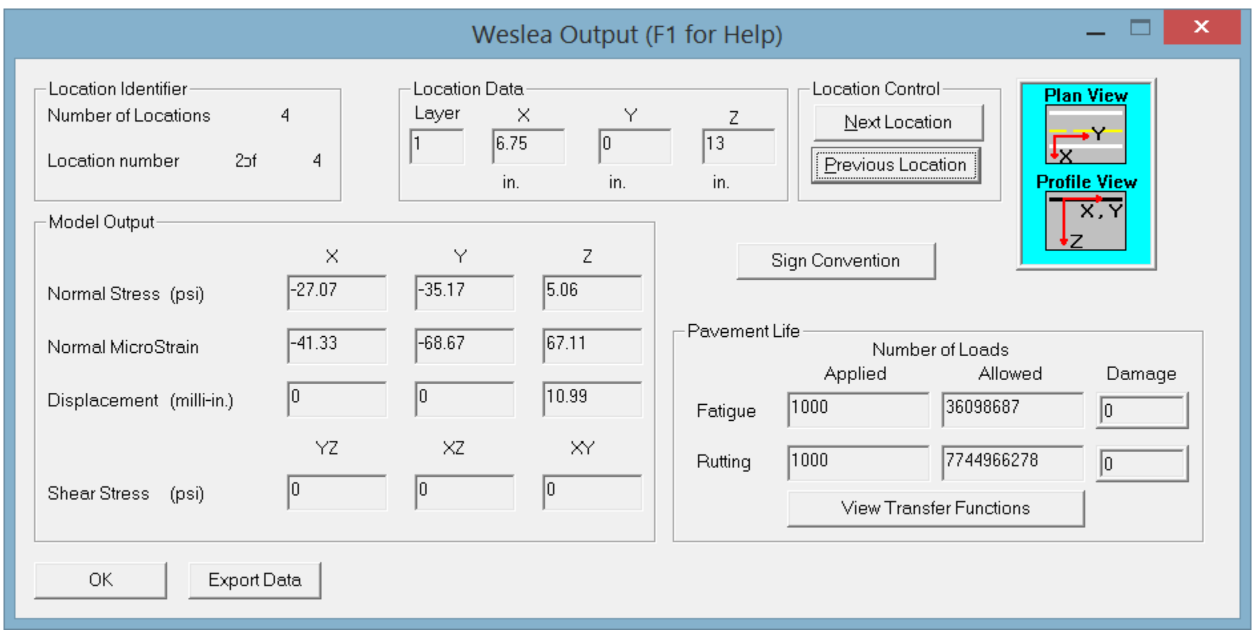


Figure 5.4: WESLEA Output Example

5.4.1 Interpretation of WESLEA Results and Determination of Structural Layer Coefficient of Stabilized Layers

The pavement responses under a standard load (dual-wheel with 18-kip axle load) were calculated for the pavement sections shown in Tables 5.1 and 5.2. Both I-75 and M-84 pavement analyses consisted of nine pavement sections each (3-untreated subgrades with different subgrades materials, 3-stabilized sections with CKD and 3-stabilized sections with LKD/FA). Tables 5.3 and 5.4 show the calculated pavement responses for each pavement section.

Both pavement sections show that failure was due to fatigue cracking of the asphalt layer. This is the generally expected failure criteria for thick pavement sections such as the I-75 and M-84 sections.

Table 5.3: Pavement Responses under Standard Load for I-75 Pavement Structure

Subgrade Soil	Treatment	HMA	Stabilized Subgrade		Failure Made	Pavement Responses	
		Thickness (inches)	Thickness (inches)	Layer Modulus (psi)		ε_t (10^{-6})	ε_c (10^{-6})
Soil-1 (CL, A-6)	Untreated	13	0	-	Fatigue Cracking (N_f)	74.31	140.16
	8% CKD	13	12	9,800	Fatigue Cracking (N_f)	72.56	77.93
	3% LKD/9% FA	13	12	24,000	Fatigue Cracking (N_f)	69.40	44.17
Soil-2 (ML, A-4)	Untreated	13	0	-	Fatigue Cracking (N_f)	74.31	140.16
	4% CKD	13	12	33,000	Fatigue Cracking (N_f)	68.40	35.66
	2% LKD/5% FA	13	12	29,000	Fatigue Cracking (N_f)	69.00	40.71
Soil-3 (ML, A-7-6)	Untreated	13	0	-	Fatigue Cracking (N_f)	74.31	140.16
	4% CKD	13	12	33,000	Fatigue Cracking (N_f)	68.40	35.66
	3% LKD/9% FA	13	12	31,000	Fatigue Cracking (N_f)	68.67	37.94

Table 5.4: Pavement Responses under Standard Load for M-84 Pavement Structure

Subgrade Soil	Treatment	HMA	Stabilized Subgrade		Failure Made	Pavement Responses	
		Thickness (inches)	Thickness (inches)	Layer Modulus (psi)		ε_t (10^{-6})	ε_c (10^{-6})
Soil-1 (CL, A-6)	Untreated	7.75	0	-	Fatigue Cracking (N_f)	146.52	146.52
	8% CKD	7.75	12	9,800	Fatigue Cracking (N_f)	144.76	144.76
	3% LKD/9% FA	7.75	12	24,000	Fatigue Cracking (N_f)	140.20	140.20
Soil-2 (ML, A-4)	Untreated	7.75	0	-	Fatigue Cracking (N_f)	146.52	146.52
	4%CKD	7.75	12	33,000	Fatigue Cracking (N_f)	138.89	138.89
	2% LKD/5% FA	7.75	12	29,000	Fatigue Cracking (N_f)	139.67	139.67
Soil-3 (ML, A-7-6)	Untreated	7.75	0	-	Fatigue Cracking (N_f)	146.52	146.52
	4% CKD	7.75	12	33,000	Fatigue Cracking (N_f)	138.89	138.89
	3% LKD/9% FA	7.75	12	31,000	Fatigue Cracking (N_f)	139.24	139.24

The tables above show the critical pavement response due to a standard load (tensile strain at the bottom of the asphalt layer) was always lower for stabilized pavement sections. This is due to the structural contribution from the stabilized layer to the overall pavement structure performance.

The structural contribution of the stabilized layer was quantified by employing an iterative process. In the WESLEA analysis, asphalt thickness values were changed to obtain the same critical response as the pavement section having untreated subgrade. For example, to determine the structural contribution of 8% CKD for Soil 1 (CL/A-6) in the I-75 pavement section, the asphalt section was reduced from 13 inches to 12.75 inches. This reduction increased the critical tensile strain at the bottom of the asphalt layer from 72.56×10^{-6} to 74.31×10^{-6}.

For the 1993 AASHTO pavement design analysis, the layer coefficient of stabilized subgrade (a_4) was calculated using the reduced HMA thickness. Assuming an AASHTO layer coefficient of 0.42 for asphalt layer, the layer coefficient for Soil 1, stabilized with 8% CKD for 12 inches, was calculated by equating Structural Numbers (SN) for a 0.25-inch thick asphalt layer to a 12-inch thick stabilized soil with 8% CKD as defined below.

$$SN_{reduced\ asphalt} = SN_{stabilized\ layer} \quad \text{(Equation 5.3)}$$

$$a_{asphalt} \times D_{reduced\ asphalt} = a_{stabilized\ layer} \times D_{stabilized\ layer} \quad \text{(Equation 5.4)}$$

$$0.42 \times 0.25 = a_{stabilized\ layer} \times 12 \quad \text{(Equation 5.5)}$$

$$a_{8\%\ CKD\ Stabilized\ Soil\ 1} = \frac{0.42 \times 0.25}{12} = 0.009 \quad \text{(Equation 5.6)}$$

Similarly, the following layer coefficients were determined for each soil type stabilized with a different percentage of stabilizing materials.

Table 5.5: Layer Coefficients for Stabilized Layer based on I-75 Pavement Section

Subgrade Soil	Treatment	Layer Coefficient
Soil 1 (CL, A-6)	8% CKD	0.009
	3% LKD/9% FA	0.020
Soil 2 (ML, A-4)	4% CKD	0.030
	2% LKD/5% FA	0.030
Soil 3 (ML, A-7-6)	4% CKD	0.030
	3% LKD/9% FA	0.030

Table 5.6: Layer Coefficients for Stabilized Layer based on M-84 Pavement Section

Subgrade Soil	Treatment	Layer Coefficient
Soil 1 (CL, A-6)	8% CKD	0.003
	3% LKD/9% FA	0.010
Soil 2 (ML, A-4)	4% CKD	0.010
	2% LKD/5% FA	0.010
Soil 3 (ML, A-7-6)	4% CKD	0.010
	3% LKD/9% FA	0.010

Using the WESLEA analysis, layer coefficients of the stabilized layer were used to determine the structural number (SN) of the stabilized layer as well as in designing pavement pursuant to AASHTO 1993 guidelines.

5.5 Modulus of Subgrade Reaction (k) for 1993 AASHTO Rigid Pavement Design

The modulus of subgrade reaction (*k*) is the design input parameter representing the in-situ soil in the AASHTO 1993 pavement design guideline for rigid pavements. Modulus of subgrade reaction is the total support provided by all layers below the concrete pavement structure including any base and subbase layers. When there is a base/subbase present under the concrete pavement, charts shown in Figures 5.5 and 5.6 are used to determine the value of *k*. The modulus of subgrade reaction is measured directly on subgrade surface using a plate test. However, the long-term effective design value of k is affected by factors such subgrade resilient modulus, subgrade moisture conditions, confinement provided by the constructed pavement structure, and loss of support, if any.

In order to incorporate the effect of the stabilized layer, hence increased stiffness, a composite value of k was used. The method used to calculate the composite k was based on American Concrete Pavement Association (ACPA) published design charts as shown in Table 5.7.

Table 5.7: Approximate Composite k Values for Various Subbase Types and Thicknesses (ACPA, 2012)

Unstabilized Subbase Composite k Values (psi/in)				
Subgrade k Value (psi/in)	**4 in.**	**6 in.**	**9 in.**	**12 in.**
50	65.2	75.2	85.2	110
100	130	140	160	190
150	175	185	215	255
200	220	230	270	320
Asphalt-Treated Subbase Composite k Values (psi/k)				
Subgrade k Value (psi/in)	**4 in.**	**6 in.**	**9 in.**	**12 in.**
50	85.2	112	155	200
100	152	194	259	325
150	217	271	353	437
200	280	345	441	541
Cement-Treated Subbase Composite k Value (psi/in)				
Subgrade k Value (psi/in)	**4 in.**	**6 in.**	**9 in.**	**12 in.**
50	103	148	222	304
100	185	257	372	496
150	263	357	506	664
200	348	454	634	823

ACPA also provides an online composite modulus of subgrade reaction (k_c) calculator for using above charts for multiple layers of subbase and subgrade materials. (http://apps.acpa.org/applibrary/KValue/#).

ACPA online composite modulus calculator was used for following combinations of base, stabilized subgrade and natural subgrade to determine k_c.

The calculated k_c values are shown in the following table for all other stabilization materials.

Table 5.8: Composite Modulus of Subgrade Reaction

Subgrade Soil	Treatment	Base/Subbase		Stabilized Subgrade		Natural Subgrade (psi)	K_c (psi/in)
		Thickness (in.)	Layer Modulus (psi)	Thickness (in.)	Layer Modulus (psi)		
Soil-1 (CL, A-6)	Un-stabilized	16	33,000	-	-	5,000	418
	8%CKD	16	33,000	12	9,800	5,000	426
	3%LKD+9%FA	16	33,000	12	24,000	5,000	482
Soil-2 (ML, A-4)	Un-stabilized	16	33,000	-	-	5,000	418
	4%CKD	16	33,000	12	33,000	5,000	524
	2%LKD+5%FA	16	33,000	12	29,000	5,000	506
Soil-3 (CL, A-7-6)	Un-stabilized	16	33,000	-	-	5,000	418
	4%CKD	16	33,000	12	33,000	5,000	524
	3%LKD+9%FA	16	33,000	12	31,000	5,000	515

Once the composite modulus of subgrade reaction is determined, the Figure 5.5 was used to correct the modulus of subgrade for the potential loss of support (LOS) due to pumping, etc. An MDOT established value of 0.5 is used for LOS (for open graded base materials) to determine the effective modulus of subgrade reaction as shown in Figure 5.5.

Figure 5.5 shows, the effective modulus of subgrade reaction is 213 psi/in for a 8% CKD stabilized subgrade material with 16 inches of aggregate base material.

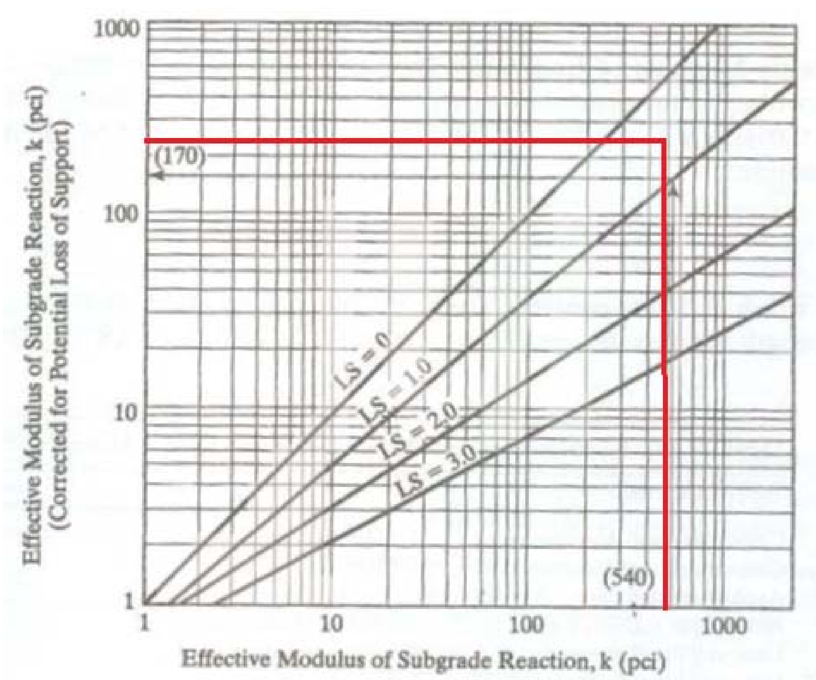


Figure 5.5: Effective Modulus of Subgrade Reaction Considering Potential Loss of Support (Ref: AASHTO 1993 Pavement Design Guideline)

Similarly, the following effective modulus of subgrade reaction values were determined for subgrades stabilized with different stabilization materials.

Table 5.9: Effective Modulus of Subgrade Reaction

Subgrade Soil	Treatment	Composite K (k_c) (psi/in)	Effective Modulus of Subgrade Reaction (K_{eff}) (psi/in)
Soil-1 (CL, A-6)	Un-stabilized	418	209
	8%CKD	426	213
	3%LKD+9%FA	482	241
Soil-2 (ML, A-4)	Un-stabilized	418	209
	4%CKD	524	262
	2%LKD+5%FA	506	253
Soil-3 (CL, A-7-6)	Un-stabilized	418	209
	4%CKD	524	262
	3%LKD+9%FA	515	257

5.6 AASHTOWare Pavement ME Design

The AASHTOWare Pavement ME Design procedure is the most recent state-of-the-art pavement design method introduced by National Cooperative Highway Research Program (NCHRP, 2004). It is significantly different procedure when compared to the popular AASHTO 1993 design procedure. The Pavement ME Design procedure uses structural models to estimate pavement responses to different traffic loading conditions considering climatic and other factors. These estimated pavement responses were used to estimate the accumulated damage and determine pavement distress levels at different intervals of the pavement lifecycle.

To determine the effects of stabilized layer properties on pavement performance, a pavement ME analysis was performed according to the MDOT ME Pavement Design guideline. The main goal of this analysis was to determine how the recommended modulus values for stabilized layers change the performance of pavement structures.

MDOT recommended values were used for layer properties, properties of materials, elastic modulus, resilient modulus, etc. (*Michigan DOT User Guide for Mechanistic-Empirical Pavement Design*, 2015). As introduced in Section 5.1, segments of I-75 and M-84 were used for this analysis. I-75 was designed as a rigid pavement with 13 inches of PCC followed by 16 inches of aggregate base. M-84 was designed as a flexible pavement with 7.75 inches of HMA followed by six inches of aggregate base and 18 inches of sand subbase. The in-situ subgrade soil was clay

(AASHTO classification – A-6). To compare the effect of stabilization, these pavements were analyzed using untreated subgrade as well as subgrades stabilized with CKD and a mix of LKD and fly ash. This process was repeated assuming other types of natural subgrade such as A-4 and A-7-6. A summary of the Pavement ME Design results are shown in Table 5.7 to Table 5.12.

The actual value of annual average daily traffic of the year of construction was used with a 3% growth rate and compound growth function. Vehicle class distribution, monthly adjustment, and axle per truck defaults were used. Annual Average Daily Truck Traffic (AADTT) growth of I-75 and M-84 with respect to time is shown in Figures 5.9 and 5.10. Cumulative truck volume is shown in Figure 5.11. MDOT recommended values were used for percent truck in design lane and operational speed. Software defaults were used for single axle, tandem axle, tridem axle, and quad axle distribution to simplify calculations. The software default for truck distribution per hour was also used (Figure 5.12).

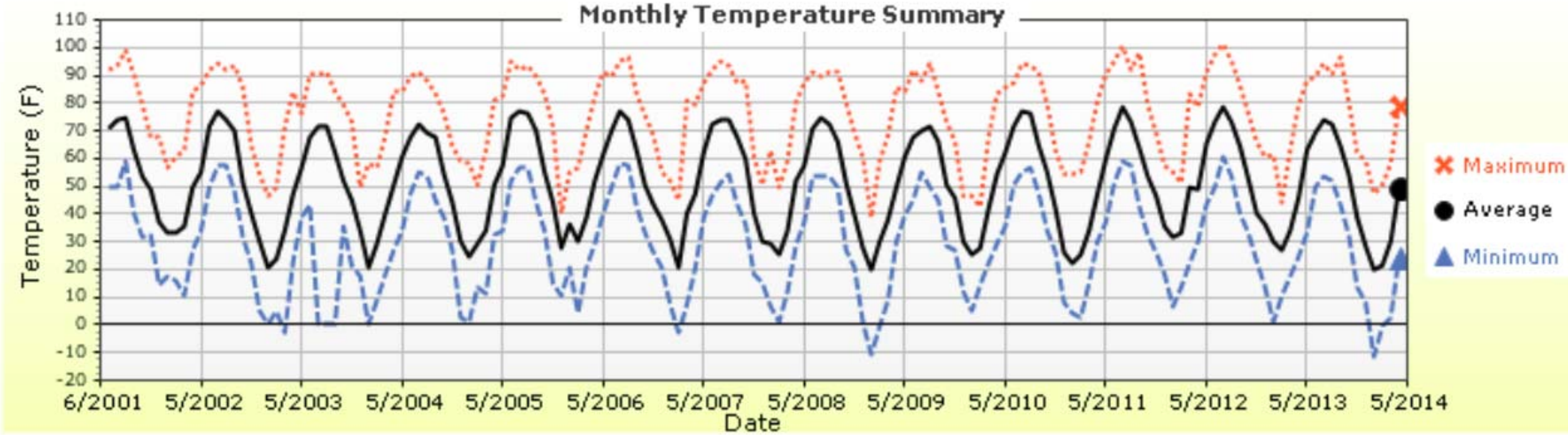


Figure 5.6: Monthly Temperature Summary (I-75)

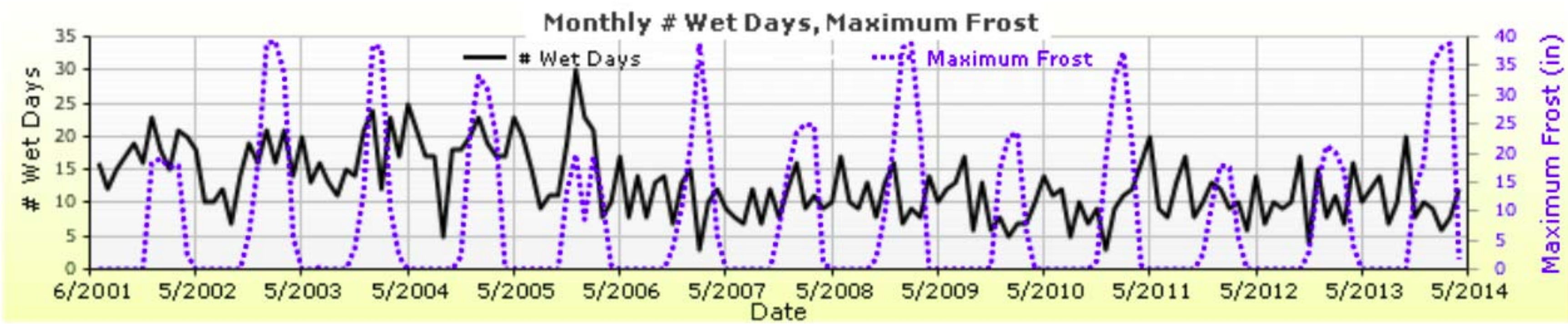


Figure 5.7: Monthly Wet Days and Maximum Frost (I-75)

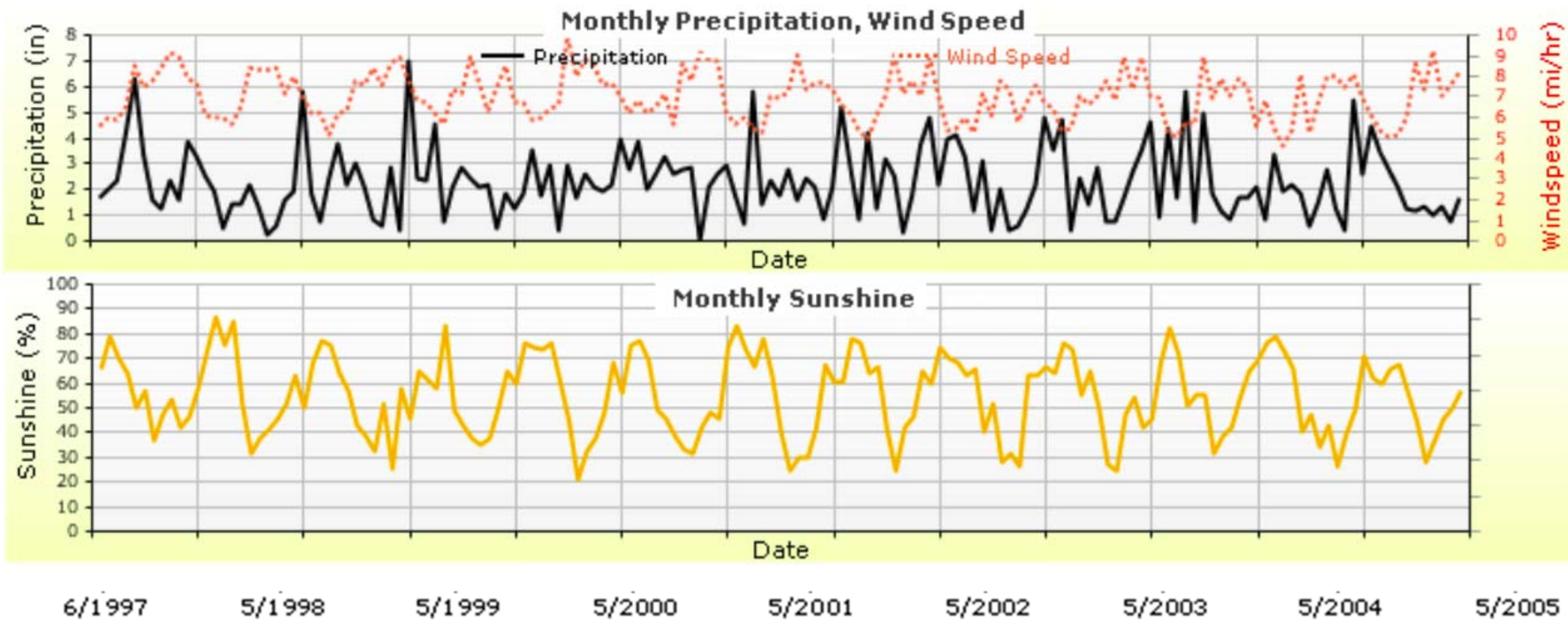


Figure 5.8: Monthly Precipitation, Wind Speed, and Sunshine (I-75)

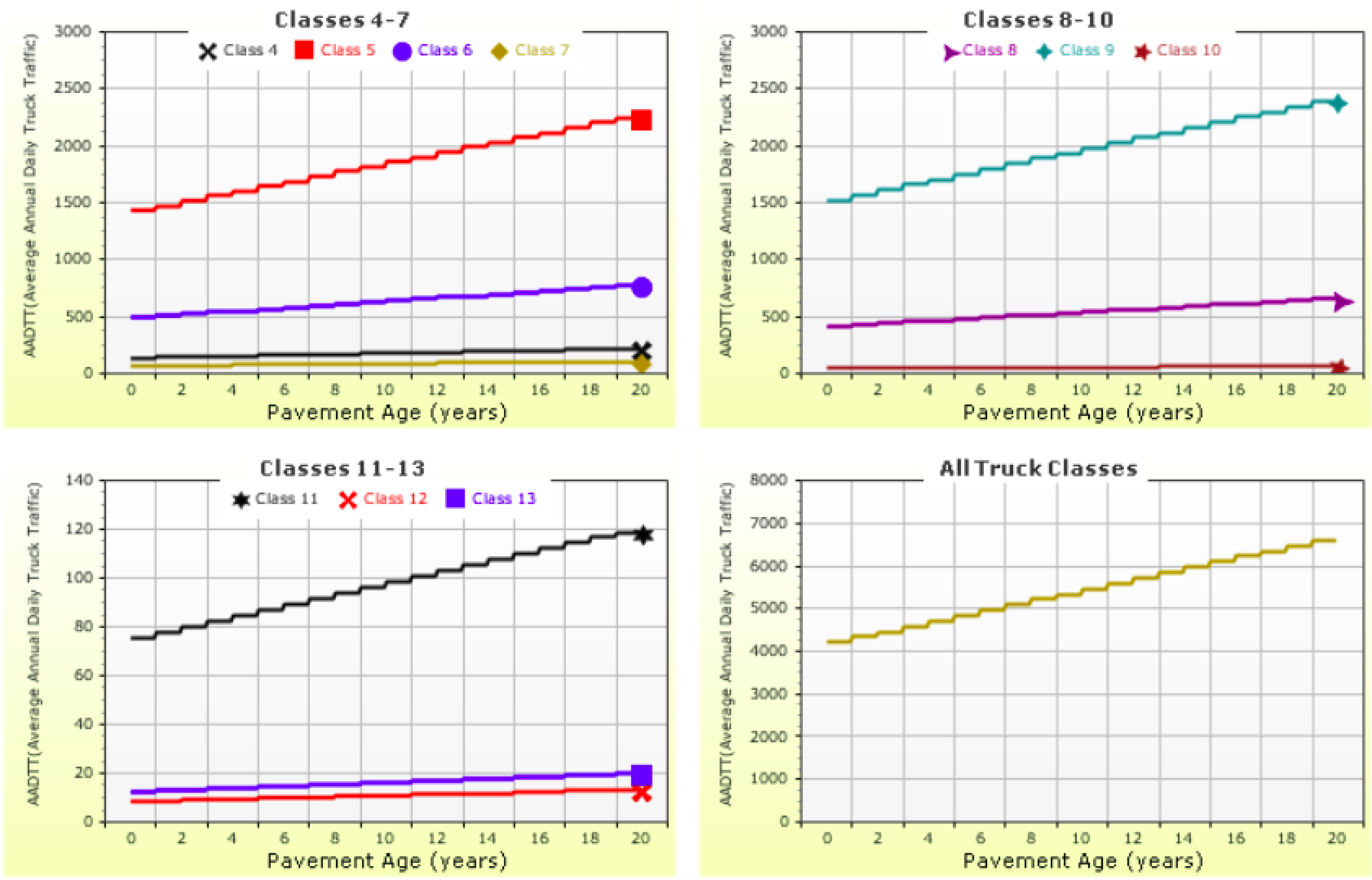


Figure 5.9: Growth of AADTT (I-75)

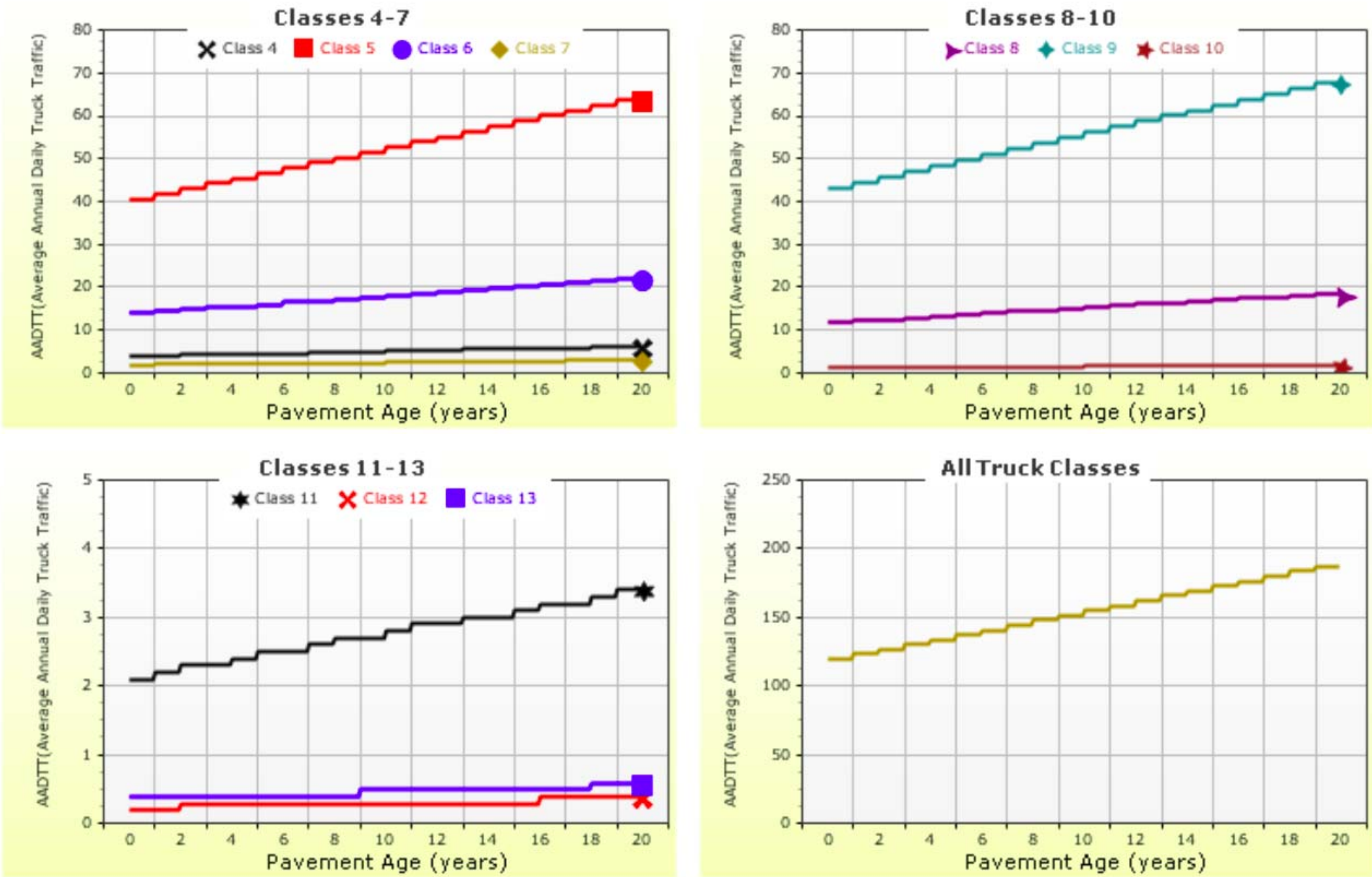


Figure 5.10: Growth of AADTT (M-84)

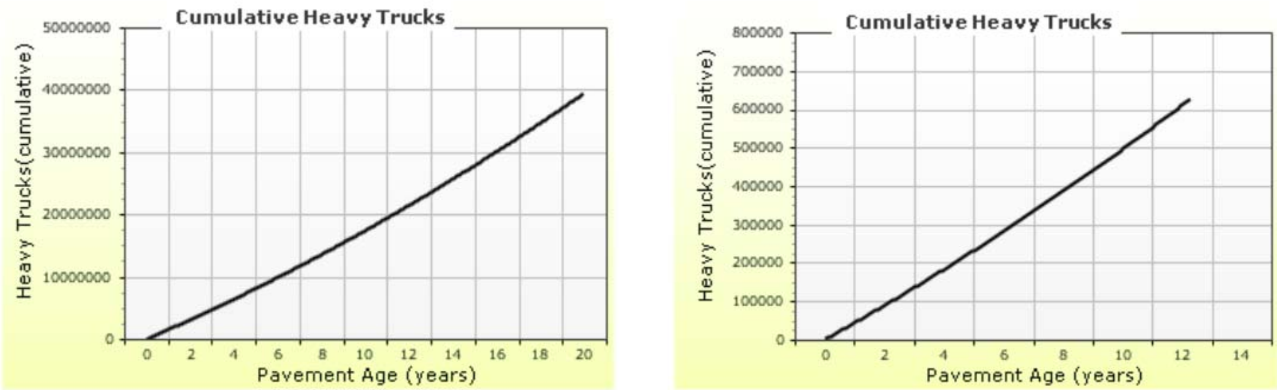


Figure 5.11: Cumulative Truck Volume [I-75 (left) and M-84 (right)]

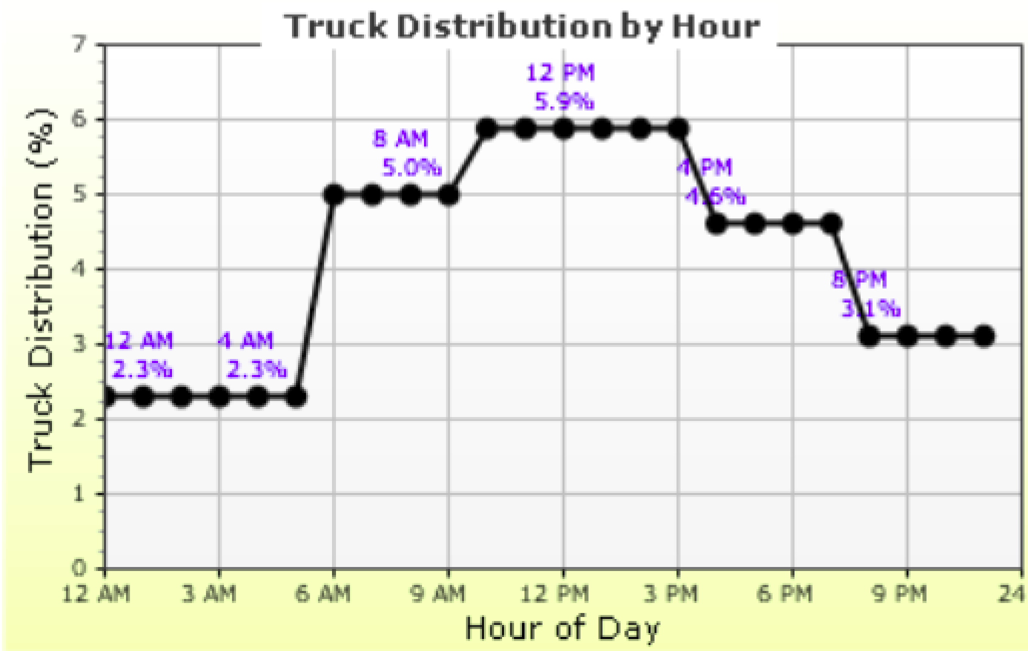


Figure 5.12: Truck Distribution per hour (I-75)

5.6.1 Rigid Pavement

A rigid pavement design was performed for a segment of I-75 in Detroit, Michigan. Resulting predicted International Roughness Index (IRI) graphs are shown in Figure 5.13. Although, the effect of stabilization on IRI was very minuscule, generally IRI reduces and reliability increases in the presence of a stabilized layer.

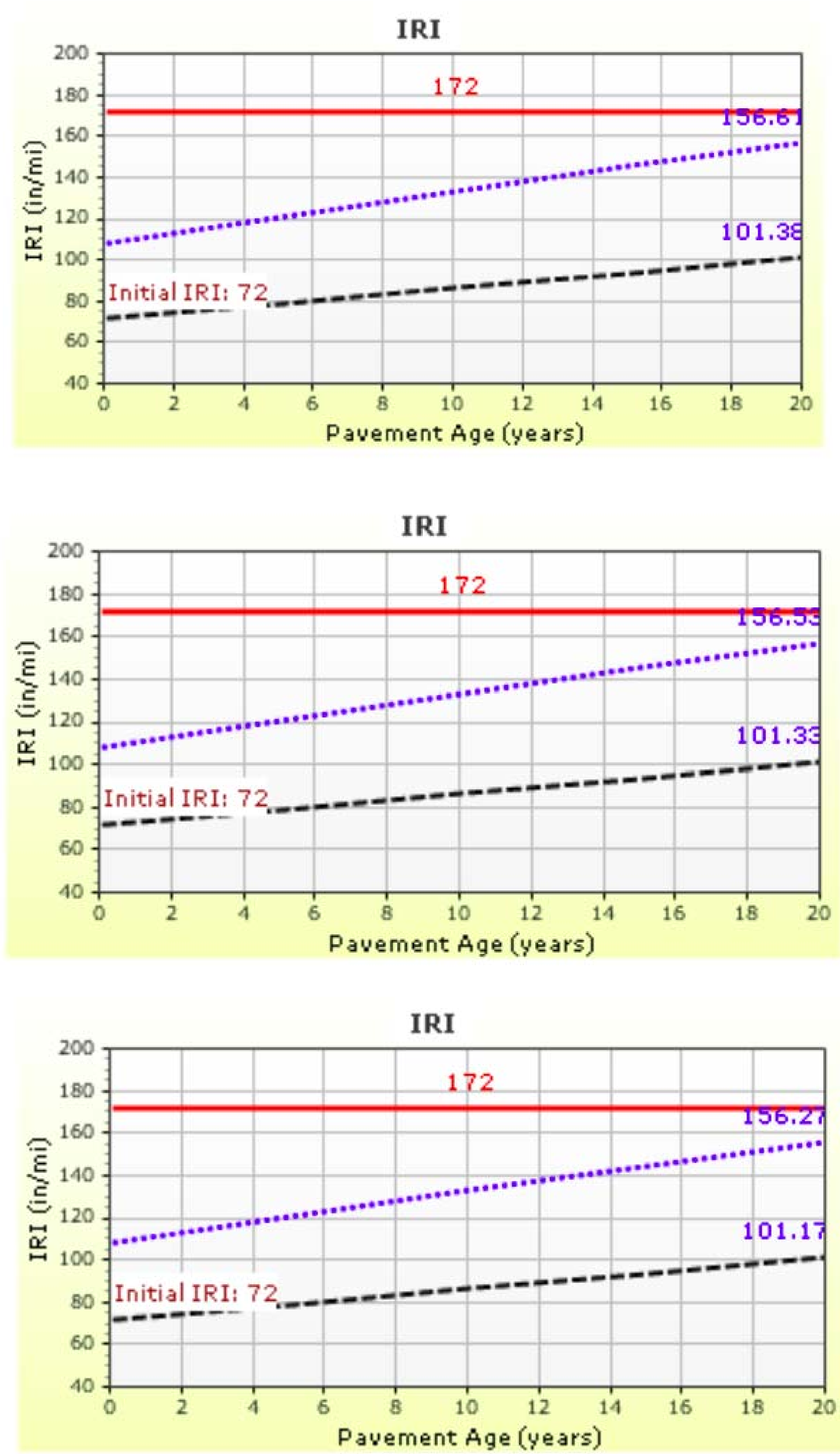


Figure 5.13: Predicted Terminal IRI for I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

The target mean joint faulting for a PCC slab was 0.125 inches with 95% reliability. Achieved reliabilities were 99.9% for untreated A-6 soil, 8% CKD-treated and 3% LKD/9% FA-treated soil, respectively. Figure 5.14 shows the predicted faulting graphs.

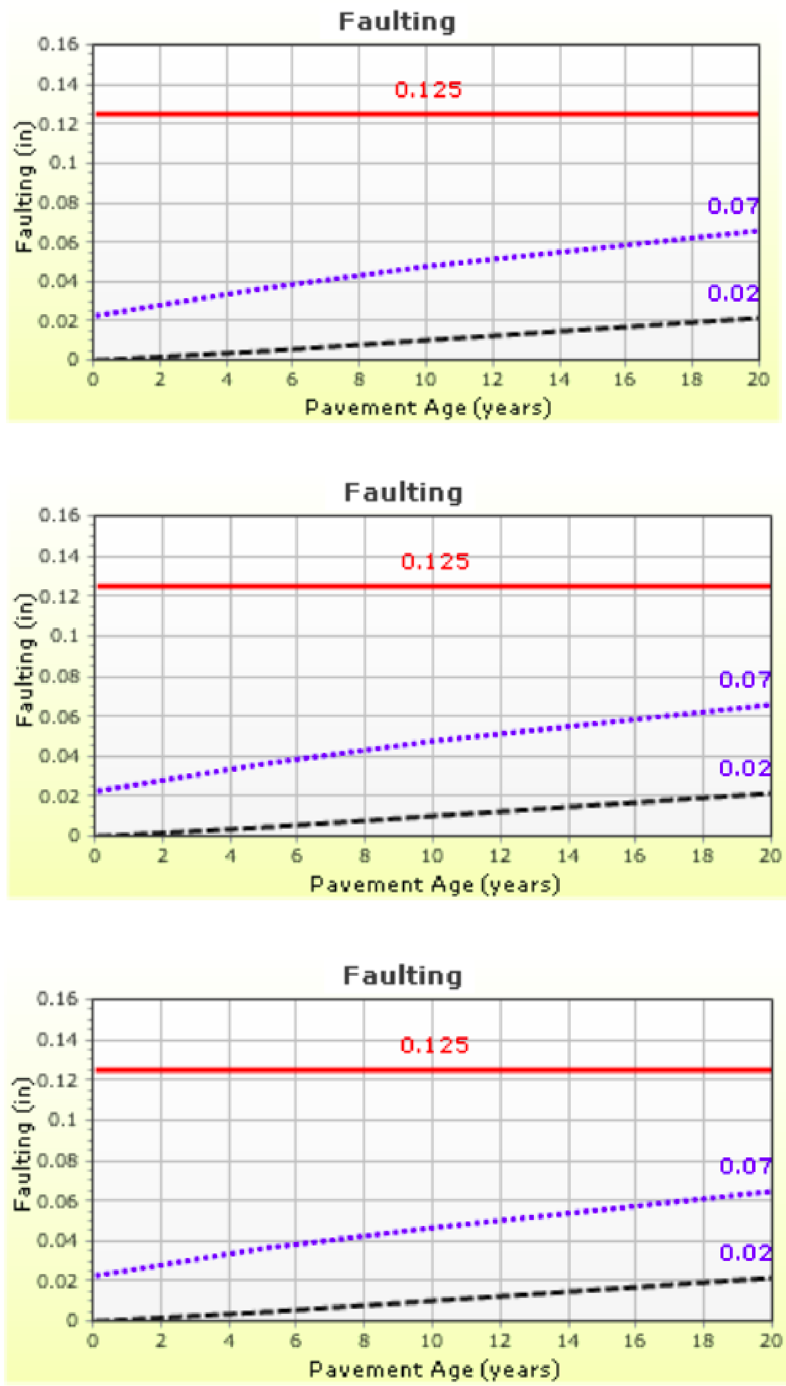


Figure 5.14: Predicted Faulting for I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

The target terminal Jointed Plain Concrete Pavement (JPCP) transverse cracking is 15% of the total slabs with 95% reliability. Predicted transverse cracking was 0.02% for untreated A-6 soil, 8% CKD-treated, and 3% LKD/9% FA-treated soil, respectively. Achieved reliabilities were 100% for all conditions. Changes in the percentage of transverse cracking were very insignificant. Figure 5.15 shows predicted transverse cracking graphs.

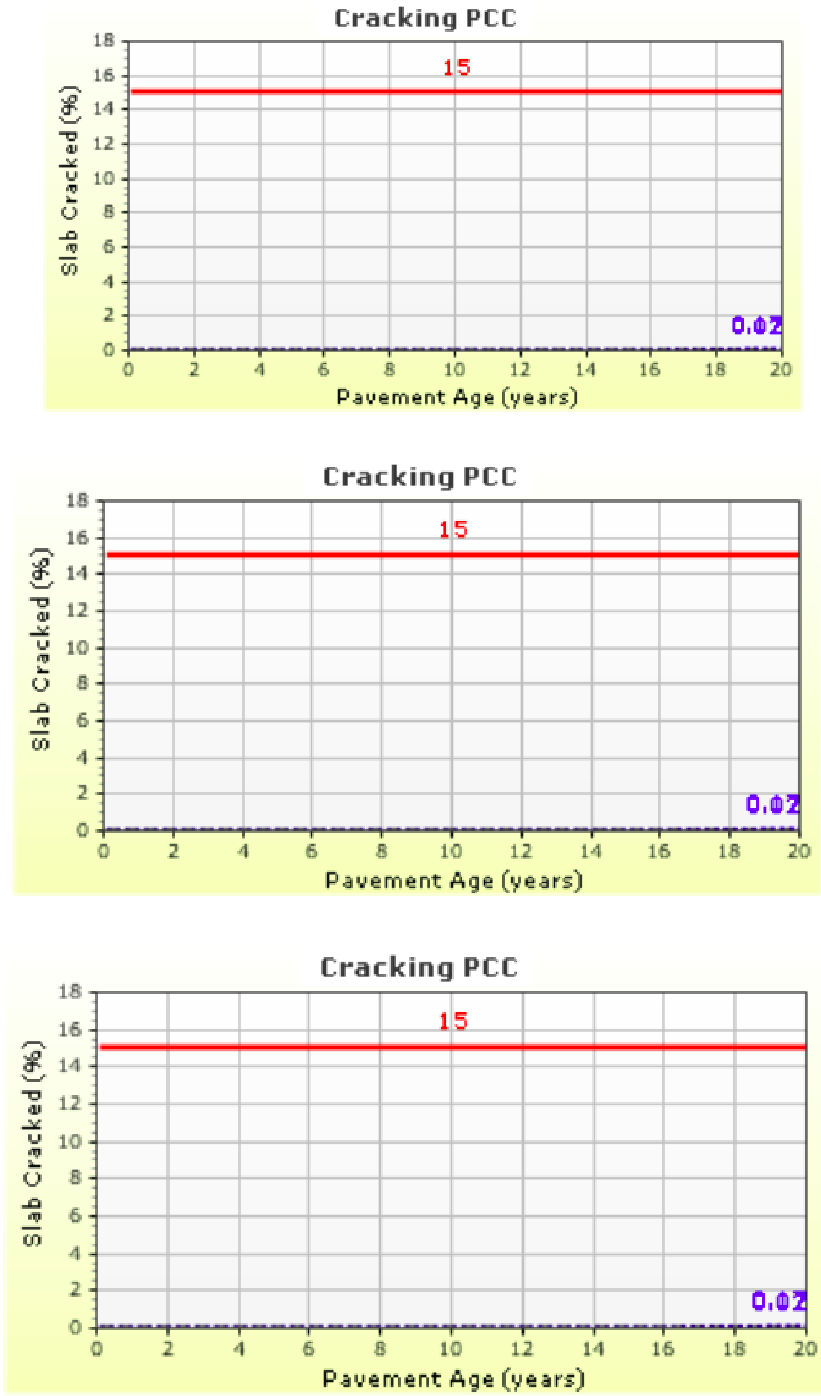


Figure 5.15: Predicted Cracking at I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

Cumulative damage of top-down cracking in PCC pavements after 20 years was 0.024 for untreated, 0.024 and 8% CKD-treated subgrade, and 0.025 for 3% LKD/9% FA-treated subgrade as shown in Figure 5.16.

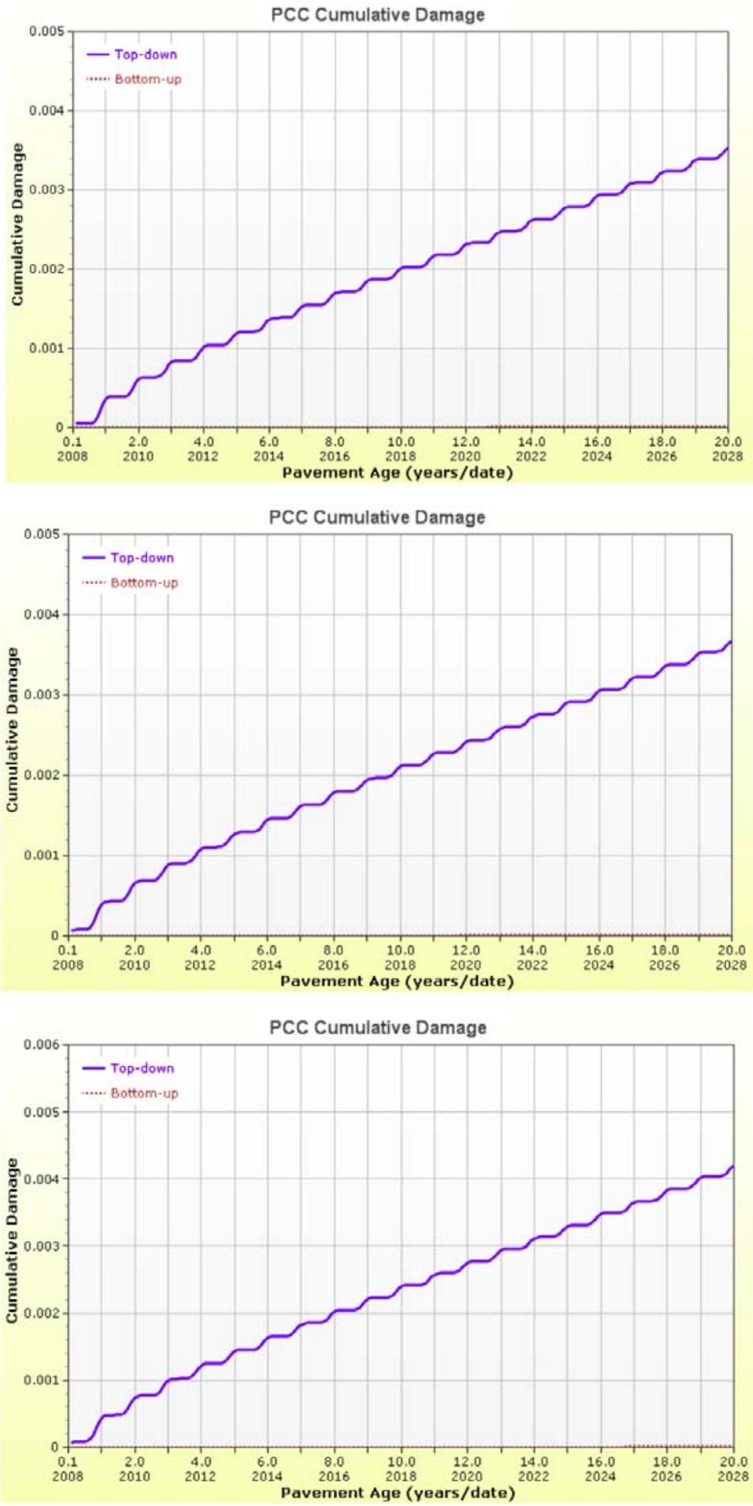


Figure 5.16: Predicted Cumulative damage of I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

There was no significant change in Load Transfer Efficiency (LTE). The LTE with time graphs are shown in Figure 5.17.

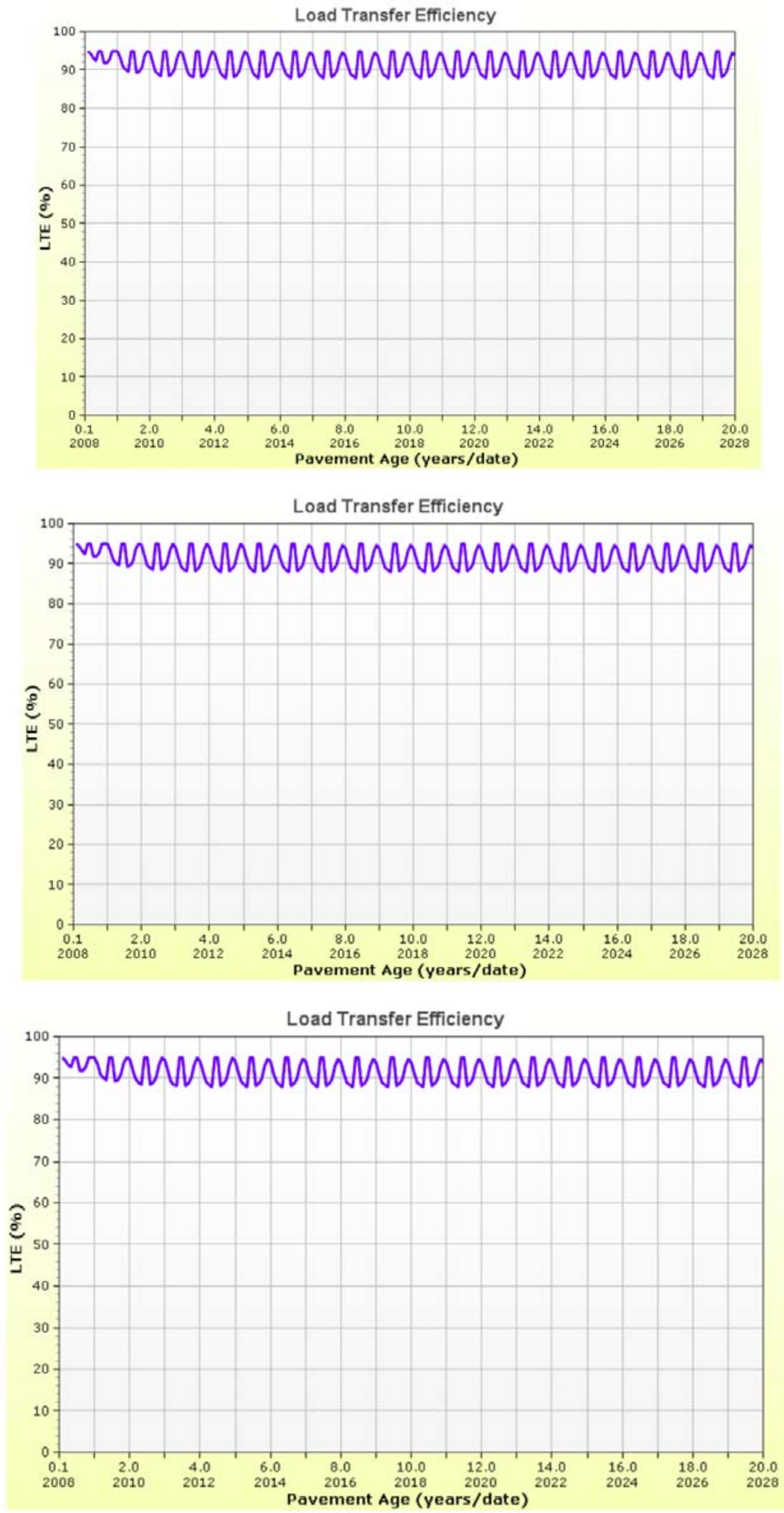


Figure 5.17: Predicted Load Transfer Efficiency of I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

5.6.2 Flexible Pavement

A flexible pavement design was performed for a section of M-84 in Bay and Saginaw Counties in Michigan. The predicted IRI graphs are shown in Figure 5.18. Generally IRI reduces and reliability increases in the presence of a stabilized layer.

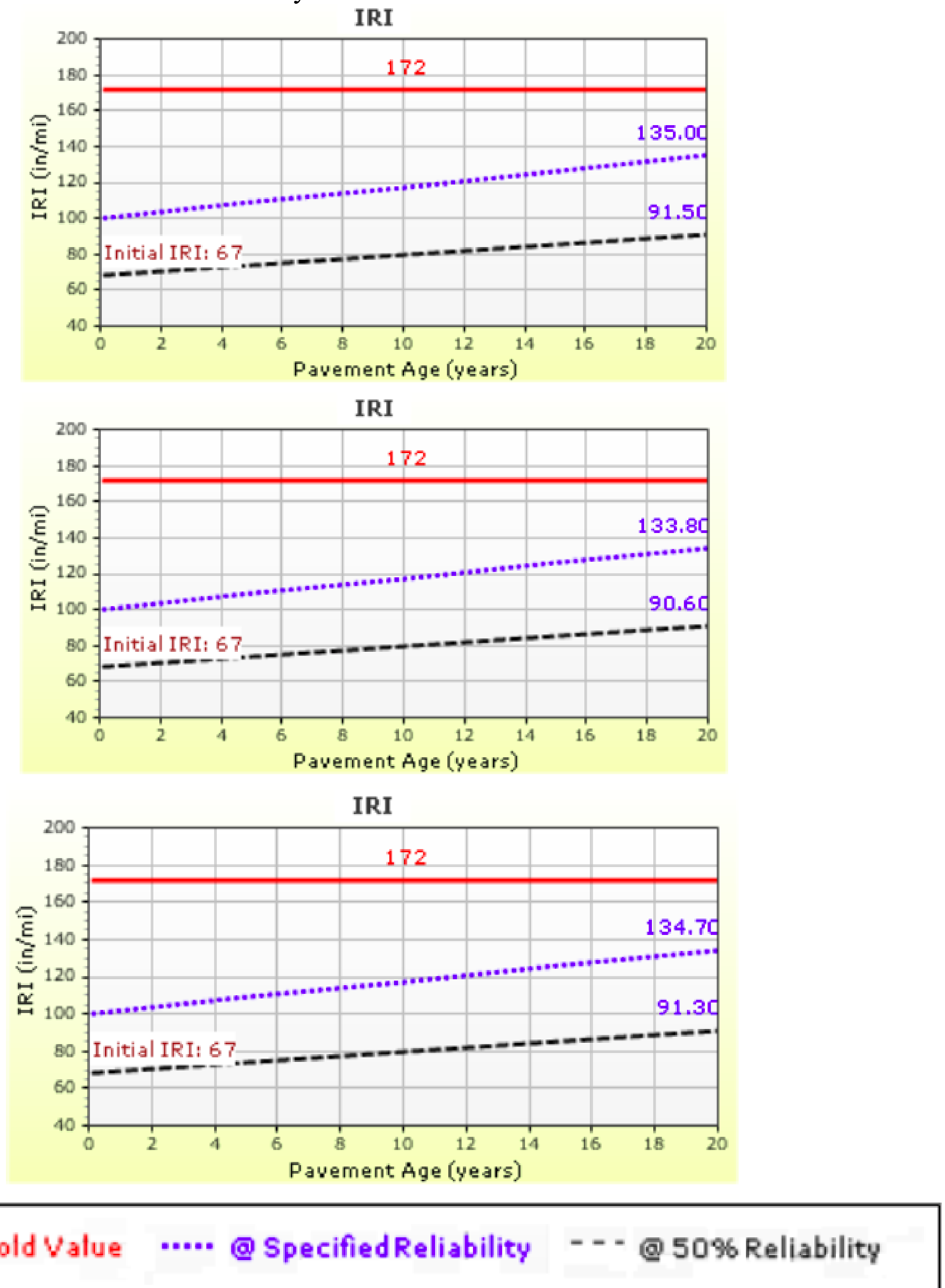


Figure 5.18: Predicted Terminal IRI for M-84 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

The maximum allowable permanent deformation (rutting) of total pavement at the end of design life is 0.50 inches. Predicted rutting was 0.3, 0.29 and 0.29 inches for untreated A-6 soil, 8% CKD-treated, and 3% LKD/9% FA-treated soil, respectively. Figure 5.19 shows rutting with respect to time on the M-84 section. Rutting in the HMA layer only was 0.27 inches for all cases, whereas the allowable maximum limit is 0.50 inches. The total predicted rutting at 50% reliability at different layers of pavement is shown in Figure 5.20.

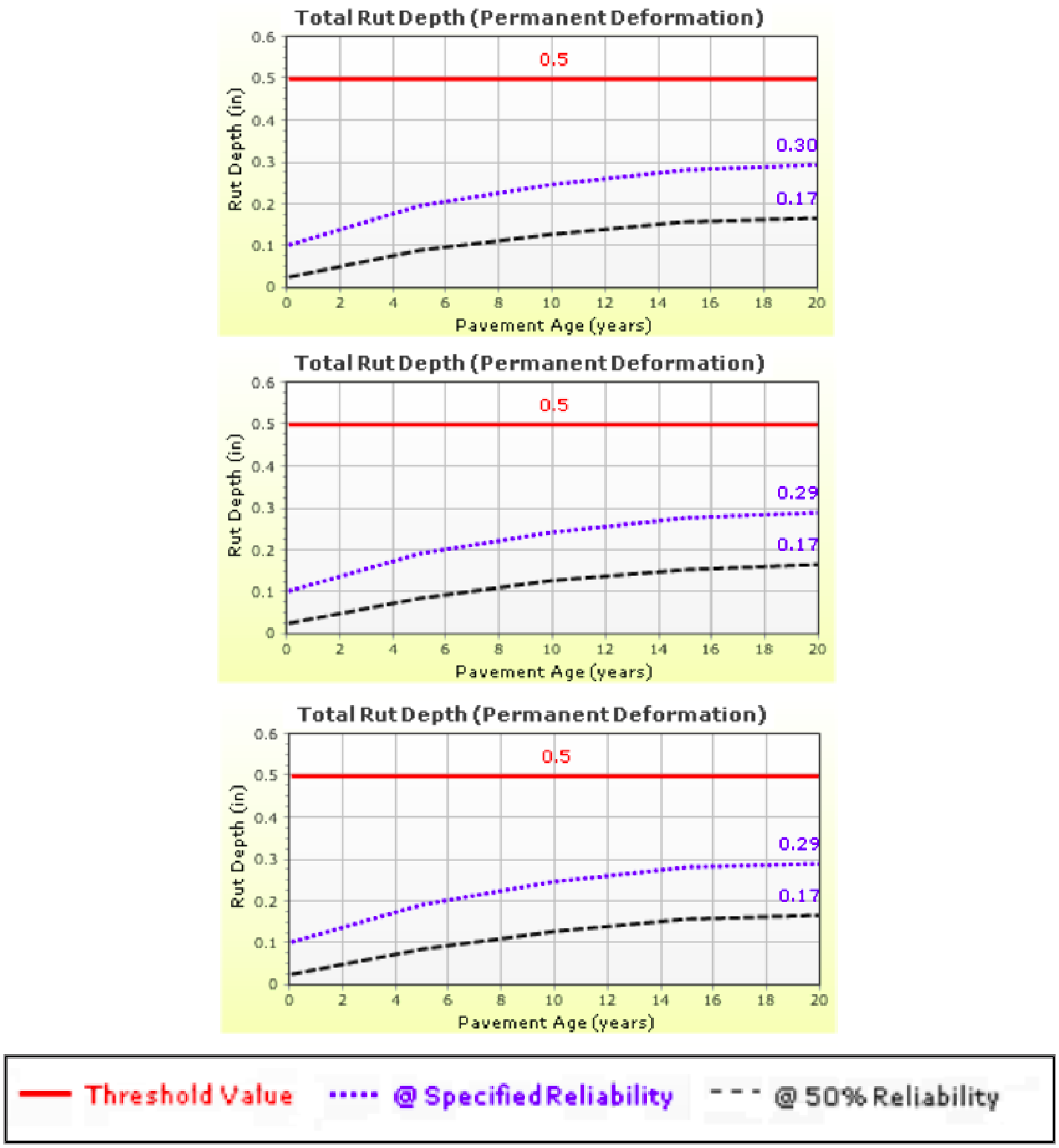


Figure 5.19: Predicted Total Rutting at M-84 (Top-Untreated A-6, Mid-8% CKD-Stabilized, Bottom-3% LKD/9% FA-Stabilized)

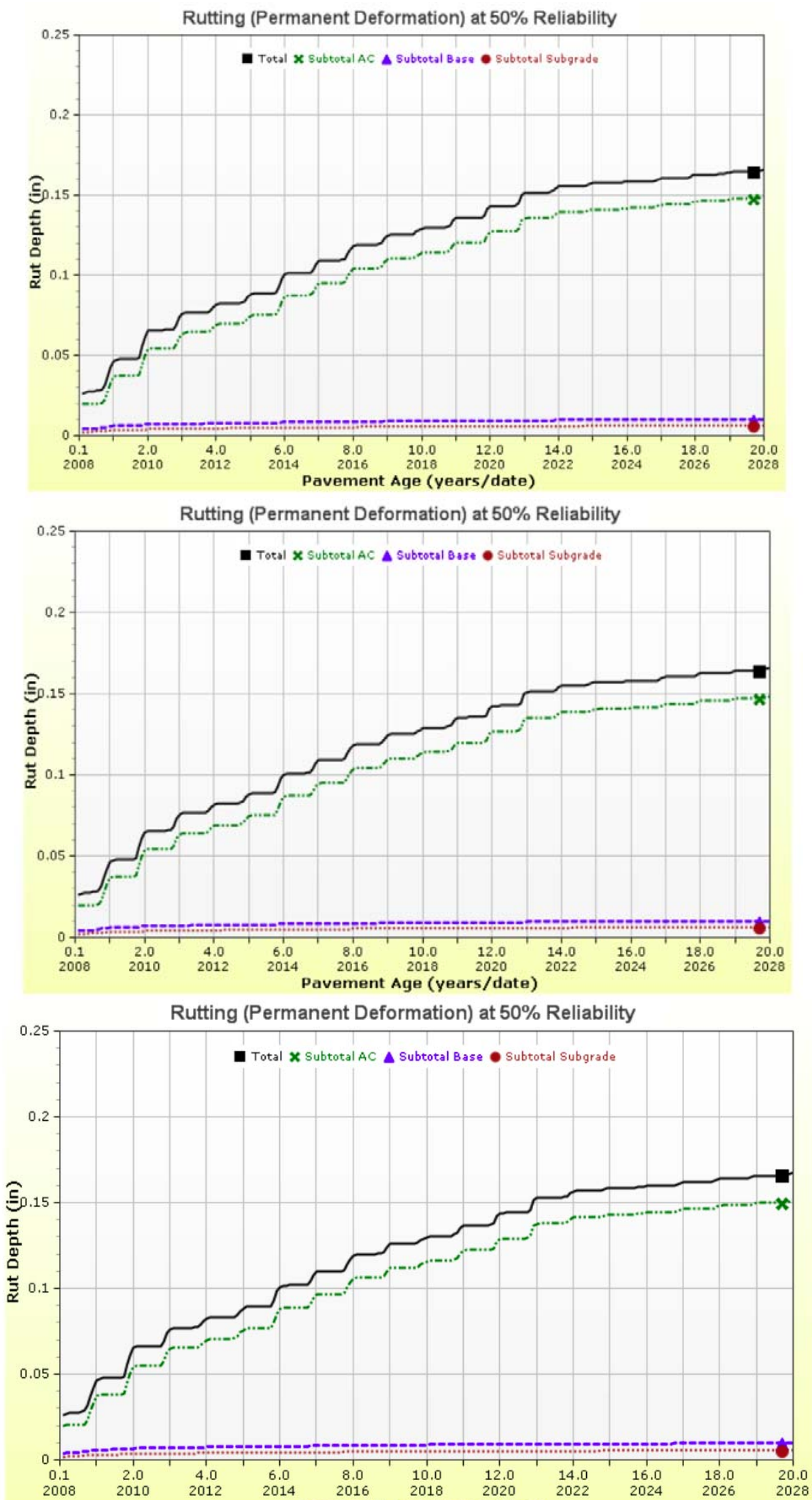


Figure 5.20: Predicted Total Rutting at Different Layers of M-84 at 50% reliability (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

The maximum allowable AC bottom-up cracking (Alligator) of total pavement at the end of design life is 20%. The predicted rutting was 13.91%, 13.95% and 13.75% for untreated A-6 soil, 8% CKD-treated, and 3% LKD/9% FA-treated soil, respectively. In A-6 soil, alligator cracking was low and hence, stabilization did not have much effect on alligator cracking. Figure 5.21 shows rutting with respect to time in M-84. Similarly, AC top-down fatigue cracking was 578.24, 561.40 and 634.83 feet/mile for untreated A-6 soil, 8% CKD-treated, and 3% LKD/9% FA-treated soil, respectively. This is low when compared to the maximum allowable limit of 2000 feet/mile.

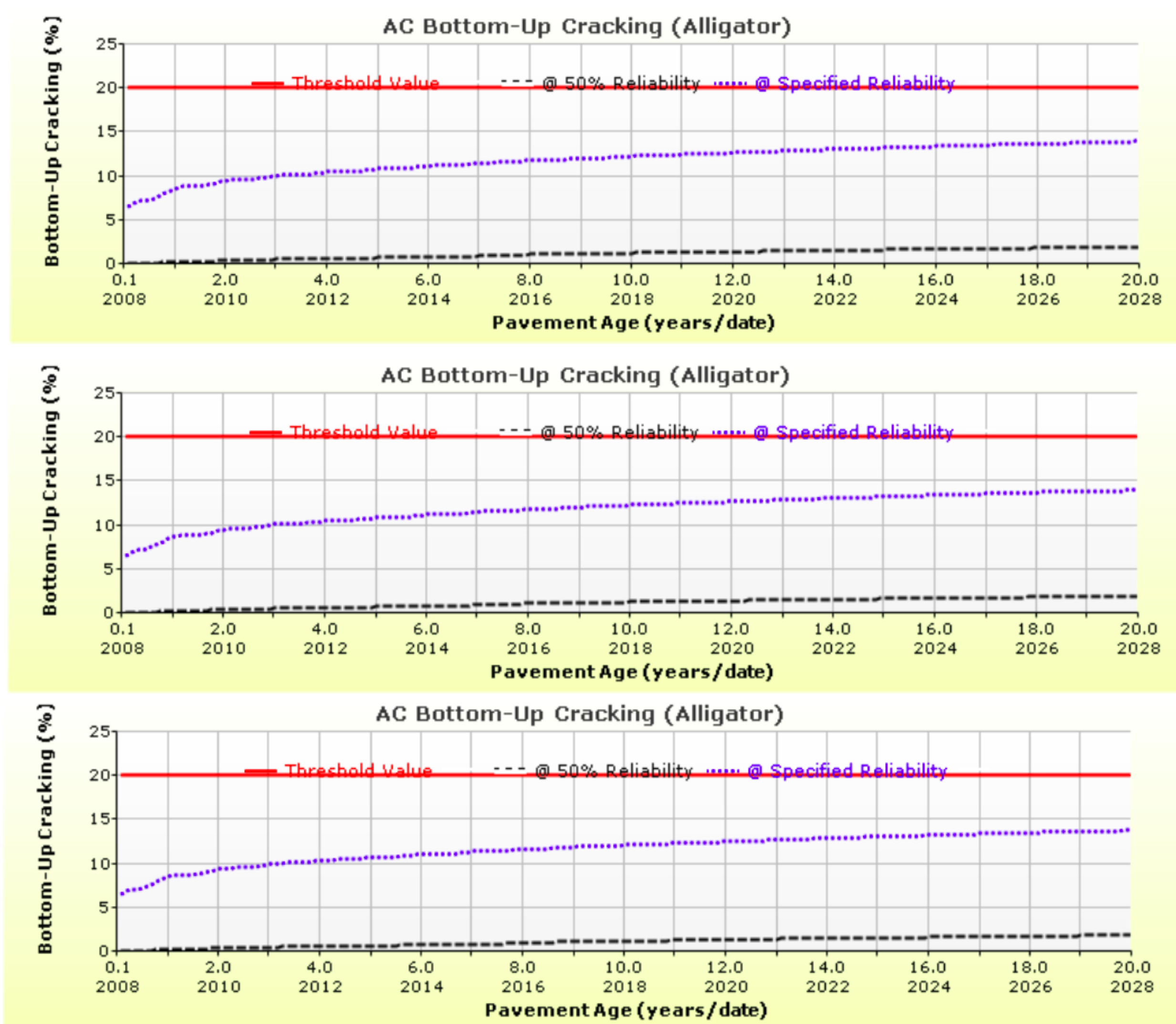


Figure 5.21: Predicted Alligator Cracking at M-84 (Top-Untreated A-6, Mid-8% CKD-Stabilized, Bottom-3% LKD/9% FA-Stabilized)

Table 5.10: Pavement ME Design Summary for I-75 Pavement (PCC)

Distress Type	Allowable Value	Predicted Value, Soil: A-6			Predicted Value, Soil: A-4			Predicted Value, Soil: A-7-6		
		Untreated	8% CKD	3% LKD / 9% FA	Untreated	4% CKD	2% LKD /5% FA	Untreated	4% CKD	3% LKD /9% FA
Terminal IRI (inches/mile)	172.00	156.61	156.53	156.27	146.59	146.24	146.26	156.36	156.03	156.03
Mean joint faulting (inches)	0.13	0.07	0.07	0.07	0.06	0.06	0.06	0.07	0.07	0.07
JPCP transverse cracking (percent slabs)	15.00	0.17	0.17	0.17	0.11	0.11	0.17	0.11	0.17	0.17

Table 5.11: Pavement ME Design Summary for M-84 Pavement (HMA)

Distress Type	Allowable Value	Predicted Value, Soil: A-6			Predicted Value, Soil: A-4			Predicted Value, Soil: A-7-6		
		Untreated	8% CKD	3% LKD /9% FA	Untreated	4% CKD	2% LKD /5% FA	Untreated	4% CKD	3% LKD /9% FA
Terminal IRI (inches/mile)	172.00	135.01	134.72	134.98	133.80	134.16	133.90	135.38	135.13	135.06
Permanent deformation - total pavement	0.50	0.29	0.29	0.29	0.29	0.30	0.30	0.29	0.29	0.29
AC bottom-up fatigue cracking (percent)	20.00	13.91	13.95	13.75	13.90	13.69	13.73	13.92	13.72	13.73
AC thermal cracking (feet/mile)	1000	346.13	345.57	346.15	346.16	345.57	346.16	346.13	345.57	346.13
AC top-down fatigue cracking (feet/mile)	2000	578.24	561.40	634.83	577.92	647.50	633.25	578.24	641.78	641.23
Permanent deformation - AC only (inches)	0.50	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27	0.27

Table 5.12: Pavement ME Design Summary of Reliability for I-75 Pavement (PCC)

Distress Type	Allowable Value	Target Reliability (%)	Achieved Reliability (A-6), %			Achieved Reliability (A-4), %			Achieved Reliability (A-7-6), %		
			Untreated	8% CKD	3% LKD 9%FA	Untreated	4% CKD	2%LKD + 5%FA	Untreated	4% CKD	3%LKD + 9%FA
Terminal IRI (inches/mile)	172	95	98.23	98.24	98.28	99.28	99.31	99.31	98.26	98.31	98.31
Mean joint faulting (inches)	0.13	95	99.99	99.99	100	100	100	100	99.99	100	100
JPCP transverse cracking (percent slabs)	15	95	100	100	100	100	100	100	100	100	100

Table 5.13: Pavement ME Design Summary of Reliability for M-84 Pavement (HMA)

Distress Type	Allowable Value	Target Reliability (%)	Achieved Reliability (A-6), %			Achieved Reliability (A-4), %			Achieved Reliability (A-7-6), %		
			Untreated	8% CKD	3%LKD + 9%FA	Untreated	4% CKD	2%LKD + 5%FA	Untreated	4% CKD	3%LKD + 9%FA
Terminal IRI (inches/mile)	172	95	99.88	99.89	99.88	99.90	99.90	99.90	99.88	99.88	99.88
Permanent deformation - total pavement	0.50	95	100	100	100	100	100	100	100	100	100
AC bottom-up fatigue cracking (percent)	20	95	99.34	99.33	99.39	99.34	99.41	99.39	99.34	99.40	99.40
AC thermal cracking (feet/mile)	1000	95	100	100	100	100	100	100	100	100	100
AC top-down fatigue cracking (feet/mile)	2000	95	100	100	100	100	100	100	100	100	100
Permanent deformation - AC only (inches)	0.50	95	100	100	100	100	100	100	100	100	100

As seen in the above results, the Pavement ME design analysis shows similar performance results for both the untreated and stabilized pavement sections. Only minor improvements were estimated from the Pavement ME Design approach. No significant pavement thickness changes were expected by using the recommended modulus values for stabilized layers during pavement design.

The pavement designers have the option to use the above recommended layer moduli values and structural layer coefficient values to gain a minor pavement thickness reduction for economical reasons. If they choose not to use the recommended values, still the subgrade stabilization will provide a stable, uniform pavement foundation for construction as well as for the in service pavement structure.

CHAPTER 6: SUMMARY AND RECOMMENDATIONS

This study quantified the characteristics of subgrades stabilized with recycled materials that would provide stabile platform during construction as well as contribute to improved long-term pavement performance. A comprehensive literature review, a series of laboratory experiments, and a field data collection program of existing stabilized pavement sections were performed to assess the characteristics of stabilized subgrades. Based on the research findings, the following conclusions, comments, and recommendations are made.

The literature review resulted in the summarization of state-of-the-art knowledge relative to subgrade stabilization with different stabilizing agents. Most of the studies reviewed, focused on the use of traditional stabilizers such as cement and lime. However, information related to test methods, performance evaluations of stabilized pavement sections, and other evaluation details obtained was utilized during the detailed development of the research tasks. Based on the literature review, discussions with MDOT staff, and the local availability (Michigan-sourced) of large quantities of recycled materials, the following were selected for evaluation: Cement Kiln Dust (CKD), Lime Kiln Dust (LKD), Fly Ash (FA), Concrete Fines (CF) and mixes of the aforementioned materials.

The soil types selected for the study represented weaker subgrade soils found in the State of Michigan. More specifically, if these soils are encountered during construction, soil removal and replacement are required to complete construction activities. After identification of the problematic soils, MDOT obtained these soils from construction sites during removal/replacement operations. The soil types used for this study included: CL (A-6)-type soil from Detroit, Michigan; ML (A-4)-type soil from Livonia, Michigan; and CL (A-7-6) type soil from Chippewa County in the Upper Peninsula of Michigan.

The laboratory investigation program included identification of basic soil properties: grain size, Liquid Limit and Plastic Limit, Maximum Dry Density and Optimum Water Content[4], and Unconfined Compressive Strength (UCS). After determining these baseline soil properties, a series of mix designs were performed to determine the minimum stabilizer percentage required for long-term stabilization or short-term subgrade modification. Long-term stabilization was defined as an increase of 50 psi over the untreated soil UCS after seven days of curing and 24 hours of capillary soaking. Whereas, the short-term modification was defined as an increase of 50 psi over the untreated soil UCS after three days of curing without capillary soaking. For all soil types, CKD and LKD mixed with FA were identified as long-term stabilization materials when used at specific percentages. FA and LKD only worked for some soil types as short-term modifier to construct upper pavement layers. Concrete Fines (CF) were ineffective as either a stabilizer or a short-term modifier for all three soil types. The mix designs listed in Section 6.1 details the problematic soil type and the percentage of CKD or LKD/FA needed for long-term stabilization.

[4] Obtained via Standard Proctor Tests

6.1 Stabilizer Recommendations for Long-Term Subgrade Stabilization

Based on the laboratory testing, the following stabilizer percentages are recommended for the different soil types evaluated in this study. However, it should be noted that these percentages should be used as guidelines for estimation purposes only. Proper mix designs should be conducted prior to the selection of any stabilizer relative to the project soil type.

Table 6.1: Recommended Stabilizer Percentages (by weight) for Long-Term Stabilization

Soil Type	CKD (%)	LKD (%)/FA (%)
CL, A-6	8	3/9
ML, A-4	4	2/5
CL, A-7-6	4	3/9

Although the laboratory results showed that 25% FA by dry weight will work as a long-term stabilizer for soil type ML (A-4), it was not recommended for use due to the very high application rate required for stabilization.

6.2 Stabilizer Recommendations for Short-Term Subgrade Modification

Based on the laboratory testing, the following stabilizer percentages are recommended for short-term modification of the three different soil types. These percentages can be used as recommended without performing any project-specific mix designs. The main goal of subgrade modification is to create a working platform to construct upper pavement layers. No long-term stabilization is expected, therefore, the following recommendations will provide sufficient subgrade strength to construct the upper pavement layers.

Table 6.2: Recommended Stabilizer Percentages (by weight) for Short-Term Modification

Soil Type	FA (%)	LKD (%)
CL, A-6	15	6
ML, A-4	15	-
CL, A-7-6	15	-

- Not recommended for short-term modification

Other stabilizers, such as CF and DLKD, were not found suitable for stabilization or modification of any of the three soil types.

6.3 Cost Analysis

The use of stabilizing materials in a project or using a remove/replace option are largely dependent upon project cost considerations. The following costs were used for comparison purposes and were obtained from MDOT bid documents for projects constructed in years 2005 and 2008. These cost items were modified from bid document pricing to include both materials and work required to complete the operation.

Table 6.3: Costs for I-96 Lime Stabilization (MDOT Project ID: 82123-52803)

Item	Engineer's Estimate	Bid Number 1	Bid Number 2
Lime Stabilization ($/syd) at 4.5%	5.14	3.46	4.64
Subgrade Undercutting ($/syd)	11.21	8.82	20.00

Table 6.4: Costs for I-75/I-96 Lime Stabilization (MDOT Project ID: 82194-37795)

Item	Engineers Estimate	Bid Number 1	Bid Number 2
Lime Stabilization ($/syd) at 5%	5.31	7.65	1.12
Subgrade Undercutting ($/syd)	12.21	13.12	1.32

As seen in Tables 6.3 and 6.4, bid cost items are subject to several variables including estimated quantity of work items, material availability, contractor's preference on certain work items, etc.

A cost analysis was performed to quickly guide MDOT project engineers in the proper selection of a subgrade treatment option during the planning phase. The following assumptions were made during these cost analyses.

Undercut cost = \$10/syd, \$12/syd and \$15/syd

Stabilization = \$3/syd, \$5/syd and \$7/syd

The undercut cost includes all work items required to treat the affected area: excavation and removal of weak material, replacement with recommended material (sound soil or sand/aggregate), and compaction to the recommended density.

The stabilization cost includes stabilizer material cost, mixing, and compaction to the recommended density.

Figure 6.1 shows the variation of different cost as a function of the required project percentage needing treatment. Undercut work is only performed at the area needing removal/replacement. Stabilizing or modification covers the total project area.

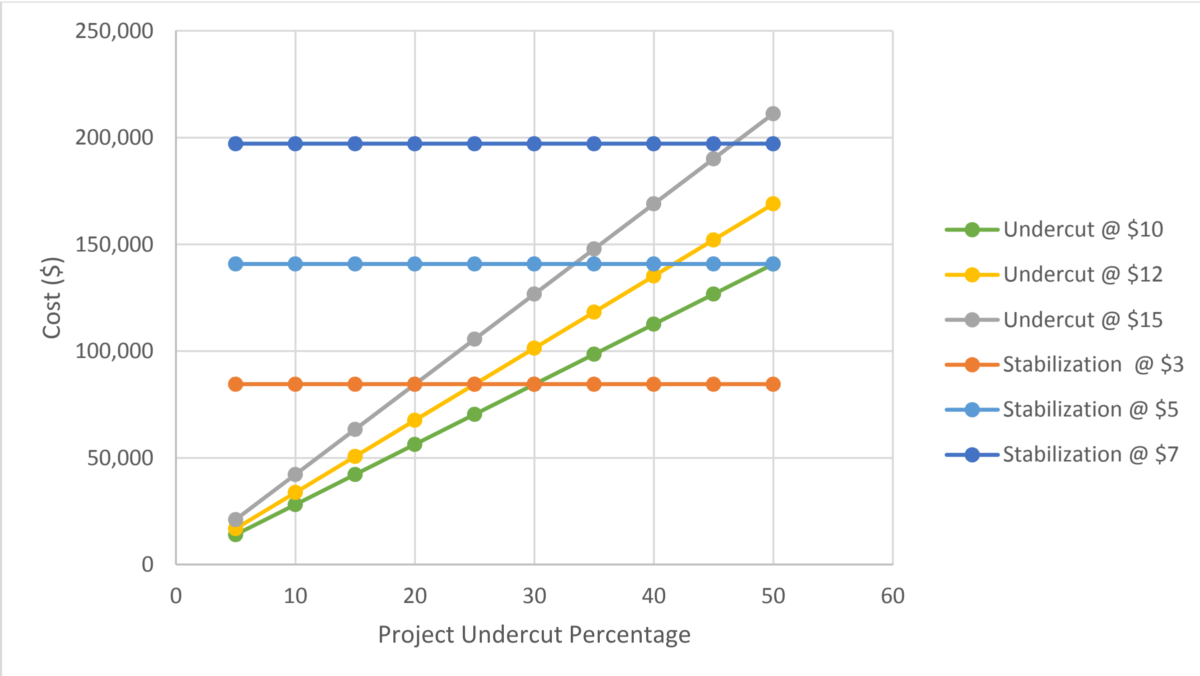


Figure 6.1: Cost Comparisons

This graph shows when stabilization/modification is economically justifiable based on cost and the percentage project area requiring some type of treatment. The percentage project area needing treatment can vary as low as 20% to as high as 50% depending on the undercut cost and stabilization cost.

6.4 Pavement Design Inputs

The following pavement design values that included stabilized subgrades as a structural layer were developed using laboratory tests and limited field study results. The field study revealed that stabilized layers retained strength after a number of freeze/thaw cycles and moisture cycles. However, the laboratory-based freeze/thaw study showed a significant drop in strength after few freeze/thaw cycles and capillary soaking. As stated earlier, these laboratory freeze/thaw cycles were extremely harsh due to the combination of extreme low freezing temperatures (-10°F) and freezing of the water saturated stabilized soils. However, if the properties of the stabilized layers were used for pavement design, it is recommended to have at least 20 inches of cover (subbase, base, and pavement surface) above the stabilized subgrade. This recommendation is based on comparing the field performance of successful stabilized pavement sections.

Table 6.5: Recommended Pavement Design Input Values based on Laboratory Tests

Soil Type	Stabilizing Treatment	Stabilized subgrade Resilient Modulus (psi) for ME Design	AASHTO Layer Coefficient for Stabilized Layer	Effective Modulus of Subgrade Reaction (k_{eff})*
CL (A-6)	CKD	9,800	0.003	213
	LKD/FA	24,000	0.01	241
ML (A-4)	CKD	33,000	0.01	262
	LKD/FA	29,000	0.01	253
CL (A-7-6)	CKD	33,000	0.01	262
	LKD/FA	31,000	0.01	257

*Using a 16-inch base/subbase with a layer moduli of 33,000 psi and a 12-inch stabilized layer

If different pavement base/subbase thicknesses or elastic moduli are used for rigid pavement design using AASHTO 1993 methodology, a composite *k* values should be calculated using ACPA methodology as described in Section 5.5. This can be performed using the online composite modulus of subgrade reaction (k_c) calculator for multiple layers of subbase and subgrade materials. (http://apps.acpa.org/applibrary/KValue/#). Once the composite modulus of subgrade reaction is determined, Loss of Support (LS) chart (Figure 5.6) should be used to calculate the effective modulus of subgrade reaction (k_{eff}).

6.5 Construction Considerations

MDOT currently uses special provisions for the construction of stabilized subgrades. These special provisions are included in Appendix B of this report. In addition, the following items should be considered for inclusion in contract documents.

6.5.1 Sulfate Testing

As many stabilizers may contain calcium, expansion can occur when stabilizers are mixed with soils having a high sulfate content. Michigan and other states reported significant heaving with several projects after chemical stabilization was implemented. Therefore, it is recommended to test soils for sulfate content prior to use of stabilization as a means to treat weak subgrade materials. Generally, soils with more than 10% sulfate should not be considered for chemical stabilization.

6.5.2 Construction Density Control

The Maximum Dry Density of chemically-stabilized soils is always be lower than the maximum dry density of untreated soils. As an example, untreated Soil Sample-1 (CL) is compared to CKD-stabilized soil in Figure 6.2. The inspectors should be made aware of the differences in compaction curves of untreated subgrades and chemically-stabilized subgrades. MDOT's One Point T-99

Chart may not be applicable to chemically-stabilized soils. During field density control, laboratory determined moisture-density charts should be compared with field-obtained density values.

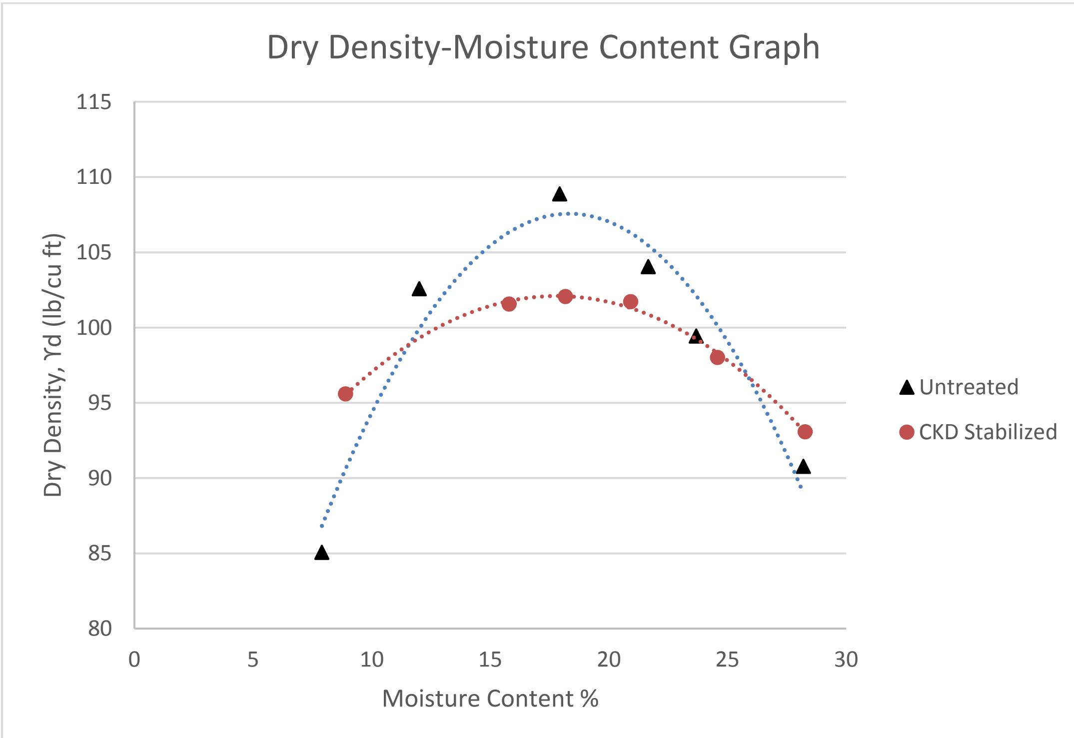


Figure 6.2: Moisture-Density Relationship for Untreated and CKD Stabilized Soils

6.5.3 Construction Quality Control

The uniform application of stabilization materials is important for long-term performance of stabilized subgrades. Generally, this can be achieved by using material application rate testing and Phenolphthalein testing for lime-based materials. Lime-based materials have a high pH meaning they are basic. When phenolphthalein is added to acidic solutions, the solution is clear. When mixed with basic solutions such as lime, the phenolphthalein indicator turns the solution reddish pink. The higher the pH, the stronger the color, the better the stabilization.

The preferred method to determine construction quality control is the Dynamic Cone Penetrometer (DCP) test. The DCP test can be used to determine the degree of stabilization and strength of the stabilized soil. Details of construction quality control using DCP are given in Appendix B "Chemical Stabilization of Subgrade Soils".

6.5.4 Weather Limitations

Laboratory freeze/thaw tests showed a substantial strength decrease of the chemically-stabilized soils due to the freeze/thaw cycles and capillary soaking. Chemical stabilization should always performed when the ambient temperature is at least 40°F and rising. If the overnight temperature is expected to be lower than 40°F, subgrade stabilization should be suspended. Before suspending work for winter, the stabilized subgrade should be covered with a sufficient layer of subbase/base materials to a minimum thickness of 20 inches.

6.6 Recommendations for Further Research

Several recommendations for further research were developed as part of this research project and are shown below.

1. Further research is needed to determine the exact temperature needed to breakdown stabilized subgrade layers. The laboratory freeze/thaw testing has indicated substantial strength loss after few cycles of freezing in a saturated state. However, field investigation results shown, stabilized layers with substantial strength after few years of service with several hundred freeze/thaw cycles and expected varying levels of saturation. If the exact breakdown temperature of stabilized subgrades is known, subgrade stabilization can be recommended even for shallow pavement applications.
2. More research is needed to better understand field performance of stabilized subgrades under freeze/thaw conditions. Due to the limited availability of stabilized subgrades with recycled materials in Michigan, only a few pavement sections were selected for field investigations. More pavement sections stabilized with recycled materials should be included from Michigan and neighboring states to further study the field performance of these stabilized layers.
3. More realistic freeze/thaw testing should be implemented to properly model actual field conditions. This includes determination of field temperature and moisture gradient from the pavement surface to the subgrade and simulation of similar temperature and moisture gradients in the environmental chamber.

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APPENDIX A

LABORATORY TEST DATA

Table A1: Unconfined Compressive Strength Test Data of Untreated Soil-1 (A-6)

	Untreated Soil-1 (A-6)			
Test	**Unsoaked**		**Soaked**	
	Stress, psi	Strain, %	Stress, psi	Strain, %
1	33.44	6.59	2.67	15.02
2	32.14	5.56	2.79	14.83
3	31.19	5.56	2.36	15.02
Average	32.26	5.90	2.61	14.96

Table A2: Unconfined Compressive Strength Test Data of Untreated Soil-2 (A-4)

	Untreated Soil-2 (A-4)			
Test	**Unsoaked**		**Soaked**	
	Stress, psi	Strain, %	Stress, psi	Strain, %
1	37.37	5.31	2.47	15.01
2	34.58	4.75	3.19	15.00
3	36.06	4.36	4.10	15.01
Average	36.00	4.81	3.25	15.01

Table A3: Unconfined Compressive Strength Test Data of Untreated Soil-3 (A-7-6)

	Untreated Soil-3 (A-7-6)			
Test	**Unsoaked**		**Soaked**	
	Stress, psi	Strain, %	Stress, psi	Strain, %
1	69.56	3.66	1.45	14.86
2	54.80	4.36	1.61	14.61
3	63.12	4.37	1.24	15.01
Average	62.49	4.13	1.43	14.83

Table A4: Unconfined Compressive Strength Test Data of CKD Treated Soil-1 (A-6)

		6% CKD, 0 days curing				6% CKD, 1 days curing				6% CKD, 3 days curing				6% CKD, 7 days curing				6% CKD, 14 days curing				6% CKD, 28 days curing			
6% CKD, Soil-1 (A-6)	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	37	6.56	2.71	4.35	50.69	4.56	17.96	6.06	60.42	3.76	28.58	4.74	105.5	4.46	30.73	2.86	-	-	40.47	2.95	-	-	54.3	2.56
	2	34.1	3.86	3.5	4.36	44.7	3.35	24.2	4.96	61.9	3.96	25.2	3.86	95.7	4.46	32.8	3.35	-	-	35.9	2.75	-	-	46	3.16
	3	42.52	5.56	5.82	7.56	49.27	5.56	13.03	4.04	62.86	4.05	32.57	3.66	123.9	4.15	27.42	3.66	-	-	40.63	2.95	-	-	51.76	2.46
	Average	37.88	5.33	4.01	5.42	48.20	4.49	18.38	5.02	61.72	3.92	28.78	4.09	108.36	4.36	30.33	3.29	N/A	N/A	39.01	2.88	N/A	N/A	50.70	2.73

8% CKD, Soil-1 (A-6)

	8% CKD, 0 days curing				**8% CKD, 01 days curing**				**8% CKD, 3 days curing**				**8% CKD, 7days curing**				**8% CKD,14 days curing**				**8% CKD,28 days curing**			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	32.21	7.06	3.72	9.33	50.92	9.09	23.17	5.03	67.32	7.31	44.71	3.15	192.34	4.07	63.56	2.84	-	-	104.3	2.55	-	-	133.6	1.56
2	33.64	7.57	5.75	9.33	47.13	9.84	23.69	4.94	71.93	7.32	51.15	2.55	185.82	3.46	79.88	2.54	-	-	106.3	1.85	-	-	132.6	2.1
3	35.63	6.3	6.83	8.86	48.52	9.59	22.01	4.56	72.88	7.82	40.46	2.35	210.81	3.35	72.28	2.64	-	-	122.2	2.24	-	-	117.1	2.55
Average	33.83	6.98	5.43	9.17	48.86	9.51	22.96	4.84	70.71	7.48	45.44	2.68	196.32	3.63	71.91	2.67	N/A	N/A	110.91	2.21	N/A	N/A	127.73	2.07

12% CKD, Soil-1 (A-6)

Test	12% CKD, 0 days curing				12% CKD, 1 days curing				12% CKD, 3 days curing				12% CKD, 7 days curing				12% CKD, 14 days curing				12% CKD, 28 days curing			
	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	48.63	6.82	10.15	5.55	103.89	4.26	43.78	4.66	155.82	3.35	49.05	3.56	177.68	2.75	77.05	2.44	-	-	109.6	1.86	-	-	141.4	2.05
2	51.26	5.31	9.87	6.31	107.98	3.55	40.22	4.25	145.90	2.35	41.35	2.66	183.01	2.94	76.84	2.95	-	-	109.6	1.65	-	-	144.8	1.55
3	49.49	3.86	7.91	5.3	119.28	4.27	38.31	4.07	158.80	3.46	43.5	2.05	180.39	3.15	79.43	2.96	-	-	109.1	2.55	-	-	156.5	1.84
Average	49.79	5.33	9.31	5.72	110.38	4.03	40.77	4.33	153.51	3.05	44.63	2.76	180.36	2.95	77.77	2.78	N/A	N/A	109.43	2.02	N/A	N/A	147.55	1.81

Table A5: Unconfined Compressive Strength Test Data of CF Treated Soil-1 (A-6)

		4% CF, 0 days curing				4% CF, 1 days curing				4% CF, 3 days curing				4% CF, 7 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
4% CF, Soil-1 (A-6)	1	51.58	5.06	1.91	15	48.14	4.77	2.26	15	56.15	3.96	1.52	13.9	68.28	4.55	3.43	15
	2	42.46	4.87	1.77	15	52.36	4.57	1.56	14.9	57.73	4.78	1.81	15	73.89	3.44	3.93	15
	3	41.53	4.56	1.74	15.02	56.86	3.56	**1.37	**4.75	53.69	3.66	1.54	14.36	82.76	4.75	5.51	15.02
	Average	45.19	4.83	1.81	15.00	52.45	4.30	1.91	14.94	55.86	4.13	1.62	14.42	74.97	4.25	4.29	15.02

** Discarded value

12% CF, Soil-1 (A-6)		12% CF, 0 days curing				12% CF% CKD, 01 day curing				12% CF, 3 days curing				12% CF, 7 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	40.52	6.35	4.41	15	49.43	7.09	4.92	15	50.06	6.31	5.24	15	61.17	6.06	15.7	15
	2	32.73	7.58	3.8	10.1	44.82	6.32	5.19	15	41.22	5.58	5.87	15	75.68	5.32	18.1	15
	3	41.69	6.58	4	15	48.73	6.08	2.94	15	54.02	5.33	5.99	15	73.80	5.57	21.3	15
	Average	38.31	6.84	4.07	13.37	47.66	6.50	4.35	15.01	48.43	5.74	5.70	15.02	70.22	5.65	18.40	15.01

25% CF, Soil-1 (A-6)

Test	**25% CF, 0 days curing**				**25% CF, 1 days curing**				**25% CF, 3 days curing**				**25% CF, 7 days curing**			
	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	33.69	5.57	3.74	15	54.96	5.82	7.73	5.82	51.59	3.87	14.22	10.08	64.79	5.06	**8.34	4.96
2	43.47	4.87	5.07	9.83	54.48	5.33	6.73	6.32	63.39	4.55	19.8	9.33	76.28	3.75	20.2	7.84
3	54.34	5.07	4.13	15	49.62	5.07	4.89	4.16	57.82	4.48	11.9	9.08	76.60	3.87	19.6	9.6
Average	43.83	5.17	4.31	13.29	53.02	5.41	6.45	5.43	57.60	4.30	15.29	9.50	72.56	4.23	19.91	7.47

** Discarded value

Table A6: Unconfined Compressive Strength Test Data of FA Treated Soil-1 (A-6)

		0 days curing				1 days curing				3 days curing				7 days curing				14 days curing				28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
10% Fly Ash, Soil-1 (A-6)	1	43.90	7.32	3.64	10.3	62.67	6.08	5.69	4.95	67.67	5.83	9.65	3.45	95.8	3.56	10.8	3.35	-	-	23.4	1.85	-	-	42.7	1.74
	2	54.11	6.32	4.65	9.95	53.24	7.82	4.89	9.33	57.95	7.07	11.7	2.75	79.5	5.32	12	3.25	-	-	30.4	1.75	-	-	35.5	2.05
	3	49.58	7.33	3.89	6.31	68.26	6.82	5.55	5.56	65.82	5.83	11.1	3.46	80.9	3.65	10	4.06	-	-	27.4	1.65	-	-	30.2	1.74
	Average	49.19	6.99	4.06	8.86	61.39	6.91	5.38	6.61	63.81	6.24	10.81	3.22	85.40	4.18	10.94	3.55	N/A	N/A	27.05	1.75	N/A	N/A	36.15	1.84

15% Fly Ash, Soil-1 (A-6)

	0 days curing				1 days curing				3 days curing				7 days curing				14 days curing				28 days curing			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	61.66	3.45	4.36	15.01	86.84	2.85	4.18	15.01	93.86	2.26	5.97	10.33	124.1	2.35	4.4	5.05	-	-	4.02	4.55	-	-	3.78	3.85
2	54.95	3.05	4.56	15.01	89.75	2.65	4.91	15.02	98.37	2.75	5.25	10.34	122.2	3.05	6.23	7.82	-	-	3.13	7.31	-	-	5.7	15
3	59.90	3.14	4.07	15.01	91.00	2.64	5.23	15	86.19	2.46	5.16	5.56	126.89	2.85	3.49	2.96	-	-	4.02	5.69	-	-	sample	broken
Average	58.84	3.22	4.33	15.01	89.20	2.71	4.77	15.01	92.81	2.49	5.46	8.74	124.39	2.75	4.71	5.28	N/A	N/A	3.72	5.85	N/A	N/A	4.74	9.43

25% Fly Ash, Soil-1 (A-6)	0 days curing				1 days curing				3 days curing				7 days curing				14 days curing				28 days curing			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	64.41	3.65	4.38	15.01	85.18	3.35	4.19	11.08	100.56	2.96	5.08	4.86	149.1	3.06	5.81	3.94	-	-	4.71	3.41	-	-	4.85	3.04
2	67.42	4.06	3.55	15.01	93.27	3.57	4.26	8.06	59.68	2.25	3.58	4.46	140.2	2.55	4.07	4.05	-	-	4.25	4.15	-	-	4.03	2.74
3	66.33	3.15	2.94	14.89	82.00	2.94	4.92	15.01	78.47	2.55	4.93	5.31	161.71	3.76	**2.66	7.57	-	-	2.45	3.96	-	-	Sample	broken
Average	66.06	3.62	3.62	14.97	86.82	3.29	4.46	11.38	79.57	2.59	4.53	4.88	150.35	3.12	4.94	5.19	N/A	N/A	3.80	3.84	N/A	N/A	4.44	2.89

** Discarded value

Table A7: Unconfined Compressive Strength Test Data of LKD+FA Treated Soil-1 (A-6)

		2% LKD & 5% FA, 0 days curing				2% LKD & 5% FA, 1 days curing				2% LKD & 5% FA,3 days curing				2% LKD & 5% FA, 7 days curing				2% LKD & 5% FA, 14 days curing				2% LKD & 5% FA, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
2%LKD+5% FA, Soil-1 (A-6)	1.00	46.14	3.34	3.91	10.70	77.94	3.05	4.70	2.95	85.16	1.45	4.60	3.45	120.96	2.15	8.22	3.66	-	-	9.43	2.95	-	-	9.74	2.56
	2.00	56.41	3.16	4.80	7.83	75.93	2.65	5.75	6.18	92.44	1.64	4.28	3.46	119.00	2.25	8.99	2.95	-	-	11.89	2.95	-	-	Broken	
	3.00	52.49	3.36	3.83	4.76	78.84	2.65	6.46	3.65	86.84	1.66	5.94	3.97	102.60	1.46	8.88	3.46	-	-	7.06	2.85	-	-	9.58	2.85
	Average	51.68	3.29	4.18	7.76	77.57	2.78	5.64	4.26	88.14	1.58	4.94	3.62	114.19	1.95	8.70	3.35	N/A	N/A	9.46	2.92	N/A	N/A	9.66	2.70

5%LKD+15% FA, Soil-1 (A-6)

Test	5% LKD & 15% FA, 0 days curing				5% LKD & 15% FA, 01 days curing				5% LKD & 15% FA, 3 days curing				5% LKD & 15% FA,7days curing				5% LKD & 15% FA,14 days curing				5% LKD & 15% FA,28 days curing			
	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1.00	61.22	3.35	13.63	3.35	181.01	1.95	74.00	1.84	196.95	2.55	85.90	2.53	185.13	2.85	148.36	0.75	-	-	136.03	2.96	263.13	1.64	155.99	1.64
2.00	56.35	3.35	15.90	4.55	145.46	1.75	83.59	1.54	187.74	3.26	135.92	2.26	**149.66	2.18	142.87	0.76	-	-	177.23	1.66	269.71	1.66	177.87	1.75
3.00	48.96	4.35	16.43	3.15	168.93	2.55	66.59	1.95	192.95	2.75	110.21	2.34	200.15	2.45	150.22	1.25	-	-	165.15	1.15	229.52	2.04	168.94	2.05
Average	55.51	3.68	15.32	3.68	165.13	2.08	74.73	1.78	192.55	2.85	110.68	2.38	192.64	2.49	147.15	0.92	N/A	N/A	159.47	1.92	254.12	1.78	167.60	1.82

** Discarded value

3%LKD+9% FA, Soil-1 (A-6)

	3% LKD & 9% FA, 0 days curing				**3% LKD & 9% FA, 01 days curing**				**3% LKD & 9% FA,, 3 days curing**				**3% LKD & 9% FA, 7days curing**				**3% LKD & 9% FA, 14 days curing**				**3% LKD & 9% FA, 28 days curing**			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1.00	69.47	3.35	7.18	3.25	140.86	1.14	16.27	2.25	169.18	2.85	44.64	1.35	181.44	2.35	88.34	3.05	-	-	87.38	1.55	-	-	104.60	1.45
2.00	71.88	3.06	7.51	3.84	157.48	1.74	16.17	1.55	143.13	2.05	59.74	1.44	173.63	1.95	86.69	3.65	-	-	**72.68	1.76	-	-	102.34	2.64
3.00	70.57	3.95	7.59	3.94	145.66	1.95	**55.6	2.45	175.13	2.65	75.94	3.15	169.01	2.54	82.78	3.15	-	-	89.77	2.44	-	-	81.31	2.16
Average	70.64	3.45	7.42	3.68	148.00	1.61	16.22	1.90	162.48	2.52	60.11	1.98	174.69	2.28	85.94	3.29	N/A	N/A	88.58	1.92	N/A	N/A	96.08	2.08

** Discarded value

Table A8: Unconfined Compressive Strength Test Data of LKD & DLKD Treated Soil-1 (A-6)

	Test	6% LKD, 0 days curing				6% LKD, 01 days curing				6% LKD, 03 days curing				6% LKD, 07 days curing				6% LKD, 14 days curing				6% LKD, 28 days curing			
		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
6% LKD, Soil-1 (A-6)	1	67.04	1.95	**3.21	**3.26	90.14	1.94	14.08	1.95	73.48	1.34	**7.29	**0.26	93.52	1.64	21.57	1.34	-	-	30.05	1.04	-	-	**27.52	**0.03
	2	53.31	1.55	7.46	3.65	83.64	1.95	14.06	2.15	82.39	1.75	25.41	0.94	106.4	1.75	24.96	0.81	-	-	**21.59	**0.73	-	-	39.74	0.88
	3	59.6	2.04	6.33	3.85	69.9	1.64	10.5	1.44	96.95	2.15	20.3	0.83	85.2	1.26	32.3	1.04	-	-	40.5	1.04	-	-	34.04	1.14
	Average	59.97	1.85	6.90	3.75	81.23	1.84	12.88	1.85	84.27	1.75	22.87	0.89	95.05	1.55	26.27	1.06	N/A	N/A	35.28	1.04	N/A	N/A	36.89	1.01

** Discarded value

12% DLKD, Soil-1 (A-6)	Test	12% DLKD, 0 days curing				12% DLKD, 01 days curing				12% DLKD, 03 days curing				12% DLKD, 07 days curing				12% DLKD, 14 days curing		12% DLKD, 28 days curing	
		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Soaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	50.2	2.66	3.65	15	49.8	2.84	3.59	4.52	71.90	2.75	8.03	6.31	74.4	2.84	11.6	3.24	8.82	2.55	10.80	2.95
	2	41.4	2.26	3.97	15	50.6	4.05	4.7	5.81	61.45	2.14	7.23	4.45	69.3	2.25	11.3	4.56	11.5	3.16	8.99	2.95
	3	47.5	2.36	3.94	15	61.1	3.26	5.09	5.31	66.89	3.05	7.56	4.06	63.3	2.25	8.86	3.56	11.7	4.16	8.52	3.45
	Average	46.37	2.43	3.85	15.01	53.84	3.38	4.46	5.21	66.75	2.65	7.61	4.94	69.00	2.45	10.59	3.79	10.68	3.29	9.44	3.12

Table A9: Unconfined Compressive Strength Test Data of CKD Treated Soil-2 (A-4)

		6% CKD, 0 days curing				6% CKD, 1 days curing				6% CKD, 3 days curing				6% CKD, 7 days curing				6% CKD, 14 days curing				6% CKD, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
6% CKD, Soil-2 (A-4)	1	41.68	3.44	10.49	6.32	101.68	2.65	46.62	2.55	135.31	1.54	57.80	1.64	204.97	1.94	117.28	1.44	-	-	146.43	1.34	-	-	169.28	1.85
	2	40.21	3.46	8.95	4.87	100.54	2.45	30.33	2.25	166.97	2.35	47.32	1.46	228.20	2.55	107.25	1.85	-	-	131.99	1.85	-	-	169.13	2.15
	3	46.15	2.75	8.34	4.96	106.43	2.46	37.07	2.46	171.75	2.65	56.35	1.63	227.97	2.05	118.38	1.35	-	-	154.89	1.34	-	-	143.42	1.25
	Average	42.68	3.22	9.26	5.38	102.89	2.52	38.01	2.42	158.01	2.18	53.82	1.58	220.38	2.18	114.30	1.55	N/A	N/A	144.44	1.51	N/A	N/A	160.61	1.75

		8% CKD, 0 days curing				8% CKD, 01 days curing				8% CKD, 3 days curing				8% CKD, 7days curing				8% CKD,14 days curing				8% CKD,28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
8% CKD, Soil-2 (A-4)	1	65.38	2.95	7.66	4.25	123.07	2.35	35.93	2.75	197.58	2.46	74.76	2.65	277.82	2.44	100.61	2.84	-	-	113.58	1.84	-	-	210.13	1.34
	2	65.60	2.36	10.24	6.82	153.46	2.64	43.40	3.34	213.26	3.15	62.13	1.84	279.64	2.45	98.61	2.04	-	-	124.17	1.15	-	-	247.17	1.35
	3	66.68	3.75	8.23	5.31	147.92	2.84	36.96	3.46	209.18	2.36	66.99	2.54	307.74	2.85	115.44	2.55	-	-	152.87	1.85	-	-	182.60	1.54
	Average	65.89	3.02	8.71	5.46	141.48	2.61	38.76	3.18	206.67	2.66	67.96	2.34	288.40	2.58	104.89	2.48	N/A	N/A	130.21	1.61	N/A	N/A	213.30	1.41

4% CKD, Soil-2 (A-4)

	4% CKD, 0 days curing				4% CKD, 1 days curing				4% CKD, 3 days curing				4% CKD, 7 days curing				4% CKD, 14 days curing				4% CKD, 28 days curing			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	39.19	4.16	6.96	3.85	83.78	2.34	28.24	1.95	99.51	2.14	52.37	1.92	142.1	2.52	82.02	1.54	-	-	118.1	2.15	-	-	133.3	1.55
2	33.95	3.54	7.55	4.95	78.44	3.21	26.09	1.75	130.15	2.76	42.23	1.25	150.1	2.35	84.74	2.05	-	-	93.76	2.05	-	-	117.5	1.85
3	36.30	3.45	8.96	4.55	75.31	1.74	26.44	1.75	124.24	2.25	36.70	1.85	153.7	3.06	78.43	2.05	-	-	109.9	1.95	-	-	123.4	2.14
Average	36.48	3.72	7.82	4.45	79.18	2.43	26.92	1.82	117.97	2.38	43.77	1.68	148.64	2.64	81.73	1.88	N/A	N/A	107.26	2.05	N/A	N/A	124.74	1.85

Table A10: Unconfined Compressive Strength Test Data of CF Treated Soil-2 (A-4)

		4% CF, 0 days curing				4% CF, 1 days curing				4% CF, 3 days curing				4% CF, 7 days curing				4% CF, 14 days curing				4% CF, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
4% CF, Soil-2 (A-4)	1	18.53	12.09	5.15	15.01	29.17	9.59	5.37	15.00	29.82	10.59	17.57	13.10	29.56	10.33	4.00	8.98	-	-	7.79	10.59	-	-	5.70	4.14
	2	19.91	10.34	5.63	15.03	28.24	13.34	4.88	14.85	28.60	9.32	13.10	11.34	27.38	8.82	11.59	15.00	-	-	23.21	15.00	-	-	8.98	7.82
	3	25.52	10.33	6.29	15.02	27.65	10.33	8.45	15.02	22.21	7.81	21.96	10.33	37.15	8.32	4.88	12.59	-	-	20.67	11.34	-	-	8.89	14.85
	Average	21.32	10.92	5.69	15.02	28.35	11.09	6.23	14.96	26.88	9.24	17.54	11.59	31.36	9.16	6.82	12.19	N/A	N/A	17.22	12.31	N/A	N/A	7.86	8.94

12% CF, Soil-2 (A-4)		12% CF, 0 days curing				12% CF, 01 days curing				12% CF, 3 days curing				12% CF, 7days curing				12% CF,,14 days curing				12% CF, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	38.26	6.31	3.57	15.01	47.37	6.07	5.16	15.01	52.84	6.57	7.16	15.00	52.32	5.05	5.88	15.01	-	-	5.64	15.01	-	-	5.85	14.96
	2	36.71	4.86	2.19	15.02	42.91	6.56	4.42	15.03	46.16	4.06	2.99	15.02	58.35	4.65	5.41	15.01	-	-	7.86	15.03	-	-	5.19	15.01
	3	35.60	5.82	4.29	15.00	53.50	5.55	4.29	15.03	49.63	5.81	3.88	13.60	60.03	3.45	5.13	15.02	-	-	10.13	15.02	-	-	5.48	15.03
	Average	36.86	5.66	3.35	15.01	47.93	6.06	4.63	15.02	49.54	5.48	4.68	14.54	56.90	4.38	5.47	15.01	N/A	N/A	7.88	15.02	N/A	N/A	5.51	15.00

25% CF, Soil-2 (A-4)	25% CF, 0 days curing				25% CF, 1 days curing				25% CF, 3 days curing				25% CF, 7 days curing				25% CF, 14 days curing				25% CF, 28 days curing			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	43.39	5.56	7.79	15.01	37.66	3.66	12.02	10.58	43.31	6.06	14.93	9.58	51.36	5.05	14.44	6.82	-	-	6.77	9.33	-	-	5.28	5.54
2	48.09	3.45	8.16	15.00	43.90	4.86	14.47	10.57	48.71	4.06	16.90	6.06	61.78	4.06	16.89	6.32	-	-	9.95	7.32	-	-	7.31	4.54
3	44.73	4.66	9.14	11.59	50.42	4.96	14.06	9.59	49.22	4.76	9.30	6.32	59.40	4.16	10.16	4.36	-	-	8.12	7.83	-	-	9.71	5.56
Average	45.40	4.56	8.36	13.87	43.99	4.49	13.52	10.25	47.08	4.96	13.71	7.32	57.51	4.42	13.83	5.83	N/A	N/A	8.28	8.16	N/A	N/A	7.43	5.21

Table A11: Unconfined Compressive Strength Test Data of FA Treated Soil-2 (A-4)

		10% FA, 0 days curing				10% FA, 1 days curing				10% FA, 3 days curing				10% FA, 7 days curing				10% FA, 14 days curing				10% FA, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
10% FA, Soil-2 (A-4)	1	42.08	3.85	5.00	15.02	45.08	2.25	3.82	8.33	sample	broken	5.68	3.55	82.99	2.05	2.89	1.75	-	-	9.22	1.85	-	-	14.11	2.44
	2	40.44	3.46	6.38	8.25	54.13	2.05	4.62	5.31	55.63	2.35	5.46	4.87	73.38	2.04	4.15	3.85	-	-	4.51	2.56	-	-	15.7	1.75
	3	44.41	3.76	6.14	10.84	58.16	3.35	5.66	5.06	63.12	2.34	4.68	2.34	78.50	2.15	5.28	3.25	-	-	10.54	2.65	-	-	10.6	3.45
	Average	42.31	3.69	5.84	11.37	52.46	2.55	4.70	6.23	59.37	2.34	5.27	3.59	78.29	2.08	4.10	2.95	N/A	N/A	8.09	2.35	N/A	N/A	13.47	2.55

15% FA, Soil-2 (A-4)

	15% FA, 0 days curing				**15% FA, 01 days curing**				**15% FA, 3 days curing**				**15% FA, 7days curing**				**15% FA,14 days curing**				**15% FA,28 days curing**			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	42.94	3.36	3.96	15.01	75.59	2.04	sample	broken	72.92	2.94	8.71	3.14	84.69	1.95	22.89	2.25	-	-	25.03	2.25	-	-	41.98	1.54
2	53.48	3.14	4.20	13.59	79.97	2.15	3.92	3.96	87.82	2.35	12.81	2.55	88.38	2.25	21.90	2.05	-	-	11.69	1.55	-	-	43.04	1.95
3	53.81	2.14	5.14	15.03	71.42	2.35	5.18	15.02	81.44	2.75	10.94	2.04	88.58	2.35	20.16	2.34	-	-	26.24	2.85	-	-	43.64	2.74
Average	50.08	2.88	4.43	14.54	75.66	2.18	4.55	9.49	80.73	2.68	10.82	2.58	87.22	2.18	21.65	2.21	N/A	N/A	20.99	2.22	N/A	N/A	42.89	2.08

		25% FA, 0 days curing				25% FA, 1 days curing				25% FA, 3 days curing				25% FA, 7 days curing				25% FA,, 14 days curing				25% FA, 28 days curing			
25% FA, Soil-2 (A-4)	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	46.1	2.95	2.01	15	105	3.36	2.49	14.96	94.9	2.15	4.06	2.44	88.6	2.82	7.04	2.55	-	-	41.5	2.15	-	-	67.25	1.14
	2	60.7	3.75	3.15	15	90.7	2.76	1.79	3.06	96.7	2.84	3.5	3.05	72.4	3.16	15.3	2.95	-	-	44.7	0.94	-	-	51.41	1.55
	3	52.6	2.85	3.08	14.1	76.9	2.34	2.07	3.66	84.4	1.54	2.45	2.85	74.3	2.84	20.2	2.35	-	-	46.4	1.85	-	-	67.87	2.06
	Average	53.13	3.18	2.75	14.72	90.75	2.82	2.12	7.23	92.00	2.18	3.34	2.78	78.40	2.94	14.15	2.62	N/A	N/A	44.21	1.65	N/A	N/A	62.18	1.58

Table A12: Unconfined Compressive Strength Test Data of LKD+FA Treated Soil-2 (A-4)

		2% LKD & 8% FA, 0 days curing				2% LKD & 8% FA, 1 days curing				2% LKD & 8% FA, 3 days curing				2% LKD & 8% FA, 7 days curing				2% LKD & 8% FA, 14 days curing				2% LKD & 8% FA, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
2%LKD+8% FA, Soil-2 (A-4)	1	57.97	3.15	8.55	2.95	135.16	2.65	72.37	0.89	163.36	2.46	76.28	1.15	**169.67	2.75	94.05	1.54	-	-	112.41	1.85	-	-	146.42	1.24
	2	65.72	3.15	9.92	2.35	162.60	3.86	83.44	1.25	208.12	2.84	74.59	2.15	203.23	3.45	98.15	2.56	-	-	95.44	2.15	-	-	108.88	0.96
	3	87.90	2.95	11.35	3.65	178.41	2.35	80.64	1.63	190.05	2.56	70.20	1.74	209.38	2.34	84.78	2.14	-	-	118.30	1.75	-	-	122.06	1.15
	Average	70.53	3.08	9.94	2.98	158.72	2.95	78.82	1.26	187.18	2.62	73.69	1.68	206.30	2.85	92.33	2.08	N/A	N/A	108.72	1.92	N/A	N/A	125.79	1.12

** Discarded value

2%LKD+5% FA, Soil-2 (A-4)

	2% LKD & 5% FA, 0 days curing				**2% LKD & 5% FA, 01 days curing**				**2% LKD & 5% FA, 3 days curing**				**2% LKD & 5% FA, 7days curing**				**2% LKD & 5% FA,14 days curing**				**2% LKD & 5% FA,28 days curing**			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	87.08	2.85	-	-	118.38	2.85	55.45	1.24	152.20	2.46	82.98	0.97	160.69	3.05	84.99	2.05	-	-	107.24	1.02	-	-	88.14	2.54
2	49.64	2.64	10.43	2.96	120.05	2.55	52.98	1.64	134.51	2.16	78.77	1.55	154.35	2.53	78.59	1.45	-	-	119.64	1.24	-	-	82.69	1.45
3	77.99	2.45	10.16	2.75	141.33	3.05	11.38	1.35	149.48	2.04	72.43	1.44	166.86	2.67	92.57	1.65	-	-	103.07	1.15	-	-	97.72	1.25
Average	82.53	2.65	10.29	2.85	126.59	2.82	54.21	1.41	145.40	2.22	78.06	1.32	160.63	2.75	85.38	1.71	N/A	N/A	109.98	1.14	N/A	N/A	89.52	1.75

Table A13: Unconfined Compressive Strength Test Data of LKD & DLKD Treated Soil-2 (A-4)

	Test	4% LKD, 0 days curing				4% LKD, 01 days curing				4% LKD, 03 days curing				4% LKD, 07 days curing				4% LKD, 14 days curing		4% LKD, 28 days curing	
		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Soaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
4% LKD, Soil-2 (A-4)	1	19.92	2.34	2.32	4.75	34.06	3.54	15.41	1.35	45.12	2.76	13.12	2.53	43.25	1.85	15.67	2.35	19.06	4.15	23.94	2.15
	2	23.00	3.34	4.57	4.56	47.20	3.26	10.56	3.16	23.93	0.87	13.86	4.16	29.50	3.05	15.74	2.05	15.27	4.56	24.62	3.25
	3	15.68	1.74	1.92	4.25	27.86	3.15	13.53	3.85	59.74	3.15	12.56	2.35	37.53	3.25	16.06	1.75	15.65	5.31	25.82	2.85
	Average	19.53	2.48	2.94	4.52	36.38	3.32	13.17	2.79	42.93	2.26	13.18	3.01	36.76	2.71	15.82	2.05	16.66	4.67	24.80	2.75

17% DLKD, Soil-2 (A-4)

Test	17% DLKD, 0 days curing				17% DLKD, 01 days curing				17% DLKD, 03 days curing				17% DLKD, 07 days curing				17% DLKD, 14 days curing		17% DLKD, 28 days curing	
	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Soaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	59.88	3.45	5.90	15.03	50.97	2.95	7.43	4.15	63.78	2.46	20.16	4.46	81.36	2.64	33.04	3.76	44.08	3.84	36.02	2.85
2	59.17	3.55	3.43	7.82	47.16	3.64	8.66	4.26	**31.82	**3.95	22.27	4.36	64.66	2.16	27.49	1.96	65.17	3.35	-	-
3	54.02	3.65	5.48	7.57	42.72	3.15	9.95	6.57	64.88	3.36	19.86	4.56	67.28	2.76	39.76	3.75	43.06	2.36	78.24	3.55
Average	57.69	3.55	4.94	10.14	46.95	3.25	8.68	5.00	64.33	2.91	20.76	4.46	71.10	2.52	33.43	3.15	50.77	3.18	57.13	3.20

** Discarded value

Table A 14: Unconfined Compressive Strength Test Data of CKD Treated Soil-3 (A-7-6)

		6% CKD, 0 days curing				6% CKD, 1 days curing				6% CKD, 3 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
6% CKD, Soil-3 (A-7-6)	1	71.25	3.24	12.51	3.36	218.58	2.35	36.79	1.34	214.82	2.65	105.37	1.54
	2	81.80	3.46	14.67	3.85	167.73	1.55	56.04	1.75	192.35	2.76	83.81	2.15
	3	82.69	2.85	15.88	4.15	160.08	2.25	41.17	1.75	262.59	2.55	97.23	2.45
	Average	78.58	3.18	14.35	3.79	163.91	2.05	44.67	1.61	223.26	2.66	95.47	2.05

		6% CKD, 7 days curing				6% CKD, 14 days curing				6% CKD, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
6% CKD, Soil-3 (A-7-6)	1	203.85	1.75	129.40	2.45	-	-	180.95	1.95	-	-	152.22	1.45
	2	236.25	2.05	97.40	2.95	-	-	140.11	1.24	-	-	145.52	1.45
	3	288.85	2.26	88.35	1.25	-	-	146.01	1.75	-	-	107.39	1.94
	Average	242.98	2.02	105.05	2.22	N/A	N/A	155.69	1.65	N/A	N/A	148.87	1.61

** Discarded value

8% CKD, Soil-3 (A-7-6)		8% CKD, 0 days curing				8% CKD, 01 days curing				8% CKD, 3 days curing				8% CKD, 7days curing				8% CKD,14 days curing				8% CKD,28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	69.57	2.06	19.02	2.45	154.69	1.85	74.17	1.71	241.68	2.76	101.76	1.55	229.89	2.05	95.64	2.25	-	-	230.20	1.55	-	-	201.25	1.76
	2	56.36	1.94	20.19	3.06	162.05	2.26	62.70	1.76	209.80	1.65	124.17	2.86	233.52	2.04	134.97	3.05	-	-	152.17	2.45	-	-	221.71	2.55
	3	68.74	2.45	18.96	3.06	146.46	2.65	66.43	2.15	209.91	2.05	111.45	2.66	254.72	2.25	169.68	1.84	-	-	164.13	3.36	-	-	218.75	2.35
	Average	64.89	2.15	19.39	2.86	154.40	2.26	67.77	1.87	220.46	2.15	112.46	2.36	239.38	2.11	133.43	2.38	N/A	N/A	182.17	2.45	N/A	N/A	213.90	2.22

4% CKD, Soil-3 (A-7-6)

Test	**4% CKD, 0 days curing**				**4% CKD, 1 days curing**				**4% CKD, 3 days curing**				**4% CKD, 7 days curing**				**4% CKD, 14 days curing**				**4% CKD, 28 days curing**			
	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	114.43	4.56	16.18	4.15	137.04	4.86	52.36	1.55	171.53	3.75	69.24	1.54	151.04	2.76	79.57	4.44	-	-	86.38	2.25	-	-	140.76	2.15
2	101.34	4.85	9.32	2.95	110.60	3.87	50.36	3.45	171.94	2.85	52.39	2.45	151.04	5.31	55.51	2.45	-	-	91.64	2.95	-	-	112.90	3.05
3	90.65	3.96	13.49	4.86	137.48	3.66	61.63	3.55	185.21	2.34	61.75	2.95	195.17	2.35	109.19	1.65	-	-	115.17	3.45	-	-	97.91	2.24
Average	102.14	4.46	13.00	3.99	128.38	4.13	54.79	2.85	176.23	2.98	61.13	2.32	165.75	3.47	81.42	2.84	N/A	N/A	97.73	2.88	N/A	N/A	117.19	2.48

Table A15: Unconfined Compressive Strength Test Data of CF Treated Soil-3 (A-7-6)

		4% CF, 0 days curing				4% CF, 1 days curing				4% CF, 3 days curing				4% CF, 7 days curing				4% CF, 14 days curing				4% CF, 28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
4% CF, Soil-3 (A-7-6)	1	98.12	3.96	5.48	7.58	82.92	3.96	2.66	15.01	78.66	4.44	8.75	15.01	52.97	3.06	4.46	12.86	-	-	4.69	9.07	-	-	6.15	10.09
	2	84.84	3.35	6.04	12.34	77.82	3.94	2.96	15.01	58.06	3.66	6.68	13.35	73.66	3.05	4.02	12.35	-	-	6.89	11.83	-	-	4.96	7.32
	3	121.75	4.67	4.94	12.83	86.07	3.56	4.84	15.01	78.58	3.75	7.39	15.01	74.10	3.46	4.27	7.83	-	-	4.42	14.85	-	-	6.14	7.07
	Average	101.57	4.00	5.49	10.92	82.27	3.82	3.49	15.01	71.77	3.95	7.60	14.46	66.91	3.19	4.25	11.01	N/A	N/A	5.33	11.92	N/A	N/A	5.75	8.16

15% CF, Soil-3 (A-7-6)		**15% CF, 0 days curing**				**15% CF, 01 days curing**				**15% CF, 3 days curing**				**15% CF, 7days curing**				**15% CF,,14 days curing**				**15% CF, 28 days curing**			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	49.72	5.57	3.11	11.84	60.10	4.75	5.35	6.07	50.29	4.27	8.34	3.95	51.86	3.75	10.20	4.16	-	-	4.77	7.07	-	-	10.09	4.56
	2	49.53	4.66	4.54	10.33	49.55	4.16	6.50	3.76	58.02	5.06	10.53	4.95	48.28	2.65	3.58	8.85	-	-	12.33	5.31	-	-	broken	sample
	3	43.24	4.56	4.26	6.57	39.92	3.35	8.16	6.07	55.21	5.05	4.89	3.75	48.93	5.82	5.95	3.35	-	-	7.95	3.45	-	-	9.75	4.46
	Average	47.50	4.93	3.97	9.58	49.86	4.09	6.67	5.30	54.51	4.79	7.92	4.21	49.69	4.07	6.58	5.45	N/A	N/A	8.35	5.28	N/A	N/A	9.92	4.51

25% CF, Soil-3 (A-7-6)

	25% CF, 0 days curing				25% CF, 1 days curing				25% CF, 3 days curing				25% CF, 7 days curing				25% CF, 14 days curing				25% CF, 28 days curing			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	55.47	3.45	6.55	7.31	47.34	4.05	4.26	5.06	48.05	4.76	9.83	4.16	59.47	4.85	**2.45	**9.57	-	-	15.83	4.15	-	-	14.33	3.56
2	80.43	5.31	4.75	7.58	39.29	5.06	5.26	4.45	62.87	4.45	9.69	3.45	59.41	4.16	broken	sample	-	-	6.58	4.86	-	-	12.44	4.55
3	65.77	4.75	4.25	6.81	48.43	3.55	3.35	4.66	64.02	4.47	9.10	3.56	48.49	3.04	13.30	3.25	-	-	15.30	3.65	-	-	6.44	3.25
Average	67.22	4.50	5.18	7.24	45.02	4.22	4.29	4.72	58.31	4.56	9.54	3.72	55.79	4.02	13.30	3.25	N/A	N/A	12.57	4.22	N/A	N/A	11.07	3.79

** Discarded value

Table A16: Unconfined Compressive Strength Test Data of FA Treated Soil-3 (A-7-6)

		10% FA, 0 days curing				**10% FA, 1 days curing**				**10% FA, 3 days curing**				**10% FA, 7 days curing**				**10% FA, 14 days curing**				**10% FA, 28 days curing**			
10% FA, Soil-3 (A-7-6)	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	43.44	8.82	5.42	4.66	78.00	5.06	6.21	3.44	90.97	4.95	7.59	2.76	127.88	3.06	27.96	2.36	-	-	49.72	2.55	-	-	37.60	3.76
	2	61.08	6.07	4.46	5.04	99.47	5.05	6.27	3.35	77.55	4.86	10.44	3.05	93.54	5.55	21.79	1.94	-	-	42.96	2.45	-	-	48.65	1.84
	3	76.35	6.57	6.41	5.55	97.81	4.75	6.46	4.34	138.93	2.85	6.97	4.26	112.20	3.24	23.03	1.25	-	-	32.86	1.14	-	-	33.66	2.64
	Average	60.29	7.15	5.43	5.09	91.76	4.95	6.31	3.71	102.48	4.22	8.34	3.35	111.21	3.95	24.26	1.85	N/A	N/A	41.85	2.05	N/A	N/A	39.97	2.75

15% FA, Soil-3 (A-7-6)	15% FA, 0 days curing				15% FA, 01 days curing				15% FA, 3 days curing				15% FA, 7days curing				15% FA,14 days curing				15% FA,28 days curing			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	52.37	4.45	5.42	6.82	100.51	3.46	27.80	2.45	108.65	4.06	54.75	2.55	118.63	1.85	69.76	3.86	-	-	59.83	3.86	-	-	85.57	2.25
2	55.00	3.75	3.62	5.57	94.03	4.25	28.04	4.37	77.18	3.45	35.13	2.75	130.77	3.26	64.54	4.45	-	-	72.59	1.95	-	-	47.12	2.75
3	48.88	6.06	3.99	5.31	71.65	3.36	33.19	5.57	87.53	3.45	54.55	4.26	124.06	3.94	69.67	2.35	-	-	63.42	4.46	-	-	93.82	2.45
Average	52.08	4.75	4.34	5.90	88.73	3.69	29.68	4.13	91.12	3.65	48.14	3.19	124.49	3.02	67.99	3.55	N/A	N/A	65.28	3.42	N/A	N/A	75.50	2.48

25% FA, Soil-3 (A-7-6)	25% FA, 0 days curing				25% FA, 1 days curing				25% FA, 3 days curing				25% FA, 7 days curing				25% FA,, 14 days curing				25% FA, 28 days curing			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	85.57	4.96	5.12	8.08	84.04	3.36	8.77	3.34	104.34	2.76	62.85	2.25	**63.62	2.57	63.86	2.25	-	-	99.03	1.54	-	-	59.54	1.52
2	56.17	4.36	4.91	6.82	79.24	2.56	9.72	3.16	118.49	2.35	22.13	4.45	117.43	2.05	94.53	1.25	-	-	81.05	2.05	-	-	88.62	3.65
3	68.15	3.26	6.73	7.05	94.38	2.85	10.21	3.36	93.25	2.15	57.92	1.55	122.47	2.15	33.31	1.34	-	-	54.06	3.85	-	-	74.42	1.75
Average	69.96	4.19	5.59	7.32	85.89	2.92	9.57	3.29	105.36	2.42	47.63	2.75	119.95	2.26	63.90	1.61	N/A	N/A	78.05	2.48	N/A	N/A	74.19	2.31

** Discarded value

Table A17: Unconfined Compressive Strength Test Data of LKD+FA Treated Soil-3 (A-7-6)

		2% LKD & 8% FA, 0 days curing				2% LKD & 8% FA, 1 days curing				2% LKD & 8% FA, 3 days curing				2% LKD & 8% FA, 7 days curing				2% LKD & 8% FA, 14 days curing				2% LKD & 8% FA, 28 days curing			
2%LKD+8% FA, Soil-3 (A-7-6)	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	71.35	2.25	4.58	5.81	80.12	3.45	broken	sample	81.46	1.44	32.48	1.35	98.47	1.44	55.00	2.15	-	-	48.55	3.05	-	-	55.24	1.05
	2	62.74	2.44	4.67	14.60	72.05	2.26	14.20	0.90	94.50	2.25	22.30	1.84	93.76	1.75	40.70	2.24	-	-	56.08	3.55	-	-	19.35	2.75
	3	58.24	2.15	5.22	5.31	93.21	2.25	17.73	2.35	72.51	3.24	35.69	3.55	77.83	3.04	45.62	2.65	-	-	34.49	3.24	-	-	72.64	2.05
	Average	64.11	2.28	4.82	8.57	81.80	2.65	15.96	1.62	82.83	2.31	30.15	2.25	90.02	2.08	47.11	2.35	N/A	N/A	46.38	3.28	N/A	N/A	63.94	1.95

** Discarded value

2%LKD+5% FA, Soil-3 (A-7-6)		2% LKD & 5% FA, 0 days curing				2% LKD & 5% FA, 01 days curing				2% LKD & 5% FA, 3 days curing				2% LKD & 5% FA, 7days curing				2% LKD & 5% FA,14 days curing				2% LKD & 5% FA,28 days curing			
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
	1	68.18	2.85	4.16	4.85	119.45	2.15	10.28	2.55	110.43	1.81	22.26	1.85	115.93	3.65	37.81	3.45	-	-	55.76	2.45	-	-	55.27	1.45
	2	67.17	2.15	5.75	5.56	107.24	1.95	9.40	2.85	96.32	1.75	19.25	2.66	101.52	2.95	55.68	1.24	-	-	43.62	1.15	-	-	57.84	1.56
	3	82.45	2.16	4.00	3.56	101.36	2.25	11.17	2.85	110.45	1.55	21.82	2.05	111.63	3.25	43.03	1.84	-	-	64.18	0.99	-	-	57.41	1.34
	Average	72.60	2.39	4.64	4.66	109.35	2.12	10.28	2.75	105.74	1.70	21.11	2.19	109.69	3.28	45.51	2.18	N/A	N/A	54.52	1.53	N/A	N/A	56.84	1.45

3%LKD+9% FA, Soil-3 (A-7-6)

	3% LKD & 9% FA, 0 days curing				**3% LKD & 9% FA, 01 days curing**				**3% LKD & 9% FA, 3 days curing**				**3% LKD & 9% FA, 7days curing**				**3% LKD & 9% FA,14 days curing**				**3% LKD & 9% FA,28 days curing**			
Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked	
	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
1	64.40	3.15	15.71	3.45	139.43	3.15	82.96	2.55	128.46	2.24	84.34	3.06	128.9	2.04	**152.55	1.65	-	-	104.31	4.05	-	-	133.78	4.96
2	66.57	3.45	11.25	2.35	134.98	3.05	76.82	2.75	120.77	2.76	101.08	3.36	broken	sample	139.10	3.36	-	-	139.27	3.86	-	-	125.81	4.76
3	72.13	3.96	8.88	1.74	137.12	2.45	63.04	3.55	115.40	2.45	82.80	3.05	116.7	1.65	121.15	2.45	-	-	143.32	3.25	-	-	132.12	4.45
Average	67.70	3.52	11.94	2.52	137.18	2.88	74.27	2.95	121.54	2.48	89.41	3.16	122.81	1.85	130.12	2.48	N/A	N/A	128.97	3.72	N/A	N/A	130.57	4.73

** Discarded value

Table A18: Unconfined Compressive Strength Test Data of LKD & DLKD Treated Soil-3 (A-7-6)

	Test	6% LKD, 0 days curing				6% LKD, 01 days curing				6% LKD, 03 days curing				6% LKD, 07 days curing				6% LKD, 14 days curing		6% LKD, 28 days curing	
		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Soaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
6%LKD, Soil-3 (A-7-6)	1	40.04	3.25	5.15	2.85	46.87	3.35	25.98	2.05	63.24	2.75	16.75	3.65	22.06	3.65	30.63	3.36	31.76	4.95	47.16	2.64
	2	43.32	3.06	4.95	2.05	45.32	5.83	23.71	1.65	38.63	1.85	17.27	2.75	60.99	3.66	38.12	3.05	28.40	3.76	46.51	2.34
	3	25.26	1.75	7.32	2.75	46.03	3.15	31.16	2.05	31.02	2.25	11.09	0.68	55.67	1.85	37.97	1.95	33.15	3.84	48.04	3.65
	Average	36.21	2.68	5.81	2.55	46.07	4.11	26.95	1.92	44.29	2.28	15.03	2.36	46.24	3.05	35.57	2.78	31.10	4.19	47.24	2.88

		16% DLKD, 0 days curing				16% DLKD, 01 days curing				16% DLKD, 03 days curing				16% DLKD, 07 days curing				16% DLKD, 14 days curing		16% DLKD, 28 days curing	
	Test	Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Unsoaked		Soaked		Soaked		Soaked	
		Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
16%LKD, Soil-3 (A-7-6)	1	59.71	3.66	1.65	5.53	73.01	2.06	10.12	1.56	68.83	5.56	10.96	2.24	116.30	3.25	31.33	4.46	32.19	3.96	45.51	1.55
	2	64.12	3.06	5.46	5.56	71.03	2.25	13.95	2.75	38.74	6.57	16.26	1.75	116.29	3.15	28.11	3.25	39.20	4.26	44.64	1.94
	3	57.25	2.95	broken	sample	50.06	3.36	9.48	1.74	**20.36	**2.35	13.95	2.36	113.89	3.16	24.46	3.15	43.84	6.09	45.35	3.36
	Average	60.36	3.22	3.55	5.54	64.70	2.55	11.18	2.02	53.78	6.07	13.72	2.12	115.49	3.19	27.96	3.62	38.41	4.77	45.17	2.28

** Discarded value

Table A19: 96-hr Soaked California Bearing Ratio Test Data

		Test-1		Test-2		Test-3		Average		CBR	Increase (%)
		CBR-1	CBR-2	CBR-1	CBR-2	CBR-1	CBR-2	CBR-1	CBR-2		
Soil-1 (CL, A-6)	Untreated	3.9	4.43	2.4	3.13	2.4	2.8	2.9	3.5	3.5	0
	8%CKD	7.2	8	5.8	6.67	9	10	7.3	8.2	8.2	138
	3%LKD+9%FA	27.4	24.2	32.8	31.53	40	29.67	33.4	28.5	33.4	867
Soil-2 (ML, A-4)	Untreated	2	2.57	1.98	2.42	2	2.43	2.0	2.5	2.5	0
	4%CKD	51	59.33	50.7	58.33	48.1	51.33	49.9	56.3	56.3	2177
	2%LKD+5%FA	45.1	41.07	54.4	50.67	32.5	43	44.0	44.9	44.9	1716
Soil-3 (ML, A-7-6)	Untreated	10.08	8.11	4.12	4.45	6	6.31	6.7	6.3	6.7	0
	4%CKD	76.4	75.13	48.7	49.19	40.88	38.47	55.3	54.3	55.3	722
	3%LKD+9%FA	44.7	47.12	43.34	38.33	61.4	56.53	49.8	47.3	49.8	640
	15% FA	39	36.27	37.55	32.63	30.4	34.53	35.7	34.5	35.7	429

Table A20: UnSoaked California Bearing Ratio Test Data

		Test-1		Test-2		Test-3		Average		CBR	Increase (%)
		CBR-1	CBR-2	CBR-1	CBR-2	CBR-1	CBR-2	CBR-1	CBR-2		
Soil-1 (CL, A-6)	Untreated	22.7	24	16.9	18.83	10.52	15.92	16.7	19.6	19.6	0
	8%CKD	31.95	32.43	20.96	21.23	21.44	26.91	24.8	26.9	26.9	37
	3%LKD+9%FA	39.5	41.6	24.27	26.6	33.9	35	32.6	34.4	34.4	76
Soil-2 (ML, A-4)	Untreated	9.5	13.9	19.5	20	15.5	18.7	14.8	17.5	17.5	0
	4%CKD	26.4	30.5	22.3	24.3	24.3	24.4	24.3	26.4	26.4	51
	2%LKD+5%FA	41.5	42.4	38.1	35	29	27.4	36.2	34.9	36.2	106
Soil-3 (ML, A-7-6)	Untreated	22.74	23.64	23.8	22.8	28.4	23.99	25.0	23.5	25.0	0
	4%CKD	23	23.6	25.6	22	77.5	78	42.0	41.2	42.0	68
	3%LKD+9%FA	29.4	32.5	31.7	30.7	45.5	44.6	35.5	35.9	35.9	44
	15% FA	25.1	26.1	26.2	26.6	30.9	32.6	27.4	28.4	28.4	14

Table A21: Laboratory Freeze/thaw Test Data (Capillary soaking at the end of design cycle)

		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-1	**8% CKD**	1	63.56	2.84	50.85	3.36	46.57	2.95	25.24	3.76	11.30	5.07
		2	79.88	2.54	**34.34	**2.35	44.61	3.06	15.04	4.15	7.90	4.97
		3	72.28	2.64	49.33	3.16	44.80	2.55	26.13	4.36	9.93	5.57
		Average	71.91	2.67	50.09	3.26	45.32	2.85	22.14	4.09	9.71	5.20
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-1	**3%LKD + 9%FA**	1	88.34	3.05	**3.61	**3.90	33.93	2.56	28.02	2.15	21.74	4.05
		2	86.69	3.65	73.73	2.66	**24.43	**3.55	45.77	2.45	33.89	2.55
		3	82.78	3.15	64.77	2.55	55.64	4.01	46.40	2.54	16.21	4.36
		Average	85.94	3.29	69.25	2.60	44.78	3.28	40.06	2.38	23.94	3.66
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-2	**4% CKD**	1	82.02	1.54	22.74	3.15	12.08	6.06	10.44	5.33	12.43	4.73
		2	84.74	2.05	25.00	4.05	12.09	5.05	10.65	4.97	10.33	4.35
		3	78.43	2.05	25.55	3.45	14.49	5.05	8.36	4.76	11.83	4.86
		Average	81.73	1.88	24.43	3.55	12.89	5.39	9.82	5.02	11.53	4.65
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-2	**2%LKD + 5%FA**	1	84.99	2.05	7.34	5.31	7.25	5.31	6.77	4.76	7.16	3.56
		2	78.59	1.45	9.16	4.65	6.89	4.66	7.12	4.86	4.18	5.05
		3	92.57	1.65	11.86	4.65	7.30	5.30	7.42	4.46	5.11	3.85
		Average	85.38	1.71	9.45	4.87	7.15	5.09	7.10	4.70	5.48	4.15

Soil	Additive	Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-3	**4% CKD**	1	79.57	4.44	49.84	3.76	35.27	3.85	30.03	4.06	33.55	3.14
		2	55.51	2.45	42.79	2.76	33.00	3.56	34.26	4.35	27.87	3.66
		3	109.19	1.65	63.30	2.95	24.26	3.35	37.64	3.75	17.82	3.45
		Average	81.42	2.84	51.98	3.15	30.84	3.59	33.98	4.05	26.41	3.42
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-3	**3%LKD + 9%FA**	1	**152.55	**1.65	78.17	3.05	46.25	3.05	50.54	3.96	24.49	3.25
		2	139.10	3.36	66.17	2.95	48.61	3.25	37.76	3.96	46.76	3.16
		3	121.15	2.45	80.78	2.76	54.26	3.15	54.57	4.46	36.99	1.45
		Average	130.12	2.48	75.04	2.92	49.71	3.15	47.63	4.12	36.08	2.62
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-3	**15% FA**	1	69.76	3.86	19.43	4.86	**5.98	**2.45	24.59	5.57	17.72	4.45
		2	64.54	4.45	16.78	3.46	15.31	4.16	19.94	5.57	17.37	6.07
		3	69.67	2.35	12.98	4.45	16.39	5.31	**29.43	**4.17	18.26	3.95
		Average	67.99	3.55	16.40	4.26	15.85	4.73	22.26	5.10	17.79	4.82

** Discarded Value

Table A22: Laboratory Freeze/thaw Test Data (Capillary soaking during every thawing period)

		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-1	**8% CKD**	1	63.56	2.84	37.04	0.87	17.99	0.63	4.76	1.44	broken	sample
		2	79.88	2.54	51.65	4.56	16.40	1.54	5.61	2.04	broken	sample
		3	72.28	2.64	55.32	3.25	18.82	1.55	8.23	3.05	broken	sample
		Average	71.91	2.67	48.00	2.89	17.74	1.24	6.20	2.18	N/A	N/A
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-1	**3%LKD + 9%FA**	1	88.34	3.05	80.53	2.34	29.63	3.75	14.14	4.75	broken	sample
		2	86.69	3.65	83.80	2.95	17.82	4.65	10.50	3.36	broken	sample
		3	82.78	3.15	74.13	2.35	20.23	4.46	11.57	4.65	broken	sample
		Average	85.94	3.29	79.49	2.55	22.56	4.29	12.07	4.25	N/A	N/A
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-2	**4% CKD**	1	82.02	1.54	60.10	1.83	22.92	3.84	8.72	3.36	broken	sample
		2	84.74	2.05	74.31	3.05	24.64	4.25	14.39	5.30	broken	sample
		3	78.43	2.05	57.69	3.25	34.92	4.45	11.59	3.75	broken	sample
		Average	81.73	1.88	64.03	2.71	27.50	4.18	11.57	4.14	N/A	N/A
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-2	**2%LKD + 5%FA**	1	84.99	2.05	9.83	2.95	10.73	5.31	4.77	6.50	broken	sample
		2	78.59	1.45	13.59	3.06	9.74	5.05	3.43	3.65	broken	sample
		3	92.57	1.65	22.04	3.55	8.04	4.05	4.97	4.35	broken	sample
		Average	85.38	1.71	15.15	3.19	9.50	4.81	4.39	4.83	N/A	N/A

		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-3	**4% CKD**	1	79.57	4.44	67.97	2.95	29.25	5.04	13.34	4.16	broken	sample
		2	55.51	2.45	60.58	4.56	27.49	3.85	19.60	5.05	4.21	6.56
		3	109.19	1.65	52.86	2.04	27.69	6.07	14.06	6.57	13.23	4.67
		Average	81.42	2.84	60.47	3.18	28.14	4.99	15.67	5.26	8.72	5.62
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-3	**3%LKD + 9%FA**	1	**152.55	**1.65	45.34	4.16	32.37	5.30	4.28	3.05	7.52	4.46
		2	139.10	3.36	57.34	4.26	30.58	5.82	14.17	4.44	4.82	2.86
		3	121.15	2.45	70.57	1.65	32.35	3.55	14.71	5.06	2.04	1.55
		Average	130.12	2.90	57.75	3.36	31.77	4.89	11.05	4.18	4.79	2.96
		Test Number	Cycle-0		Cycle-1		Cycle-3		Cycle-7		Cycle-12	
			Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %	Stress, psi	Strain, %
Soil-3	**15% FA**	1	69.76	3.86	46.06	2.25	10.43	3.63	5.03	5.56	3.99	4.55
		2	64.54	4.45	45.84	2.86	5.93	4.06	6.54	6.32	5.00	6.82
		3	69.67	2.35	38.66	3.65	11.50	5.31	5.50	6.56	5.98	6.32
		Average	67.99	3.55	43.52	2.92	9.29	4.33	5.69	6.15	4.99	5.90

** Discarded Value

Table A23: Chemical Composition of Fly Ash

Analyte	Weight %
SiO_2	37.86
Al_2O_3	18.07
Fe_2O_3	5.77
CaO	21.51
MgO	4.36
SO_3	2.65
Na_2O	1.33
K_2O	0.87
TiO_2	1.21
P_2O_5	1.14
Mn_2O_3	0.03
SrO	0.29
Cr_2O_3	0.02
ZnO	0.02
BaO	0.68
L.O.I. (950°C)[2]	4.02
Total	99.84

APPENDIX B

CONSTRUCTION SPECIFICATIONS FROM DIFFERENT AGENCIES

APPENDIX B 1

CONSTRUCTION SPECIFICATIONS FROM MICHIGAN DEPARTMENT OF TRANSPORTATION

MICHIGAN
DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION
FOR
LIME STABILIZED SUBGRADE

a. Description. This work consists of all material including water, equipment, labor and testing for constructing a 12 inch compacted uniform layer of lime stabilized subgrade and determining the minimum amount of lime, or lime and fly ash combination required for the soil. The work shall be performed in accordance with this specification, 2003 Standard Specifications for Construction, as directed by the Engineer, and shall conform to the lines, grades, notes, and typical sections shown on the plans. **For the bidding purpose only, minimum rate of quicklime application is 5% on a dry weight basis of the soil. Fly Ash may or may not be required as determined by the Contractor Design Tests. For bidding purpose only, the estimated quantity of Fly Ash is 1,200 tons.**

b. Materials.

Lime. Lime shall be quicklime conforming to the requirements of ASTM C 977 specifications with the modification that all quicklime shall pass the 3/8 inch size sieve. A lime shall be certified by "Test Data Certification" method as per the MDOT Material Source Guide and shall represent each lot of lime delivered on the project.

Fly ash. Fly ash shall conform to ASTM C 618 for class F. Bulk fly ash may be transported dry in bulk trucks and stored in tanks or may be transported in dampened condition (15 percent moisture, maximum). The fly ash class F must be selected from current MDOT Material Source Guide Approved Manufacturer's list. A proper documentation must be supplied from the manufacturer to meet the ASTM C 618 requirements.

Water. Water for mixing and curing shall meet the requirements of subsection 911.02 of the 2003 Standard Specifications for Construction.

Soil. Soil for the lime stabilization as used in this specification is the in-place subgrade soil material. The soil shall be uniform in quality and gradation, be free of roots, sod, weeds, and stones larger than 2-1/2 inches, and shall be approved by the Engineer.

c. **Contractor Designed Lime or lime-fly ash, and Soil Mix.** The Contractor shall develop and submit, for approval, a mix design specifying percent of lime, or lime and fly ash in the soil to be stabilized. The Contractor's qualified representative or geotechnical engineer shall collect representative soil samples that are evenly distributed along the project length under the direction of the Engineer. Take one sample for every 20,000 square yard area of soil to be treated, one per major type of soil, or a minimum of 5 samples per project, whichever is greater, and submit to an AASHTO or ASTM accredited geotechnical laboratory to determine the recommended percentage of lime, or lime and fly ash for each soil sample taken. The station, offset and the depth of these soil borings shall be recorded and submitted to the Engineer. Prior to sampling, the Contractor shall submit the sampling location plan to the Engineer for review.

The AASHTO or ASTM accredited geotechnical laboratory shall perform the following tests and services for the untreated soil and lime-treated soil. Samples must be prepared with the same stabilizing material(s) that will be supplied to the job.

1. Soil Classification per AASHTO M145 and ASTM D2487 for the untreated soil and lime-treated soil.
2. Moisture and density testing per AASHTO T99 for the untreated soil and lime-treated soil.
3. California Bearing Ratio (CBR) laboratory test of uncured soil-lime mixture as per ASTM D 1883.
4. Perform Liquid, Plastic, and Plasticity Index of soil samples as per ASTM D 4318
5. Perform unconfined compressive strength test as per ASTM 5102. Prepare sample cylinders according to ASTM, Method B with the following modification: Revise Section 12 to read: Cure compacted specimens in a plastic air tight moisture proof container at 40 degrees Celsius temperature for 7 days.
6. Determine the minimum amount of lime, or lime and fly ash using each sample of untreated soil as per ASTM D 6276 that results in a soil-lime pH of 12.4, CBR of 10 for uncured soil-lime mixture, and a minimum unconfined compressive strength of 125 psi. Note: Fly Ash is not required if lime alone can meet these pH, CBR, and unconfined compressive strength requirements.
7. Submit copies of test reports from the geotechnical lab with all of the data to the Engineer for review and approval a minimum of 10 days prior to the commencement of the test strip construction.

Upon the Department's acceptance of the lime or lime-fly ash combination percentages, the contractor shall make moisture density curves for the chosen percentages of lime, or lime-and-fly ash combination and soil mix according to AASHTO T 99 for each soil sample taken above. Thoroughly mix the lime or lime-fly ash combination with the soil, and allow the mixtures to mellow for at least 24 hours before making the curves. Plot the wet and dry weight on a graph. Submit this data to the Engineer a minimum of 10 working days before the work begins. Engineer will use these curves or MDOT Typical Density Curves for compaction acceptance.

d. Equipment, Machines, and Tools. The equipment, machines, and tools used in the work shall be subject to approval and shall be maintained in satisfactory condition at all times. Other compacting equipment may be used in lieu of that specified where it could be demonstrated that the results are equivalent. Protective equipment, apparel, and barriers shall be provided to protect the eyes, respiratory system, and the skin of workers who are exposed to lime or lime dust.

1. <u>Sheep-foot or Vibratory pad foot roller</u>. Self propelled type with a minimum weight of 15 tons or greater as needed for compaction.

2. <u>Steel-Wheeled Smooth Rollers</u>. Steel-Wheeled rollers shall be the self-propelled type with a total weight of not less than 10 tons, and a minimum weight of 300 pounds per inch width of rear wheel. Wheels of the rollers shall be equipped with adjustable scrapers. The use of vibratory rollers is optional.

3. <u>Pneumatic-Tired Rollers</u>. Pneumatic-tired rollers shall be self-propelled and weigh when ballasted at least 8 tons but not more than 30 tons. It shall be equipped with a

minimum of 7 wheels situated on axles in such a way that the rear group of tires will not follow in the tracks of forward group of tires.

4. Mechanical Spreader. Mechanical spreader shall be cyclone, screw-type box, pressure manifold, or other approved equipment. A motor grader shall not be used to spread lime.

5. Watering Equipment. Watering equipment shall consist of tank trucks fitted with pressure distributors, or other approved equipment designed to apply controlled quantities of water uniformly over variable widths of surface without the truck adversely affecting the quality of the subgrade.

6. Tampers. Tampers shall be of an approved mechanical type, operated by either pneumatic pressure or internal combustion, and shall have sufficient weight and striking power to produce the compaction required.

7. Rotary Pulvamixer. A rotary pulvamixer shall be used for all mixing. Pulvamixer shall utilize a direct hydraulic drive and be capable of mixing the full 12 inch depth in one lift.

e. Construction.

1. General. Perform subgrade stabilization work when the air temperature is 40 degrees Fahrenheit or above and rising. Do not apply lime, or lime-fly ash combination to frosted subgrade under any circumstances. All work must be performed between April 1st and October 31st. **The depth of subgrade to be stabilized is 12 (twelve) inches**. Uniformly mix the approved proportion of the stabilizing materials through the entire 12-inch stabilized depth, and compact subgrade to the minimum 95% of required density. The Engineer will verify that a minimum of 12 inches of uniformly stabilized and compacted subgrade is achieved by digging 12-inch deep test holes at representative intervals. Adequate drainage shall be provided during the entire construction period to prevent water from collecting or standing on the area to be modified or on pulverized, mixed, or partially mixed material. Finished and completed lime stabilized subgrade shall conform to the lines, grades, and cross sections, and dimensions indicated in the plans.

2. Lime Stabilization Omission Locations: If during construction, the Engineer determines that certain locations have soils that are unsuitable for lime, or lime-and-fly ash stabilization, the Engineer may request for a modification of the lime stabilization procedure, or the use of other methods as necessary or cost effective. These soil types include, but not limited to granular soils and cohesive soils with excessive moisture content.

3. Contractor's Quality Control (QC) Plan: The contractor shall submit a QC plan, for the approval of the Engineer, a minimum of 5 days prior to starting the construction of the test strip. The QC plan shall include, but not limited to, name and description of the equipments to be used, personnel responsible for monitoring application raters, methods of determining and adjusting moisture contents.

4. Test Section. Upon the Engineer's approval of the Contractor's QC plan, a 600 linear feet test section comprising of either one or more lane width (depending upon construction staging) will be selected (with the approval of the Engineer) to implement the details of lime stabilization. The contractor shall submit a work plan for the test strip a minimum of 5 working days in advance of construction of the test strip. The work for this test section will be in accordance with this special provision. The Contractor can proceed with the stabilization of roadway subgrade if the test section meets the approval of the Engineer. At the Engineer's discretion, the test section may be accepted as part of the total required lime stabilized area.

5. Subgrade Preparation. Prior to adding the stabilizing materials, remove all deleterious materials such as topsoil, roots, organic material, and rock fragments larger than 2-1/2 inches. The subgrade treatment area shall be graded to conform to the lines, grades, and cross sections shown in the plans prior to being processed for stabilization. All the deleterious material removed as part of subgrade preparation will be the property of the Contractor and shall be considered included in the payment for lime stabilized subgrade.

6. Lime or lime-and-fly ash Application. Apply the contractor designed lime, or lime-and-fly ash combination on a dry weight basis. Submit verification testing to show that the required application rate is utilized, and provide the results to the Engineer at the end of each workday. Spread the lime, or lime-and-fly ash combination uniformly on the scarified subgrade by means of distributors or equipment approved by the Engineer. Place a canvas shroud on the distribution bar and extend to the subgrade. Do not apply lime or lime-and-fly ash when the wind conditions are such that blowing material would become objectionable to the adjacent property owners or create potential hazards to traffic. In order to enhance dust control, the Contractor may use moisture-conditioned fly ash (if the fly ash is determined necessary as per the Contractor design mix). Lime and fly ash can be spread as individual components.

7. Spreading. The spreading of stabilizing material shall be limited to an area that can be incorporated and mixed, within the same working day. While spreading lime, or lime-and-fly ash combination, minimize dusting and impact to the traffic by periodic water sprinkling at no cost to the Department.

8. Mixing.
   Initial Mixing: Immediately after the lime or lime-and-fly ash combination has been spread, thoroughly mix the lime or lime-and-fly ash into the subgrade by using an approved rotary mixer to a depth of 12 inches. Add enough water to raise the moisture content of the soil mixture to 3% to 5% above the optimum moisture content. Continue mixing until lime, or lime-and-fly ash combination has been uniformly incorporated into the subgrade to the required depth with the mixture being homogeneous and friable. Complete this initial mixing within 4 hours of spreading the lime. A waiting period after initial mixing of 1 to 24 hours or longer may be required or necessary for all lime particles to hydrate.

   Final Mixing: After the waiting period, soil shall be remixed, adding water as needed to raise the moisture content to 2 to 3% above optimum. Continue mixing until the quicklime has been uniformly incorporated into the subgrade to the required depth and with soil clods broken down to pass a 2-inch screen and at least 60% passing No. 4 sieve, exclusive of rock particles. Final mixing shall be accomplished within 5 days of initial mixing. There shall be no unhydrated lime pebbles present before compaction operations start. The Engineer may verify that any visible particles are not unhydrated lime before compaction begins.

It is the contractor's responsibility to determine the in-situ moisture content of the soil or soil-lime mixture in order to determine the quantity of water required to raise the moisture content to the required level above the optimum moisture content.

The Engineer may run the field gradation testing to determine the adequacy of mixing. In order to determine the adequacy of the mixing, two control sieves, 2-inch and No. 4, shall be used. All of soil clods during the mixing must pass 2-inch sieve and at least 60% pass a No. 4 sieve, exclusive of rock particles.

9. Compaction. Begin compaction immediately after final mixing. Add water or aerate the subgrade to bring the soil-lime mixture to optimum moisture content, plus or minus 2%. Continue final compaction until the stabilized subgrade has a density of not less than 95% of maximum density established as above for the soil-lime, or soil-lime-and-fly ash combination mixture. Rolling shall begin at the outside edge of the surface and proceed to the center, overlapping on successive trips at least one-half width of the roller, or as determined by the Engineer based upon construction staging. At all times, the speed of the roller shall not cause displacement of the mixture to occur. Areas inaccessible to the rollers shall be compacted with mechanical tampers, and shall be shaped and finished by hand methods. Final compaction shall be done with steel wheel smooth drum rollers. **The Engineer will perform the density and moisture testing for the compacted subgrade for acceptance as per this special provision and 2003 Standard Specifications for Construction.**

10. Curing and protection. Immediately after the stabilized subgrade has been compacted and finished as specified above, the surface shall be protected against rapid drying, for 7 days by periodic sprinkling and shall be kept moist for 7-day curing period, unless covered by subsequent layers of pavement section (sand subbase or aggregate base). Other suitable method of curing the compacted lime-soil mix may be approved the Engineer at his/her discretion.

11. Re-stabilization. If an approved stabilized area shows failure, tenderness or damage after curing, the Engineer shall require re-stabilization to be performed, where appropriate at no additional cost to the Department.

f. Use of Moisture Conditioned Fly Ash. The use of moisture-conditioned fly ash for lime-and-fly ash combination of soil treatment is acceptable (only Class F fly ash is permitted). Moisture conditioned fly ash shall contain no more than 15% moisture by dry weight of fly ash. When moisture-conditioned fly ash is used, the lime and fly ash shall be spread in two separate applications and the following additional construction procedures shall apply:

The lime shall be added to the subgrade and mixed in accordance with the above mixing subsection 8 of section "e". After the lime is thoroughly mixed, the subgrade shall be compacted with a steel wheel roller to achieve the surface strength and smoothness required to spread the moisture-conditioned fly ash.

Within 72 hours of mixing the lime and soil, the moisture-conditioned ash shall be uniformly spread onto the lime treated soil to provide the equivalent dry weight basis content of fly ash as determined by the contractor designed mix. The soil shall be remixed to blend the moisture-conditioned fly ash homogeneously with the lime treated soil.

g. Construction Traffic. Completed portion of lime stabilized subgrade may be opened immediately to light construction traffic at the contractor's own risk and option, provided the curing is not impaired. After 7-day curing period has elapsed, completed areas may be opened to construction traffic. Placement of subsequent pavement layers may begin the day following completion of lime stabilization, provided the lime-stabilized completed area has strengthened sufficiently to prevent marring or distorting of the surface by equipment or traffic. Lime and water may be hauled over the completed area with pneumatic-tired equipment if approved by the Engineer. Finished portions of lime-modified subgrade that are traveled on by the equipment used in construction of adjoining section shall be protected in a manner to prevent equipment from marring and damaging the completed work. The Contractor is responsible for correcting and restabilizing the damaged areas at his/her cost.

h. Field Quality Control and Assurance. Results of field quality control testing shall verify that the materials comply with this special provision and 2003 Standard Specification for Construction. When a material source is changed, the new material shall be tested for compliance. When deficiencies are found, the initial analysis shall be repeated and the material already placed shall be retested to determine the extent of unacceptable material. All in-place unacceptable material shall be replaced or repaired, as directed by the Engineer at no additional cost to the Department.

Completed **thickness** of the lime stabilized subgrade soil layer shall be within ½ inch of specified thickness of 12 inches. When the measured thickness of the lime stabilized subgrade soil is more than ½ inch deficient, such areas shall be corrected by scarifying, adding additional lime, remixing and recompacting as directed by the Engineer. Where the measured thickness of the lime stabilized subgrade layer is more than ½ inch thicker than required, it shall be considered conforming to the specified thickness requirement, provided the elevation of finished subgrade is within the tolerance as per the 2003 Standard Specification for Construction. Thickness of lime stabilized subgrade layer shall be measured for each 4000 square yards, at least one per day, or as determined by the Engineer. Measurements shall be made in 3 inch diameter or larger test holes penetrating the lime stabilized subgrade.

Lime content of uncured lime, or lime-and-fly ash and soil mixture shall be determined in accordance with ASTM D 3155. Sample for testing shall be obtained from mid-depth of the lime stabilized subgrade layer. Sampling and testing shall be conducted at the rate of one test per 4000 square yards, at least one per day, or as determined by the Engineer. Lime content shall not be less than 1% below the Contractor designed lime-soil mix design.

At least one **field density test** shall be performed for each 4,000 square yards of lime stabilized subgrade, but at least one per day.

i. Contractor Warranty and Maintenance. Perform the following work at no cost to the Department. Repeat this work as often as necessary to keep the lime stabilized subgrade intact.

1. Maintain the lime or lime-fly ash stabilized subgrade in good condition until the work is completed and accepted.

2. Maintain a smooth surface of the lime stabilized subgrade by blading.

3. Immediately repair any defects that occur.

j. Method of Measurement. Actual area of the Lime Stabilized Subgrade as ordered and completed to the 12 inch thickness and cross sections shown on the plans, and accepted, will be measured in square yards. All calculations of areas measured for payment shall be based on measurements made to the nearest 0.1 yard with area calculated to the nearest square yard. The length will be measured along the surface of the completed roadbed at its centerline. The width will be the top surface width of the completed roadbed specified on the plans, measured perpendicular to the center line of roadbed. Additional areas required for tapers, etc shall be measured by length and width along the surface area stabilized.

Lime and fly-ash actually incorporated in the work will be measured by the ton. A certified delivery tickets shall be furnished to the Engineer for lime and fly-ash used in the construction of lime stabilized subgrade.

k. Basis of Payment. The completed work as described shall be paid for at the contract unit price for the following contract items (pay items):

Contract Item (Pay Item)	**Pay Unit**
Lime Stabilized Subgrade	Square Yard
Lime	Ton
Fly ash	Ton

The ordered and accepted area of **Lime Stabilized Subgrade,** measured as noted above, will be paid for at the contract unit price bid per square yard. Said unit price bid shall be full compensation for all the sampling, design of lime or lime-fly ash soil mix, scarifying, pulverizing, mixing, shaping, water, curing, compacting, and application of lime, testing; and for all equipment, tools, labor, and incidentals needed for completion of the work.

The accepted quantity of **lime** and **fly ash** actually incorporated in the work except as noted herein, measured as provided above, will be paid for at the contract unit price per ton of lime, or fly ash, which price shall be payment in full for furnishing, transporting, storing, handling, and spreading; and for all equipment, tools, labor, and incidentals needed for completion of the work.

MICHIGAN
DEPARTMENT OF TRANSPORTATION

SPECIAL PROVISION
FOR
CHEMICALLY STABILIZED SUBGRADE

a. Description. This work consists of providing all labor, equipment, materials, testing and determining the optimum amount of chemical required to construct a 12 inch compacted uniform stabilized subgrade layer. The work must be performed in accordance with this special provision, as detailed on the plans, the standard specifications and as directed by the Engineer.

b. Materials. Lime Kiln Dust (LKD) and Cement Kiln Dust (CKD) are the only chemical stabilizers acceptable for use on this project.

CKD and LKD must conform to the requirements of ASTM D 5050. All CKD and LKD must be certified by Test Data Certification method according to the *Materials Quality Assurance Procedures Manual*. CKD must be tested under the appropriate sections of ASTM C 25 and AASHTO T 105 to determine the total alkalis (K_2O+Na_2O) and total sulfates (SO_3). Test data must be within the following limits:

Property	Limit, % maximum
Total alkalis (K_2O+Na_2O)	10
Total sulfates (SO_3)	15

Water for dust control, mixing and curing must be according to section 911 of the Standard Specifications for Construction.

Soil for stabilization as specified herein is the in-place subgrade soil. The soil must be visually free of deleterious materials such as topsoil, roots, organic material and rock fragments larger than 2½ inches, and must be approved by the Engineer prior to treatment.

c. Contractor Mix Design for Chemically Stabilized Soils. Develop and submit, for approval, a mix design specifying percent of chemical stabilizer (LKD or CKD) in the soil to be stabilized. The Engineer reserves the right to reject the selected chemical stabilizer and request a new mix design with a different chemical stabilizer at no cost to the Department based on the mix-design results. One chemical stabilizer from the same source must be used on this project, unless otherwise approved by the Engineer.

1. Untreated Soil Characteristics and Properties. The Contractor’s qualified representative or geotechnical engineer must collect representative soil samples that are evenly distributed along the project length at the direction of the Engineer. Take one sample for every 20,000 square yard area of soil to be treated, one per major type of soil, or a minimum of 5 samples per project, whichever is greater. The station, elevation, offset and depth of these soil samples must be recorded and submitted to the Engineer. Prior to sampling, the Contractor must submit the sampling location plan to the Engineer for review and approval.

An AASHTO or ASTM accredited geotechnical laboratory must determine the following for the untreated soil samples:

A. Soil Classification according to AASHTO M 145 and ASTM D 2487.

B. Moisture and density testing according to AASHTO T 99.

C. Liquid Limit, Plastic Limit and Plasticity Index according to ASTM D 4318.

Submit copies of test reports from the geotechnical laboratory with all pertinent data to the Engineer for review and approval. The Engineer is permitted up to 10 days to review this information.

2. Mix Design Procedure.

A. Moisture and Density Testing. Perform moisture and density testing according to AASHTO T 99. Prepare four mixtures of soil treated with the CKD or LKD percentages in the soil samples initially at 3 percent, 6 percent, 9 percent and 12 percent for each soil sample. Prepare the mixtures according to ASTM D 558. Alternate percentages may be allowed as directed by the Engineer.

B. California Bearing Ratio (CBR) Laboratory Testing. Perform CBR testing for uncured treated soil mixtures according to ASTM D 1883.

C. Unconfined Compression Strength (UCS) Test Specimens. P repare four mixtures of treated soil with the CKD or LKD percentages in the soil samples initially at 3 percent, 6 percent, 9 percent and 12 percent for each soil sample. Prepare sample cylinders according to ASTM D 1633 Method A.

D. Curing. Cure compacted specimens in an air tight, moisture proof container at 70 degrees F (21 degrees C) for 7 days.

3. UCS of the Cured Specimens. Determine, calculate and r eport UCS of each specimen in accordance with ASTM D 1633 Method A.

4. Minimum Chemical Mixture Content for Soil Stabilization. Recommend the minimum chemical mixture content (LKD or CKD) that results in a CBR of 10 percent for uncured treated soil mixtures, and a minimum unconfined compressive strength of 125 psi for cured specimens. Add 1 percent to this percentage for application in the field.

A. Upon the Department's approval of the chemical percentages, the Contractor must make moisture density curves for the chosen percentages of chemical and soil mix according to AASHTO T 99 for each soil sample taken above.

B. Thoroughly mix the chemical with the soil and immediately make the mixtures for moisture and density testing.

C. Plot the wet and dr y weight on a gr aph. Submit this data to the Engineer for approval a minimum of 10 working days before the work begins. The Engineer will use these curves for compaction acceptance.

d. Equipment. The equipment used to conduct the work is subject to approval by the Engineer and must be maintained in satisfactory condition at all times. Other compaction equipment may be used in lieu of that specified where it can be demonstrated that the results are equivalent. Protective equipment, apparel and barriers must be provided to protect eyes, respiratory system and skin of the workers who are exposed to chemical stabilizer.

1. Sheepsfoot or Vibratory Pad Foot Roller. Self propelled type with a minimum weight of 15 tons or greater as needed for compaction.

2. Steel-Wheeled Smooth Drum Rollers. Steel-wheeled smooth drum rollers must be self-propelled with a total weight of at least 10 tons, and a minimum weight of 300 pounds per inch width of rear wheel. The wheels of the rollers must be equipped with adjustable scrapers. The use of vibratory rollers is optional.

3. Pneumatic-Tired Rollers. Pneumatic-tired rollers must be self-propelled and weigh when ballasted at least 8 tons but not more than 30 tons. The roller must be equipped with a minimum of 7 tires situated on two axles such that the rear tires will not follow in the tracks of the forward tires.

4. Mechanical Spreader. Mechanical spreader must be a cyclone, screw-type box, pressure manifold or other approved type. A motor grader must not be used to spread the chemical.

5. Watering Equipment. Watering equipment must consist of tank trucks fitted with pressure distributors, or other approved equipment designed to apply controlled quantities of water uniformly over variable widths of surface without the truck adversely affecting the stability of the subgrade.

6. Tampers. Tampers must be of an approved mechanical type, operated by either pneumatic pressure or internal combustion and must have sufficient weight and striking power to produce the required compaction.

7. Rotary Pulvimixer. A rotary pulvimixer must be used for all mixing and must utilize a direct hydraulic drive and be capable of mixing the full 12 inch depth in one lift.

e. Construction.

1. General. Perform subgrade stabilization work when the air temperature is 40 degrees Fahrenheit or above and rising. Do not apply chemical to frozen subgrade under any circumstances. All work must be performed between April 1st and October 31st. The depth of subgrade to be stabilized is 12 inches. Uniformly mix the approved proportion of the stabilizing material through the entire 12-inch depth to be stabilized and compact subgrade to at least 95 percent of the maximum unit weight. Adequate drainage must be provided during the entire construction period to prevent water from collecting or standing on the area to be modified, or on pulverized, mixed, or partially mixed material. Finished and completed stabilized subgrade must conform to the lines, grades, and cross sections as indicated on the plans.

2. Chemical Stabilization Omission/Modification Locations. If during construction the Engineer determines that certain locations are inappropriate for chemical stabilization, the treatment may be omitted or the Engineer may request a modified stabilization procedure.

3. If the Engineer modifies the stabilization procedure to stabilize to a depth greater than 12 inches, those modified locations will be paid for 1.5 times the unit bid price.

4. Contractor's Quality Control (QC) Plan. The Contractor must submit a QC plan, for approval by the Engineer, a minimum of 10 days prior to starting construction of the test strip. The QC plan must include, but not be limited to, name and description of the equipment to be used, personnel responsible for monitoring application rates, methods of determining and adjusting moisture content.

5. Test Section. Upon the Engineer's approval of the Contractor's QC plan, a 600 foot long test section a minimum of one lane width will be selected to implement the chemical stabilization work. The Contractor must submit a work plan for the test strip a minimum of 10 working days in advance of construction of the test strip. The work for this test section will be in accordance with this special provision. The Contractor can proceed with the stabilization of roadway subgrade if the test section meets the approval of the Engineer. At the Engineer's discretion, the test section may be accepted as part of the total required stabilized area.

6. Subgrade Preparation. Prior to adding the stabilizing materials, remove and dispose of all deleterious materials such as topsoil, roots, organic material and rock fragments larger than 2½ inches. The subgrade treatment area must be graded to conform to the lines, grades, and cross sections shown on the plans prior to being processed for stabilization.

7. Chemical Application. Apply the chemical combination as approved by the Engineer on a dry weight basis. Submit verification testing to show that the required application rate is utilized, and provide the results to the Engineer at the end of each workday. The Contractor will conduct a rate application test in the field to demonstrate the chemical is being applied at the prescribed rate. The test will incorporate a receptacle made of metal, plastic, canvas or similar material of known area and volume. The spreader will pass over the receptacle and spread the chemical at the anticipated rate. It will be weighed in the field and the actual application rate will be determined. Spread the chemical uniformly on a scarified subgrade by means of distributors or equipment approved by the Engineer. Place a canvas shroud on the distribution bar and extend to the subgrade to minimize dust. Do not apply chemical when the wind conditions are such that blowing material would become objectionable to the adjacent property owners or create potential hazards to traffic.

8. Spreading. While spreading chemical, minimize dusting and impact to traffic by periodic water sprinkling at no cost to the Department. The spreading of stabilizing material must be limited to an area that can be incorporated and mixed, within 1 hour of application.

9. Mixing. Immediately after the chemical has been spread, mix into the subgrade soil using a rotary pulvimixer to a depth determined by the Engineer. Add enough water to raise the moisture content of the soil mixture within the range of 1 percent below to 2 percent above the optimum moisture content. Continue mixing until the chemical has been uniformly incorporated into the subgrade to the required depth with the mixture being homogenous and friable.

It is the Contractor's responsibility to determine the in situ moisture content of the soil or soil-chemical mixture in order to determine the quantity of water required to raise the moisture content to the required level above the optimum moisture content.

The Engineer may run the field gradation testing to determine the adequacy of mixing. In order to determine the adequacy of the mixing, two control sieves, 1 inch and No. 4, will be used. All soil clods must pass the 1 inch sieve and at least 60 percent must pass the No. 4 sieve, exclusive of rock particles. Mixing must continue until the required gradation is achieved.

10. Compaction. After mixing, shape the subgrade. Start compaction within 1 hour after the final mixing. Add water or aerate the subgrade to bring the soil-chemical mixture to optimum moisture content, plus or minus 2 percent. Continue final compaction until the stabilized subgrade has a density of at least 95 percent of maximum unit weight established as above for the soil-chemical mixture. Rolling must begin at the outside edge of the surface and proceed to the center, overlapping on successive trips at least one half width of the roller, or as determined by the Engineer. At all times, the speed of the roller must not cause displacement of the mixture to occur. Areas inaccessible to the rollers must be compacted with mechanical tampers and must be shaped and finished by hand methods. Final compaction must be done with steel wheel smooth drum rollers. The Engineer will perform the density, moisture and DCP testing for the compacted subgrade for acceptance as per this special provision.

Complete the mixing, compacting, shaping and fine grading within 3 hours from start to finish.

11. Curing and Protection. Immediately following the final grading, cure the compacted subgrade for a minimum of 24 hours before placement of the overlying course. The surface must be protected from rapid drying during this period by periodic sprinkling unless covered by subsequent layers of pavement section. Other suitable methods of curing the compacted stabilized subgrade may be approved by the Engineer. The Engineer may modify the amount of time required for curing based on site conditions. Protect the stabilized subgrade from disturbance. Do not operate construction equipment on the treated soil during the curing period. Do not allow the treated soil to freeze during the cure period.

12. Re-stabilization. If an approved stabilized area shows failure, rutting or damage after curing, re-stabilization must be performed at no additional cost to the Department.

g. Construction Traffic. The completed portions of stabilized subgrade may be opened immediately to light construction traffic at the Contractor's own risk and option, provided the curing is not impaired. After the curing period has elapsed, completed areas may be opened to construction traffic. Placement of subsequent pavement section layers may begin the day following completion of subgrade stabilization provided the stabilized area has strengthened sufficiently to prevent marring or distorting of the surface by equipment or traffic. Chemical and water may be hauled over the completed area with pneumatic-tired equipment if approved by the Engineer. Finished portions of stabilized subgrade that are traveled on by the equipment used during construction of adjoining sections must be protected in a manner to prevent marring and damaging the completed work. The Contractor is responsible for correcting and re-stabilizing the damaged areas at no cost to the Department.

h. Field Quality Control and Acceptance Testing. Results of field quality control must verify that the materials comply with this special provision and the standard specifications. All in-place unacceptable material must be replaced or repaired, as directed by the Engineer at no additional cost to the Department.

The Engineer will use a Dynamic Cone Penetrometer (DCP) at representative intervals to verify that a minimum of 12 inches of uniformly stabilized and compacted subgrade has been achieved.

The thickness of the stabilized subgrade layer must be within ½ inch of the specified thickness of 12 inches. When the measured thickness of the stabilized subgrade soil is more than ½ inch deficient, such areas must be corrected by scarifying, adding additional chemical, remixing and re-compacting as directed by the Engineer with no additional cost to the Department. Where the measured thickness of the stabilized subgrade layer is more than 12 inches, it is acceptable, provided the elevation of finished subgrade is within the tolerance according to the standard specifications.

Stabilized thickness and field stabilized subgrade stiffness must be evaluated in accordance with ASTM D 6951. Stabilized subgrade thickness and stiffness is measured by plotting cumulative penetration blows versus depth. A change of slope on this graph will indicate the stabilized thickness. Average CBR for the stabilized layer is calculated in accordance with ASTM D 6951. A minimum average CBR of 10 percent in the stabilized zone is required for acceptance. Areas where the average CBR is less than 10 percent must be corrected by scarifying, adding additional chemical, remixing and re-compacting as directed by the Engineer. When the average CBR is less than 10 percent, the Engineer will verify the chemical application rate to determine whether the Contractor is following the specification and mix design appropriately. If the Engineer determines that the Contractor has not followed the mix design and the specification, all corrections must be completed with no additional cost to the Department.

At least one field density test must be performed for each 4000 square yards of stabilized subgrade or at least one per day.

i. Contractor Warranty and Maintenance. Perform the following work at no cost to the Department. Repeat this work as often as necessary to keep the stabilized subgrade intact.

1. Maintain the stabilized subgrade in good condition until the work is completed and accepted.

2. Maintain a smooth drainable stabilized subgrade surface.

3. Immediately repair any defects that occur.

j. Measurement and Payment. The completed work, as described, will be measured and paid for at the contract unit price using the following pay items:

Pay Item	**Pay Unit**
Chemically Stabilized Subgrade	Square Yard
Chemical Stabilizer	Ton

The area of stabilized subgrade completed to the 12 inch thickness and cross sections shown on the plans, and accepted, will be measured in square yards. All calculations of area measured for payment must be based on measurements made to the nearest 0.1 yard with area calculated to the nearest square yard. The length will be measured along the surface of the completed roadbed at centerline. The width will be the top surface width of the completed roadbed specified on the plans, measured perpendicular to the center line of roadbed. Additional areas required for tapers, etc. must be measured by length and width along the surface area stabilized.

Chemically Stabilized Subgrade, measured as noted above, will be paid for at the contract unit price bid per square yard and includes full compensation for all sampling, mix design, scarifying, pulverizing, mixing, shaping, water, curing, compacting, application of stabilizer, and testing; and for all equipment, tools, labor, and incidentals needed for completion of the work as described herein.

Chemical Stabilizer measured as noted above will be paid for at the contract unit price bid per ton and includes full compensation for furnishing, transporting, storing, handling, and spreading; and for all equipment, tools, labor, and incidentals needed for completion of the work as described herein. Only chemical stabilizer actually incorporated into the work will be included in the pay item. Additional compensation will not be made for excess waste or otherwise unused chemical stabilizer.

APPENDIX B 2

CONSTRUCTION SPECIFICATIONS FROM OHIO DEPARTMENT OF TRANSPORTATION

STATE OF OHIO
DEPARTMENT OF TRANSPORTATION

SUPPLEMENT 1120
MIXTURE DESIGN FOR CHEMICALLY STABILIZED SOILS

July 18, 2014

1120.01 Description. This work consists of sampling and testing soils mixed with cement, lime, or lime kiln dust to determine the optimum mix design. This supplement can be used in design to compare alternative mixes, and in construction to determine the optimum spreading percentage rate.

1120.02 Testing Laboratory. Use an accredited Geotechnical Testing Laboratory with a qualified staff experienced in testing and designing chemical stabilization and capable of performing the tests listed in the tables below. The staff must be under the supervision of a Professional Engineer with at least five years of geotechnical engineering experience. The Geotechnical Testing Laboratory must be currently accredited by either of the following:

AASHTO Materials Reference Laboratory (AMRL)
National Institute of Standards and Technology
100 Bureau Drive, Stop 8619
Building 202, Room 211
Gaithersburg, Maryland 20899-8619
(301)-975-5450 www.amrl.net

American Association of Laboratory Accreditation (A2LA)
5301 Buckeystown Pike, Suite 350
Frederick, Maryland 21704
(301)-644-3248 www.A2LA.org

The Geotechnical Testing Laboratory minimum accreditations required are a general laboratory inspection and the following AASHTO or ASTM designation tests:

TABLE 1120.02-1

Test Method	AASHTO Designation	ASTM Designation
Dry Preparation of Soil Samples	T 87	D 421
Particle Size Analysis of Soils	T 88	D 422
Determining the Liquid Limit of Soils	T 89	D 4318
Determining the Plastic Limit and Plasticity Index of Soils	T 90	D 4318
Moisture-Density Relations of Soils (Standard Proctor)	T 99	D 698
Specific Gravity of Soils	T 100	D 854
Unconfined Compressive Strength of Cohesive Soil	T 208	D 2166
Laboratory Determination of Moisture Content of Soils	T 265	D 2216

Ensure the Geotechnical Testing Laboratory is also proficient in the following tests:

ABLE 1120.02-2

Test Method	AASHTO Designation	ASTM Designation	Other Test Method
Family of Curves – One Point Method	T 272	–	–
Classification of Soils (as modified by the Department Specifications for Geotechnical Explorations)	M 145	–	–
Organic Content by Loss on Ignition	T 267	D 2974	–
Determining Sulfate Content in Soils – Colorimetric Method	–	–	TEX-145-E [1]
Moisture-Density Relations of Soil-Cement Mixtures	–	D 558	–
Wetting and Drying Compacted Soil-Cement Mixtures	–	D 559	–
Making and Curing Soil-Cement Compression and Flexure Test Specimens in the Laboratory	–	D 1632	–
Compressive Strength of Molded Soil-Cement Cylinders	–	D 1633	–
Laboratory Preparation of Soil-Lime Mixtures Using a Mechanical Mixer	–	D 3551	–
One Dimensional Expansion, Shrinkage, and Uplift Pressure of Soil-Lime Mixtures	–	D 3877	–
Unconfined Compressive Strength of Compacted Soil-Lime Mixtures	–	D 5102	–
Using pH to Estimate the Soil-Lime Proportion Requirement for Soil Stabilization	–	D 6276	–

[1] Texas Department of Transportation (Feb. 2005) ftp.dot.state.tx.us/pub/txdot-info/cst/TMS/100-E_series/pdfs/soi145.pdf

1120.03 Sampling and Testing of Untreated Soil. Collect one soil sample for every 5000 square yards (4000 m^2) of treated subgrade area or 2000 cubic yards (1500 m^3) of treated embankment, but not less than a total of four soil samples for a project. Each sample consists of 75 pounds (35 kg) of soil (about a five gallon bucket). Record the station, offset, geographic coordinates (Latitude and Longitude as decimal degree to six decimal places), and elevation of each sample location.

When this supplement is used during construction for stabilizing embankment (Item 205), collect samples from locations and elevations that represent the soils that will be chemically treated. When this supplement is used during construction for stabilizing subgrade (Item 206), collect samples of in-place soil at the proposed subgrade elevation. However, if the chemical stabilization will be performed on embankment fill, collect the soil samples from the source or sources of the embankment material that will be stabilized. Collect each sample from a different location. For in-place soil samples, collect the samples from locations distributed across the treated area. Obtain the Department's approval before collecting samples from outside the treated area.

When this supplement is used during the design phase, the geotechnical consultant shall submit a plan to modify the above sampling procedure to quantify the effects of chemical mixtures on the soil that will be stabilized.

Visually inspect each soil sample for the presence of gypsum ($CaSO_4 \cdot 2H_2O$). Gypsum crystals are soft (easily scratched by a knife; they will not scratch a copper penny), translucent (milky) to transparent, and do not have perfect cleavage (do not split into thin sheets). Photos of gypsum crystals are shown in Figures 1120-1 to 1120-4. If gypsum is present, immediately notify the Department.

Perform the following tests on each soil sample. Perform each test according to the test method shown and as modified by the Department Specifications for Geotechnical Exploration (Section 603.3). If more than one test method is shown for a test, use any of the given test methods to perform the test. If the sulfate content is greater than 3,000 parts per million (ppm), immediately notify the Department.

TABLE 1120.03-1 TESTS FOR UNTREATED SOIL

Test	**AASHTO Designation**	**ASTM Designation**	**Other Test Method**
Moisture content	T 265	D 2216	–
Particle-size analysis	T 88	D 422	–
Liquid limit	T 89	D 4318	–
Plastic limit and plasticity index	T 90	D 4318	–
Family of curves – one point method	T 272	–	–
Organic content by loss on ignition	T 267	D 2974	–
Sulfate content in soils – colorimetric method	–	–	TEX-145-E [1]

[1] Texas Department of Transportation (Feb. 2005) ftp.dot.state.tx.us/pub/txdot-info/cst/TMS/100-E_series/pdfs/soi145.pdf

Classify the soil sample according to the ODOT soil classification method described in the Department Specifications for Geotechnical Exploration (Section 603). Determine the optimum moisture content and maximum dry density of the soil using the one-point Proctor test and the Ohio typical moisture-density curves according to Supplement 1015.

Submit the soil classification and test results for each sample to the Department for review before continuing with the mixture design test procedure. Also submit to the Department for review and acceptance a recommendation as to how the soil samples will be combined or grouped for the remaining mixture design test procedures. Obtain written acceptance from the Department before continuing with the mixture design test procedure. Allow seven days for the review. During construction, submit the information to the Project Engineer, who will forward the submittal to the District Geotechnical Engineer, the Office of Geotechnical Engineering, and the Office of Construction Administration. During design, submit the information to the District Geotechnical Engineer.

1120.04 Mixture Design Test Procedure. Use the following procedure to prepare four mixtures from each soil sample that will be tested. From each mixture, prepare three specimens for testing. This results in a total of 12 test specimens for each soil sample.

Each mixture consists of soil mixed with varying amounts of the stabilization chemical, except for the first mixture which consists of the untreated soil. The percentage of stabilization chemical in each mixture is shown in the table below. Calculate the quantity of stabilization chemical to add to the mixture by multiplying the given percentage by the dry weight of the soil.

TABLE 1120.04-1 PERCENTAGE OF CHEMICAL FOR TRIAL MIXES

	Cement	Lime	Lime Kiln Dust
Mix 1 (Untreated soil)	–	–	–
Mix 2	3%	MLP	4%
Mix 3	5%	MLP + 2%	6%
Mix 4	7%	MLP + 4%	8%

MLP – Minimum Lime Percentage (1120.04.A)

Carefully store the cement, lime, or lime kiln dust until used so that it does not react with moisture or excess carbon dioxide. When this supplement is used during construction, use cement, lime, or lime kiln dust from the same source that will supply the chemical for soil stabilization.

A. Minimum Lime Percentage. If using lime for chemical stabilization, determine the minimum percentage of lime required for soil stabilization using ASTM D 6276 (also known as the “Eades-Grim” test). Determine the lowest percentage of lime that produces a pH of 12.4. Report this value as the Minimum Lime Percentage. ASTM D 6276 addresses special cases where the highest measured laboratory pH is less than 12.4. Notify the Department if the measured pH is less than 12.3 or if the Minimum Lime Percentage is greater than 8 percent.

Not all laboratory pH-measuring devices are capable of accurate calibration to determine pH levels above 12.0. Ensure the pH meter can accurately measure pH up to 14 and can be calibrated with a pH 12 buffer solution.

B. Optimum Moisture Content and Maximum Dry Density. Determine the optimum moisture content and maximum dry density of treated soil mixtures using the one-point Proctor test and the Ohio typical moisture-density curves according to Supplement 1015 (the optimum moisture content and maximum dry density of the untreated soil were determined in 1120.03 above.) Prepare the mixtures according to ASTM D 3551 if using lime, and according to ASTM D 558 if using cement or lime kiln dust.

Thoroughly mix the soil, stabilization chemical, and water until the chemical appears to be consistently blended throughout the soil. Use a laboratory or commercial-grade mixer, such as a Hobart mixer. Do not mix by hand.

If using lime for stabilization, seal the mixture in an airtight, moisture-proof bag or container, and store it at room temperature for 20 to 24 hours. This is called the "mellowing" period. Remove the soil-lime mixture from the sealed container and lightly remix it for one to two minutes before performing the one-point Proctor test. Cement and lime kiln dust do not require a "mellowing" period.

C. Unconfined Compressive Strength Specimens. Prepare three specimens for unconfined compressive strength (UCS) testing from each mixture shown in Table 1120.04-1. If using lime for stabilization, use ASTM D 5102, Procedure B. If using cement or lime kiln dust, use ASTM D 1633, Method A. Compact the specimens at the moisture content shown in Table 1120.04-2.

TABLE 1120.04-2 MOISTURE CONTENT FOR PREPARING UCS SPECIMENS

	Cement	Lime	Lime Kiln Dust
Mix 1 (Untreated soil)	OMC (*u*)	OMC (*u*)	OMC (*u*)
Mix 2	OMC (*2*)	OMC (*2*) + 2%	OMC (*2*) + 1%
Mix 3	OMC (*3*)	OMC (*3*) + 2%	OMC (*3*) + 1%
Mix 4	OMC (*4*)	OMC (*4*) + 2%	OMC (*4*) + 1%

OMC (*u*) – Optimum moisture content of untreated soil (determined in 1120.03)
OMC (*n*) – Optimum moisture content of Mix *n* (determined in 1120.04.B)

D. Curing. Immediately wrap each specimen with plastic wrap and store each specimen in a separate airtight, moisture-proof bag. If using lime for stabilization, store the specimens at 104 ºF (40 ºC). If using cement or lime kiln dust for stabilization, store the specimens at 70 ºF (21 ºC). Allow the specimens from the treated soil mixtures (mixes 2, 3, and 4) to cure undisturbed for seven days. Do not cure the untreated soil specimens for more than 24 hours before performing the strength tests on them.

E. Moisture Conditioning. After curing, moisture condition the specimens from the treated soil mixtures by capillary soaking before performing the unconfined compressive strength tests. Do the following:

1. Remove the specimens from the airtight bag and remove the plastic wrap.
2. Use a caliper or pi-tape to measure the height and diameter of the specimens. Record at least three height and diameter measurements each. Calculate the average height and diameter.
3. Wrap the specimens with a damp, absorptive fabric.

4. In a shallow tray, place each wrapped specimen on a porous stone.
5. Add water to the tray until the water level is near the top of the stone and in contact with the absorptive fabric, but not in direct contact with the specimen.
6. Allow the specimens to capillary soak for 24 hours (± 1 hour).
7. Remove and unwrap the specimens and proceed with expansion testing.

Do not moisture condition the untreated soil specimens.

F. Expansion Testing. After moisture conditioning the specimens from the treated soil mixtures, but before performing the strength tests, measure the height and diameter again. Record and average at least three height and diameter measurements for each specimen. Calculate the volume change from before to after moisture conditioning. Report this change as a percentage. Notify the Department if the volume change exceeds 1.5 percent. Further expansion testing may be required using ASTM D 3877. If further expansion testing is required, the Department will pay for it as Extra Work. Do not perform the expansion testing on the untreated soil specimens.

G. Unconfined Compressive Strength Testing. Determine the unconfined compressive strength of each specimen according to the following:

1. For untreated soil, use AASHTO T 208 or ASTM D 2166.
2. For lime, use ASTM D 5102, Procedure B.
3. For cement or lime kiln dust, use ASTM D 1633, Method A.

Calculate the average unconfined compressive strength for each mixture.

1120.05 Recommended Spreading Percentage Rate. Estimate the recommended spreading percentage rate using the following procedure.

A. Generate a graph that shows the average unconfined compressive strength for each mixture versus the percent of stabilization chemical in the mixture (include the strength for the untreated soil at zero percent). Include the results from all tested soil samples.

B. Determine the minimum percentage of chemical that results in an average 8-day unconfined compressive strength that meets the minimum strengths shown in the following table. Interpolate the minimum percentage between points on the graph. If the average strength for the mixture with the greatest percentage of stabilization chemical does not meet the minimum strengths, contact the Department.

TABLE 1120.05-1 MINIMUM UNCONFINED COMPRESSIVE STRENGTH

	UCS after 8 days	Increase over UCS of Mix 1 (untreated soil)
Cement	100 psi (0.7 MPa)	+50 psi (+0.35 MPa)
Lime	100 psi (0.7 MPa)	+50 psi (+0.35 MPa)
Lime Kiln Dust	100 psi (0.7 MPa)	+50 psi (+0.35 MPa)

C. Round the minimum percentage up to the nearest 0.5 percent.

D. Add 0.5 percent to the percentage.

E. The minimum recommended spreading rate shall be 4.0 percent.

The Department may adjust the recommended spreading percentage rate due to site specific conditions.

1120.06 Mixture Design Report. Submit a mixture design report to the Department for review that includes the following information:

A. For each soil sample, report the following:

1. Soil classification
2. Moisture content
3. Particle-size analysis
4. Liquid limit
5. Plastic limit and plasticity index
6. Sulfate content (ppm)
7. Sample location, i.e., station, offset, geographic coordinates, and elevation

B. For each specimen, report the following:

1. Height and diameter measurements and averages from before and after moisture conditioning
2. Calculated percent volume change (swell)
3. Unconfined compressive strength

C. For each mixture, report the following:

1. Percent of chemical in the mixture
2. Optimum moisture content
3. Maximum dry density
4. Average volume change (swell)
5. Average unconfined compressive strength

D. The graph of average strength versus the percent of stabilization chemical in the mixture.

E. The recommended spreading percentage rate for the stabilization chemical.

During construction, submit the report to the Project Engineer for review. Allow seven days for the review. The Project Engineer will forward the submittal to the District Geotechnical Engineer, the Office of Geotechnical Engineering, and the Office of Construction Administration. The Department will determine the spreading percentage rate based on the mixture design report and site specific conditions.

During design, submit the report to the District Geotechnical Engineer.

1120.07 Field Verification of the Mix Design. During construction, sample the treated soil after mixing but before compaction. Take three samples from random locations for every 15,000 cubic yards (11,500 cubic meters) of treated soil for Item 205 and for every 40,000 square yards (33,500 square meters) for Item 206. Prepare three test specimens in the field from each sample according to 1120.04.C above, except compact the specimens at the in-place moisture content. Immediately wrap each specimen with plastic wrap and store each specimen in a separate airtight, moisture-proof bag before transporting the specimens to the lab. Perform the procedures described in 1120.04.D through 1120.04.G.

Submit the measurements and test results for each set of field verification samples to the Project Engineer as they are completed. The Project Engineer will forward the submittal to the District Geotechnical Engineer, the Office of Geotechnical Engineering, and the Office of Construction Administration.

PHOTOS OF GYPSUM CRYSTALS

FIGURE 1120-1 Gypsum crystals

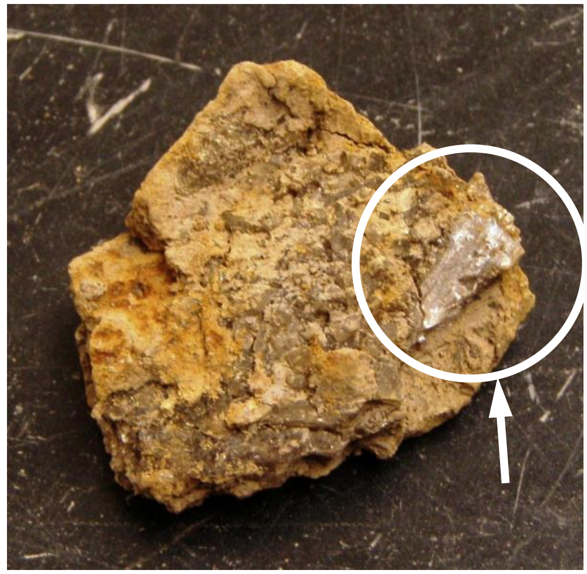

FIGURE 1120-2 Gypsum crystal in clay

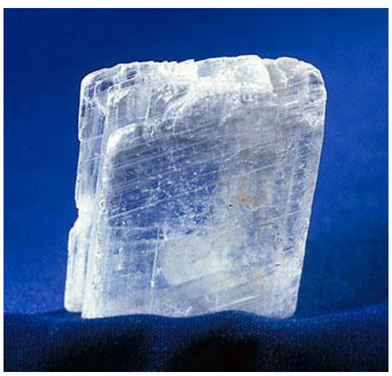

FIGURE 1120-3
Specimen quality gypsum crystal

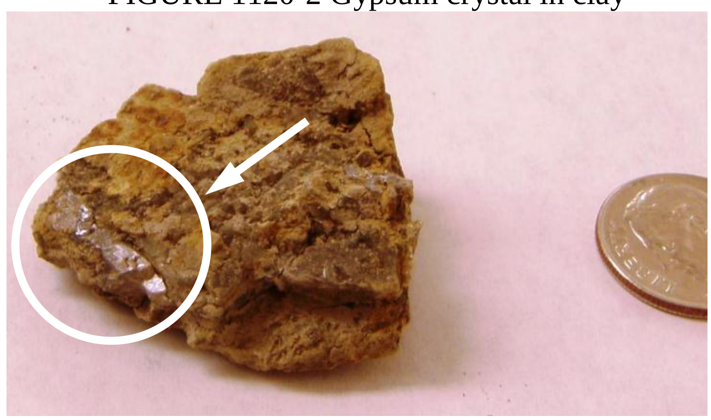

FIGURE 1120-4 Gypsum crystal in clay

FIGURE 1120-5 Gypsum crystals in clay

For more information about identifying minerals, see FHWA (1991) *Rock and Mineral Identification for Engineers*, Publication No. FHWA-HI-91-025, U.S. Department of Transportation.

204	Cubic Yard (Cubic Meter)	Granular Embankment
204	Cubic Yard (Cubic Meter)	Granular Material Type ___
204	Square Yard (Square Meter)	Geotextile Fabric

ITEM 205 CHEMICALLY STABILIZED EMBANKMENT

205.01 Description. This work consists of constructing a chemically stabilized embankment by mixing cement, lime, or lime kiln dust into the embankment soil using the method for the specified chemical.

The Contract Documents include an estimated quantity for the specified chemical.

205.02 Materials. Furnish materials conforming to:

Portland cement 701.04
Lime (quick lime) 712.04.B
Lime kiln dust.. 712.04.C

Furnish water conforming to 499.02. Furnish suitable natural soil, from on or off the project site, conforming to 703.16 and 203.03.

205.03 Submittals. Submit, for the Engineer's acceptance, a report that lists the type of equipment to be used, speed of the intended equipment usage, rate of application of the chemical, and calculations that demonstrate how the required percentage of chemical will be applied. Submit the report to the Engineer for acceptance at least 2 workdays before the stabilization work begins.

If the pay item for Mixture Design for Chemically Stabilized Soils is included in the Contract Documents, prepare and submit reports according to Supplement 1120.

205.04 Construction. Perform chemically stabilized embankment work when the air temperature is 40 °F (5 °C) or above and when the soil is not frozen.

Do not perform this work during wet or unsuitable weather.

Drain and maintain the work according to 203.04.A.

A. Spreading. If the pay item for Mixture Design for Chemically Stabilized Soils is not included in the Contract Documents, use the following spreading percentage rate for the specified chemical. The percentage is based on a dry density for soil of 110 pounds per cubic foot (1760 kg/m³):

TABLE 205.04-1

Chemical	Spreading Rate
Cement	6 %
Lime	5 %
Lime Kiln Dust	7 %

Spread the chemical uniformly on the surface using a mechanical spreader at the approved rate and at a constant slow rate of speed.

Use a distribution bar with a maximum height of 3 feet (1 meter) above the ground surface. Use a canvas shroud that surrounds the distribution bar and extends to the ground surface.

Minimize dusting when spreading the chemical. Control dust according to 107.17. Do not spread chemical when wind conditions create blowing dust that exceeds the limits in 107.19.

Do not spread the chemical on standing water.

B. Mixing. Immediately after spreading the chemical, mix the soil and chemical by using a power driven rotary type mixer. If necessary, add water to bring the mixed material to at least optimum moisture content for cement and lime kiln dust, and to at least 3 percent above optimum moisture content for lime. Continue mixing until the chemical is thoroughly incorporated into the soil, all soil clods are reduced to a maximum size of 2 inches (50 mm), and the mixture is a uniform color.

For areas not under pavements or paved shoulders, the Contractor may use a spring tooth or disk harrow in place of the power-driven rotary type mixer by modifying the above procedure as follows:

1. Open the soil with a spring tooth or disc harrow before spreading.

2. Spread the chemical.

3. Use a minimum disc harrow coverage of ten passes in one direction and ten passes in the perpendicular direction to thoroughly incorporate the chemical into the soil. Continue mixing until all soil clods are reduced to a maximum size of 1 inch (25 mm) and the mixture is a uniform color.

C. Compacting. Construct and compact chemically stabilized embankment according to 203.07, except use 98 percent of the maximum dry density for acceptance.

Determine the maximum dry density for acceptance using the Ohio Typical Moisture Density Curves, the moisture density curves from the Contractor's mixture design submittal, or the maximum dry density obtained by test section method.

205.05 Mixture Design for Chemically Stabilized Soils. When included in the plans, perform a mixture design for chemically stabilized soils according to Supplement 1120.

205.06 Method of Measurement. The Department will measure chemically stabilized embankment by the number of cubic yards (cubic meters) used in the complete and accepted work, as determined by Item 203.

The Department will measure cement, lime, and lime kiln dust by the number of tons (metric tons) incorporated in the complete and accepted work.

205.07 Basis of Payment. The Department will pay lump sum for all work, labor, and equipment described in 205.05. The Department will pay two-thirds of the lump sum amount bid when the sampling and testing is complete and the report is accepted by the Department. The Department will pay one-third of the lump sum amount bid when the chemically stabilized embankment is completed and accepted by the Department, and the field verification test results are all submitted.

The Department will pay for accepted quantities at the contract prices as follows:

Item	Unit	Description
205	Cubic Yard (Cubic Meter)	Cement Stabilized Embankment
205	Cubic Yard (Cubic Meter)	Lime Stabilized Embankment
205	Cubic Yard (Cubic Meter)	Lime Kiln Dust Stabilized Embankment
205	Ton (Metric Ton)	Cement
205	Ton (Metric Ton)	Lime
205	Ton (Metric Ton)	Lime Kiln Dust
205	Lump Sum	Mixture Design for Chemically Stabilized Soils

ITEM 206 CHEMICALLY STABILIZED SUBGRADE

206.01 Description. This work consists of constructing a chemically stabilized subgrade by mixing cement, lime, or lime kiln dust into the subgrade soil using the method for the specified chemical. The Contract Documents include an estimated quantity for the specified chemical.

206.02 Materials. Furnish materials conforming to:

Portland cement 701.04
Lime (quick lime) 712.04.B

APPENDIX B 3

DESIGN PROCEDURES FOR SOIL MODIFICATION OR STABILIZATION

INDIANA DEPARTMENT OF TRANSPORTATION

Design Procedures for Soil Modification or Stabilization

Production Division
Office of Geotechnical Engineering
120 South Shortridge Road
Indianapolis, Indiana 46219

January 2008

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DESIGN PROCEDURES FOR SOIL MODIFICATION OR STABILIZATION

1.0 General

It is the policy of the Indiana Department of Transportation to minimize the disruption of traffic patterns and the delay caused today's motorists whenever possible during the construction or reconstruction of the State's roads and bridges. INDOT Engineers are often faced with the problem of constructing roadbeds on or with soils, which do not possess sufficient strength to support wheel loads imposed upon them either in construction or during the service life of the pavement. It is, at times, necessary to treat these soils to provide a stable subgrade or a working platform for the construction of the pavement. The result of these treatments are that less time and energy is required in the production, handling, and placement of road and bridge fills and subgrades and therefore, less time to complete the construction process thus reducing the disruption and delays to traffic.

These treatments are generally classified into two processes, soil modification or soil stabilization. The purpose of subgrade modification is to create a working platform for construction equipment. No credit is accounted for in this modification in the pavement design process. The purpose of subgrade stabilization is to enhance the strength of the subgrade. This increased strength is then taken into account in the pavement design process. Stabilization requires more thorough design methodology during construction than modification. The methods of subgrade modification or stabilization include physical processes such as soil densification, blends with granular material, use of reinforcements (Geogrids), undercutting and replacement, and chemical processes such as mixing with cement, fly ash, lime, lime by-products, and blends of any one of these materials. Soil properties such as strength, compressibility, hydraulic conductivity, workability, swelling potential, and volume change tendencies may be altered by various soil modification or stabilization methods.

Subgrade modification shall be considered for all the reconstruction and new alignment projects. When used, modification or stabilization shall be required for the full roadbed width including shoulders or curbs. Subgrade stabilization shall be considered for all subgrade soils with CBR of less than 2.

INDOT standard specifications provide the contractor options on construction practices to achieve subgrade modification that includes chemical modification, replacement with aggregates, geosynthetic reinforcement in conjunction with the aggregates, and density and moisture controls. Geotechnical designers have to evaluate the needs of the subgrade and include where necessary, specific treatment above and beyond the standard specifications.

Various soil modification or stabilization guidelines are discussed below. It is necessary for designers to take into consideration the local economic factors as well as environmental conditions and project location in order to make prudent decisions for design.

It is important to note that modification and stabilization terms are not interchangeable.

2.0 Modification or Stabilization of Soils

2.01 Mechanical Modification or Stabilization

This is the process of altering soil properties by changing the gradation through mixing with other soils, densifying the soils using compaction efforts, or undercutting the existing soils and replacing them with granular material.

A common remedial procedure for wet and soft subgrade is to cover it with granular material or to partially remove and replace the wet subgrade with a granular material to a pre-determined depth below the grade lines. The compacted granular layer distributes the wheel loads over a wider area and serves as a working platform. (1)

To provide a firm-working platform with granular material, the following conditions shall be met.

1. The thickness of the granular material must be sufficient to develop acceptable pressure distribution over the wet soils.

2. The backfill material must be able to withstand the wheel load without rutting.

3. The compaction of the backfill material should be in accordance with the Standard Specifications.

Based on the experience, usually 12 to 24 in. (300 to 600mm) of granular material should be adequate for subgrade modification or stabilization. However, deeper undercut and replacement may be required in certain areas

The undercut and backfill option is widely used for construction traffic mobility and a working platform. This option could be used either on the entire project or as a spot treatment. The equipment needed for construction is normally available on highway construction projects.

2.02 Geosynthetic Stabilization

Geogrid has been used to reinforce road sections. The inclusion of geogrid in subgrades changes the performance of the roadway in many ways (6). Tensile reinforcement, confinement, lateral spreading reduction, separation, construction uniformity and reduction in strain have been identified as primary reinforcement mechanisms. Empirical design and post-construction evaluation have lumped the above described benefits into better pavement performance during the design life. Geogrid with reduced aggregate thickness option is designed for urban area and recommendations are follows;

Excavate subgrade 9 in. (230 mm) and construct the subgrade with compacted aggregate No. 53 over a layer of geogrid, Type I. This geogrid reinforced coarse aggregate should provide stable working platform corresponding to 97 percent of CBR. Deeper subgrade problem due to high moisture or organic soils requires additional recommendations.

Geogrid shall be in accordance with 918.05(a) and be placed directly over exposed soils to be modified or stabilized and overlapped according with the following table.

SPT blow Counts per foot (N)	Overlap
> 5	12 in. (300 mm)
3 to 5	18 in. (450 mm)
less than 3	24 in. (600 mm)

2.03 Chemical Modification or Stabilization

The transformation of soil index properties by adding chemicals such as cement, fly ash, lime, or a combination of these, often alters the physical and chemical properties of the soil including the cementation of the soil particles. There are the two primary mechanisms by which chemicals alter the soil into a stable subgrade:

1. Increase in particle size by cementation, internal friction among the agglomerates, greater shear strength, reduction in the plasticity index, and reduced shrink/swell potential.

2. Absorption and chemical binding of moisture that will facilitate compaction.

3.0 Design Procedures

3.01 Criteria for Chemical Selection

When the chemical stabilization or modification of subgrade soils is considered as the most economical or feasible alternate, the following criteria should be considered for chemical selection based on index properties of the soils. (2)

1. Chemical Selection for Stabilization.

   a. Lime: If PI > 10 and clay content (2μ) > 10%.

   b. Cement: If PI ≤ 10 and < 20% passing No. 200.

   Note: Lime shall be quicklime only.

2. Chemical Selection for Modification

   a. Lime: PI ≥ 5 and > 35 % Passing No. 200

   b. Fly ash and lime fly ash blends: 5 < PI < 20 and > 35 % passing No. 200

   c. Cement and/ or Fly ash: PI < 5 and ≤ 35 % Passing No. 200

   Fly ash shall be class C only.
   Lime Kiln Dust (LKD) shall not be used in blends.
   Appropriate tests showing the improvements are essential for the exceptions listed above.

3.02 Suggested Chemical Quantities For Modification Or Stabilization

a. Lime or Lime By-Products: 4% to7 %

b. Cement: 4% to 6%

c. Fly ash Class C: 10% to 16%

% for each combination of lime-fly ash or cement-fly ash shall be established based on laboratory results.

3.03 Strength requirements for stabilization and modification

The reaction of a soil with quick lime, or cement is important for stabilization or modification and design methodology. The methodology shall be based on an increase in the unconfined compression strength test data. To determine the reactivity of the soils for lime stabilization, a pair of specimens measuring 2 in. (50 mm) diameter by 4 in. (100 mm) height (prepared by mixing at least 5% quick lime by dry weight of the natural soil) are prepared at the optimum moisture content and maximum dry density (AASHTO T 99). Cure the specimens for 48 hours at 120^{o} F (50^{o} C) in the laboratory and test as per AASHTO T 208. The strength gain of lime-soil mixture must be at least **50 psi** (350 kPa) greater than the natural soils. A strength gain of **100 psi** (700 kPa) for a soil-cement mixture over the natural soil shall be considered adequate for cement stabilization with 4% cement by dry weight of the soils and tested as described above

In the case of soil modification, enhanced subgrade support is not accounted for in pavement design. However, an approved chemical (LKD, cement, and fly ash class C) or a combination of the chemicals shall attain an increase in strength of **30 psi** over the natural soils when specimens are prepared and tested in the same manner as stabilization.

4.0 Laboratory Test Requirements

Soil Sampling and Suitability: An approved Geotechnical Engineer should visit the project during the construction and collect a bag sample of each type of soil in sufficient quantity for performing the specified tests. The geotechnical engineer should review the project geotechnical report and other pertinent documents such as soil maps, etc., prior to the field visit. The geotechnical consultant shall submit the test results and recommendations, along with the current material safety data sheet or mineralogy to the engineer for approval.

When the geotechnical engineer determines the necessity of chemical-soil stabilization during the design phase, they should design a subgrade treatment utilizing the chemical for the stabilization in the geotechnical report in accordance with INDOT guidelines. Following tests should be performed and the soils properties should be checked prior to any modification or stabilization.

a. Grain size and Hydrometer test results in accordance with AASHTO T 89, 90, and M145,

b. Atterberg limits,

c. Max. Dry unit weight of 92 pcf (Min.) in accordance with AASHTO T 99,

d. Loss of ignition (LOI) not more than 3% by dry weight of soil in accordance with AASHTO T 267,

e. Carbonates not more than 3 % by dry weight of the soils, if required,

f. As received moisture content in accordance with AASHTO T 265.

4.01 <u>Lime or Lime By-Products Required for Modification or Stabilization</u>.

Lime reacts with medium, moderately fine and fine-grained soils to produce decreased plasticity, increased workability, reduced swelling, and increased strength. The major soil properties and characteristics that influence the soils ability to react with lime to produce cementitious materials are pH, organic content, natural drainage, and clay mineralogy. As a general guide, treated soils should increase in particle size with cementation, reduction in plasticity, increased in internal friction among the agglomerates, increased shear strength, and increased workability due to the textural change from plastic clay to friable, sand like material.

The following procedures shall be utilized to determine the amount of lime required to stabilize the subgrade. Hydrated or quick lime and lime by-products should be used in the range of 4 ± 0.5% and 5 ± 1% by weight of soil for modification respectively. The following procedures shall be used to determine the optimum lime content.

Perform mechanical and physical tests on the soils.

Determine the separate pH of soil and lime samples.

Determine optimum lime content using Eades and Grim pH test.

- A sufficient amount of lime shall be added to soils to produce a pH of 12.4 or equal to the pH of lime itself. An attached graph is plotted showing the pH as lime content increases. The Optimum lime content shall be determined corresponding to the maximum pH of lime-soil mixture. (See Figure 4.0 A).

- Representative samples of air-dried, minus No. 40 soil is equal to 20 g of oven-dried soil are weighed to the nearest 0.1 g and poured into 150-ml (or larger) plastic bottles with screw on tops.

- It is advisable to set up five bottles with lime percentages of 3, 4, 5, 6, and 7. This will insure, in most cases, that the percentage of lime required can be determined in one hour. Weigh the lime to the nearest 0.01 g and add it to the soil. Shake the bottle to mix the soil and dry lime.

- Add 100 ml of CO_2-free distilled water to the bottles.

- Shake the soil-lime mixture and water until there is no evidence of dry material on the bottom. Shake for a minimum of 30 seconds.

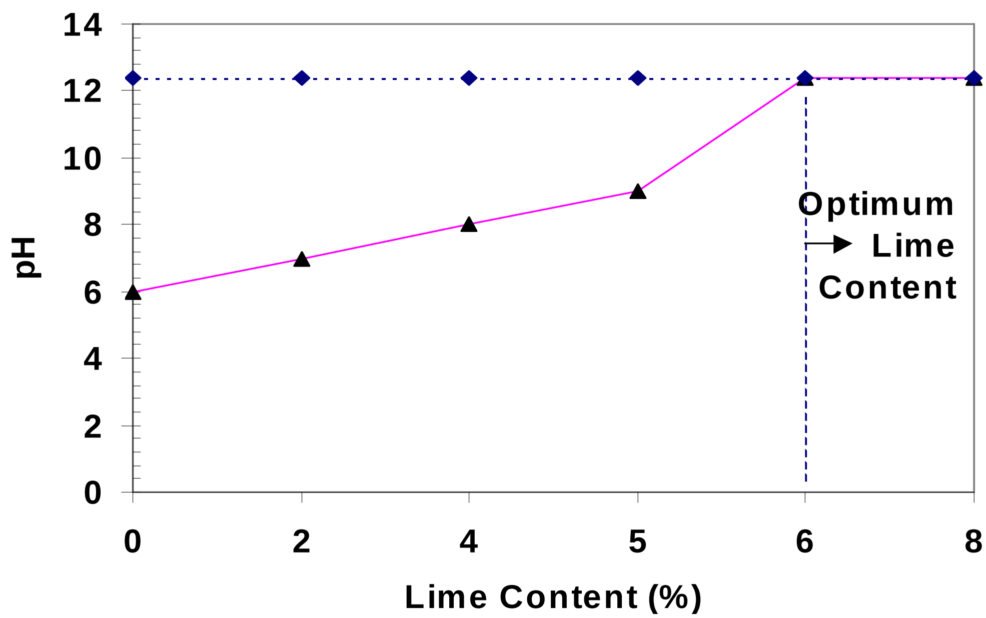
- Shake the bottles for 30 seconds every 10 minutes.

- After one hour, transfer part of the slurry to a plastic beaker and measure the pH. The pH meter must be equipped with a Hyalk electrode and standardized with a buffer solution having a pH of 12.00.

- Record the pH for each of the lime-soil mixtures. If the pH readings go to 12.40, then the lowest percent lime that gives a pH of 12.40 is the percentage required to stabilize the soil. If the pH does not go beyond 12.30 and 2 percentages of lime give the same readings, the lowest percent which gives a pH of 12.30 is the amount required to stabilize the soil. If the highest pH is 12.30 and only 1 percent lime gives a pH of 12.30, additional test bottles should be started with larger percentages of lime.

d. Atterberg limits should be performed on the soil-lime mixtures corresponding to optimum lime content as determined above.

e. Compaction shall be performed in accordance with AASHTO T 99 on the optimum lime and soil mixture to evaluate the drop in maximum dry density in relation to time (depending on the delay between the lime-soil mixing and compaction.)



pH vs. Lime Content

Figure 4.0 A

In the case of stabilization, the Unconfined Compression Test (AASHTO T 208) and California Bearing Ratio (AASHTO T 193, soaked) or resilient modulus (AASSHTO T 307) tests at 95% compaction shall be performed in addition to the above tests corresponding to optimum lime-soil mixture of various predominant soils types.

4.02 <u>Cement Required for Stabilization or Modification</u>

The criteria for cement percentage required for stabilization shall be as follows. The following methodology shall be used for quality control and soil-cement stabilization.

1. Perform the mechanical and physical property tests of the soils.

2. Select the Cement Content based on the following:

AASHTO Classification	Usual Cement Ranges for Stabilization (% by dry weight of soil)
A-1-a	3 – 5
A-1-b	5 – 8
A-2	5 – 9
A-3	7 – 10

Suggested Cement Contents
Figure 4.0B

3. Perform the Standard Proctor on soil-cement mixtures for the change in maximum dry unit weight in accordance with AASTO T 134.

4. Perform the unconfined compression and CBR tests on the pair of specimens molded at 95% of the standard Proctor in case of stabilization. A gain of 100 psi of cement stabilization is adequate enough for stabilization and % cement shall be adjusted.

Although, there is no test requirement for the optimum cement content when using cement to modify the subgrade. An amount of cement 4% ± 0.50% by dry weight of the soil should be used for the modification of the subgrade.

4.03 <u>Fly Ash Required for Modification</u>

1. The in-situ soils should meet the criteria for modification.

2. Standard Proctor testing should be performed in accordance with AASHTO T 99 to determine the maximum dry density and optimum moisture content of the soil.

3. A sufficient amount of fly ash (beginning from 10% by dry weight of soil) should be mixed with the soil in increments of at least 5%. The moisture content of the mix shall be in the range of optimum moisture content + 2%. Each blend of the fly ash soil mixture should be compacted as per the standard Proctor to determine the maximum dry density.

4. The compaction of the mixes shall be completed within 2 hours.

5. The percentage of fly ash, which provides the maximum dry density, should be considered the **optimum amount of fly ash** for that soil.

6. The compressive strength of the **optimum fly ash mix** should be determined 2, 4, and 8 hours after compaction.

7. A pair of specimens of the **optimum fly ash** mix should be molded of standard Proctor and soaked for 4 days. The swelling should be observed daily. A percentage swell of more than 3 not be allowed in soils modification.

4.04 Combination of Cement Fly Ash and Lime Mixture

To enhance the effectiveness of lime, cement or fly ash modification or stabilization combinations, the subsequent guidelines shall be used. An increase of **50 to 100 psi** over the natural soil is required for the stabilization and an increase of **30 psi** over the natural soils is required for modification.

1. Lime and Fly ash: The ratio between lime and fly ash mixture should be in the range of 1:1 to 1:9 respectively.

2. Cement and Fly ash: The ratio of cement and fly ash should be in the range of 1:3 to 1:4 respectively.

3. Lime, cement, and fly ash ratio should be 1:2:4 respectively.

5.0 Construction Considerations

Modification of soils to speed construction by drying out wet subgrades with lime, cement and fly ash is not as critical as completely stabilizing the soil to be used as a part of the pavement structure. With the growth of chemical modification throughout Indiana, a variety of applications are being suggested due to such factors as soil types, percentage of modification/stabilization required, environmental restraints, and availability of chemicals. Furthermore, when chemically stabilized subgrades are used to reduce the overall thickness of the roadway then the stabilized layer must be built under tight construction specifications; whereas the requirements for the construction of a working platform are more lenient. Following are a few recommendations for modification or stabilization of subgrade soils.

1. Perform recommended tests on each soil to see if the soil will react with chemicals then determine the amount of chemical necessary to produce the desired results.

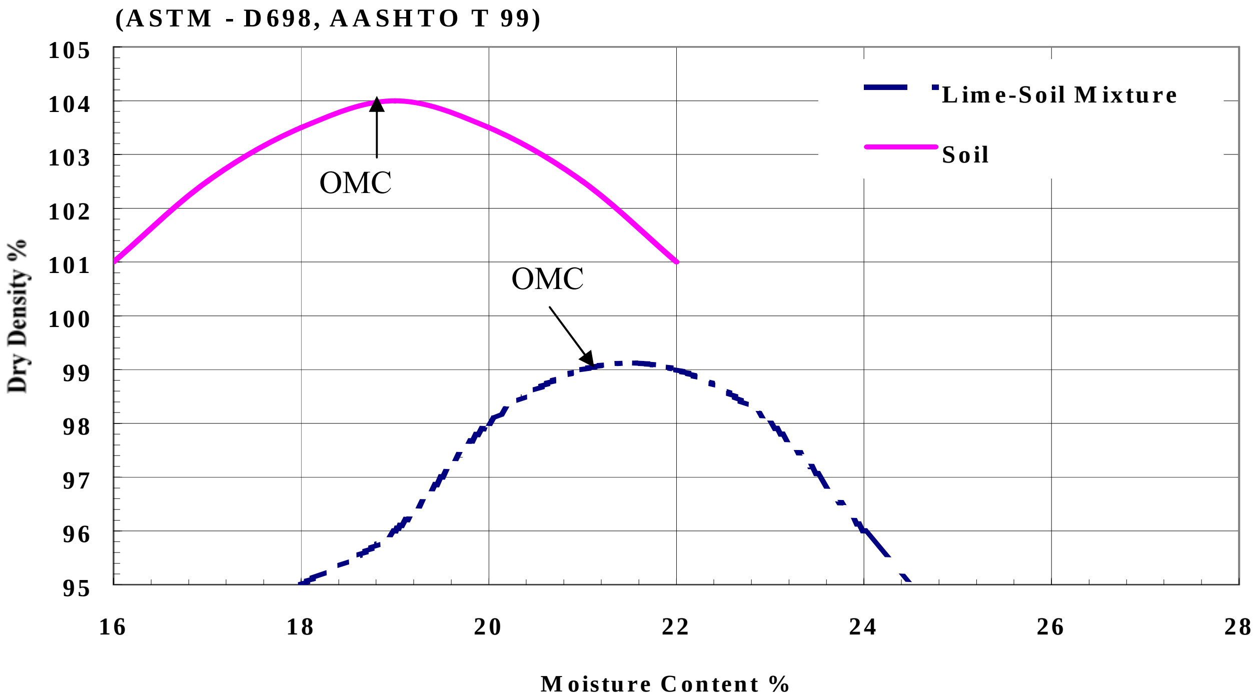
2. More chemicals may not always give the best results.

3. Sulfate, when mixed with calcium will expand. Soils having over 10% sulfate content shall not be mixed with chemicals.

4. Chemicals used shall meet the INDOT Standard Specifications.

5. One increment of chemical is recommended to produce a working platform. Proofrolling is required before placing the base or subbase. Pavement shall not be installed before curing is completed.

6. The density of cement treated soils may likely be different than that of untreated soils. Standard Proctor tests should be performed in the laboratory to estimate the appropriate target density.



Moisture Density Relationship
Figure 5.0 A

7. The grade should be set low to account for the swell in the lime. A swell factor of 10% is an approximate estimate.

8. Uniform distribution of chemical, throughout the soil is very important.

9. Curing takes 7 days of 50° F or above weather for stabilization. No heavy construction equipment should be allowed on the stabilized grade during the curing period.

10. The maximum dry density of the soil-lime mixture is lower than in untreated soils. Maximum dry density reduction of 3-5 Pcf approximately, is common for a given compactive effort. It is, therefore, important that the laboratory for field control purposes provide appropriate density. (See Figure 5.0A).

11. The modified or stabilized roadbed must be covered with pavement before suspending work for the winter and construction traffic shall be limited

12. Cement or fly ash treated soils exhibit shrinkage cracks due to soil type, curing, chemical contents, etc. Therefore, it is recommended to provide surface sealing on stabilized subgrade after the curing period.

13. Moisture content of modified or stabilized subgrade should be maintained above the optimum moisture content of modified subgrade during the curing.

14. Lime raises the pH of the soil. Phenolphthalein, a color sensitive indicator solution can be sprayed on the soil to determine the presence of lime. If lime is present, a reddish-pink color develops. (See Figure 5.0B).

Lime Modified Subgrade Uniformity Determination by Phenolphthalein
Figure 5.0B

15. Because lime can cause chemical burns, safety gear, such as gloves, eye protection, and dust masks shall be used during construction and inspection.

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APPENDIX B 4

MIXTURE DESIGN AND TESTING PROCEDURES FOR LIME STABILIZED SOILS

NATIONAL LIME ASSOCIATION

LIME

The Versatile Chemical

Technical Brief

Mixture Design and Testing Procedures for Lime Stabilized Soil

Steps for Mixture Design and Testing for Lime Stabilized Soil

Evaluate soil to gain a general understanding of its suitability for lime stabilization.

Determine minimum amount of lime required for stabilization.

Evaluate lime-stabilized soil strength for long term durability within its exposure environment, with special attention to cyclic freezing and thawing and periods of extended soaking.

If soils to be stabilized are expansive, evaluate using capillary soaking and expansion measurements.

The use of lime to dry, modify, and stabilize soil is a well established construction technique, documented in studies dating back to the 1950s and 1960s [see Ref. 1]. A variety of mixture proportioning procedures have evolved, as various agencies have developed criteria and procedures to fit their specific design needs and objectives, often reflecting local conditions and experience [1].*

The procedures outlined in this publication are intended for soil that is to be stabilized with lime, not merely dried or modified. These procedures are intended to help ensure the long term strength and durability of a lime stabilized soil and are not typically required when soil drying and modification is the desired goal. Other laboratory tests, such as measuring decrease in soil moisture content or reduction in plasticity index (PI), are more appropriate when soil drying/modification is the intended result.

In 1999, the National Lime Association commissioned Dr. Dallas Little to evaluate various procedures and develop a definitive lime stabilization mixture design and testing procedure (MDTP) that specifying agencies, design engineers, and laboratory personnel could use with confidence for soil conditions and environmental exposures throughout the United States. The resulting series of reports summarize the literature on lime stabilization [2, 3]; describe mix proportioning and testing procedures for lime stabilized soil [4]; and present a field validation of the protocol [5].

Lime-Treated Soil – Drying, Modification, and Stabilization

Lime has a number of effects when added into soil [6, 7], which can be generally categorized as soil drying, soil modification, and soil stabilization:

- Soil drying is a rapid decrease in soil moisture content due to the chemical reaction between water and quicklime and the addition of dry material into a moist soil. [8]
- Modification effects include: reduction in soil plasticity, increase in optimum moisture content, decrease in maximum dry density, improved compactability, reduction of the soil's capacity to swell and shrink, and improved strength and stability after compaction. These effects generally take place within a short time period after the lime is introduced – typically 1 to 48 hours – and are more pronounced in soils with sizable clay content, but may or may not be permanent.
- Lime stabilization occurs in soils containing a suitable amount of clay and the proper mineralogy to produce long-term strength; and permanent reduction in shrinking, swelling, and soil plasticity

** Construction techniques are not addressed in this publication--see Ref. 6.*

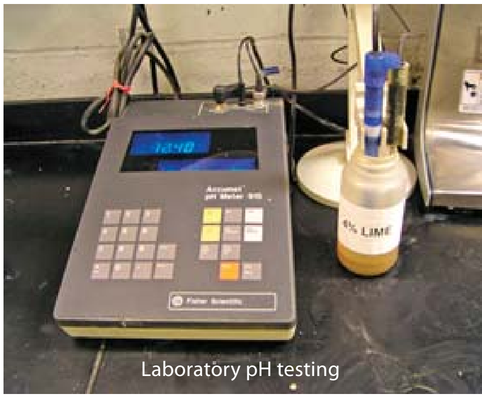
with adequate durability to resist the detrimental effects of cyclic freezing and thawing and prolonged soaking. Lime stabilization occurs over a longer time period of "curing." The effects of lime stabilization are typically measured after 28 days or longer, but can be accelerated by increasing the soil temperature during the curing period. A soil that is lime stabilized also experiences the effects of soil drying and modification.

Lime Stabilization Mix Design and Testing Procedures

The procedures outlined in this document are to evaluate if a soil can be stabilized with lime and, if so, determine the minimum amount of lime required for long-term strength, durability and the other desired properties of the stabilized soil. This is achieved by:

- Initially evaluating the soil to gain a general understanding of its suitability for lime stabilization.
- Determining the minimum amount of lime required for stabilization.
- Evaluating the lime-stabilized soil strength for long term durability within its exposure environment, with special attention to cyclic freezing and thawing and periods of extended soaking.
- If the soils to be stabilized are expansive, evaluate using capillary soaking and expansion measurements.

Steps for Mixture Design and Testing for Lime Stabilized Soil



Laboratory pH testing

Step 1 – Initial Soil Evaluation

Purpose: Evaluate key soil characteristics as an initial step to determine if it is suitable for lime stabilization.

Procedure: Use ASTM C136 [10] procedures to determine the amount of soil passing the 75 micron (75-µm) screen and ASTM D 4318 (wet method) [11] to determine the soil plasticity index (PI).

Criteria: Generally, soil with at least 25% passing a 75 micron screen and having a PI of 10 or greater are candidates for lime stabilization. Some soils with lower PI can be successfully stabilized with lime, provided the pH and strength criteria described in this document can be satisfied.

Additional Considerations: Soil with organics content above 1-2% by weight as determined by ASTM D 2974 [12] may be incapable of achieving the desired unconfined compressive strength for lime stabilized soil (Step 6) [13]. Soils containing soluble sulfates greater than 0.3% can be successfully stabilized with lime, but may require special precautions (see NLA's "Technical Memorandum – Guidelines for Stabilization of Soils Containing Sulfates" Ref. 14 for more information).

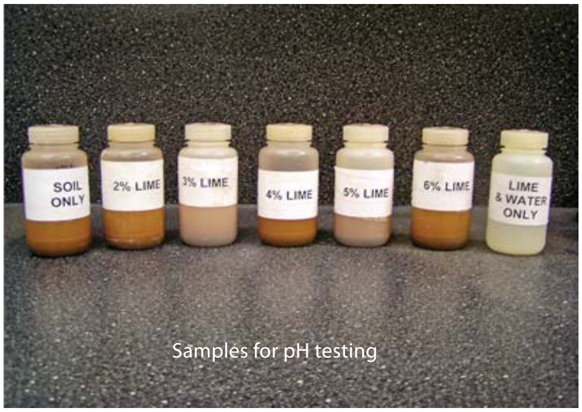
Step 2 – Determine the Approximate Lime Demand

Purpose: Determine the minimum amount of lime required for stabilization.

Procedure: Use ASTM D 6276 [15] procedures. This is also known as the "Eades-Grim" test.

Criteria: The lowest percentage of lime in soil that produces a laboratory pH of 12.4 [flat section of the pH vs. lime percentage curve produced by the test] is the minimum lime percentage for stabilizing the soil.

Additional Considerations: ASTM D 6276 has additional provisions for cases in which the measured laboratory pH is 12.3 or less. Note that lime can react with moisture and carbon dioxide. Careful storage is required to maintain lime's integrity and produce reliable results.



Samples for pH testing

Step 3 – Determine Optimum Moisture Content and Maximum Dry Density of the Lime-Treated Soil

Purpose: Determine optimum moisture content (OMC) and maximum dry density (MDD) of the soil after lime has been added. This is necessary because adding lime will change the soil's OMC and MDD.


Lab mixing of soil & lime

Procedure: Make a mixture of soil, lime, and water at the minimum percentage of lime as determined from Step 2 (Eades-Grim test), using a water content of OMC + 2-3%. Seal the mixture in an airtight, moisture proof bag stored at room temperature for 1-24 hours. Determine the OMC and MDD of the mixture using ASTM D 698 procedures (standard compaction effort) [16].

Criteria: Determine the OMC and MDD for Step 4.

Additional Considerations: When using quicklime, the mixture should be stored for 20-24 hours to ensure hydration.

Step 4 – Fabricate Unconfined Compressive Strength (UCS) Specimens

Purpose: Fabricate test specimens for UCS testing (Step 6).

Procedure: Using ASTM D 5102 [17] procedure B, fabricate a minimum of two test specimens of lime, soil and water using the amount (percentage) of lime determined from Step 2 at the OMC (± 1%) as determined from Step 3. The soil-lime-water mixture should be stored in an airtight, waterproof bag for 1-24 hours prior to fabricating the test specimens.

Desired Result: A minimum of two specimens for UCS testing.


Making Proctor specimen

Additional Considerations: When using quicklime, the mixture should be stored for 20-24 hours to ensure hydration. Additional specimens may be fabricated if additional testing is desired. In some cases it may be advisable to make test specimens at higher lime content(s) than that determined from ASTM D 6276 testing (Step 2). These additional specimens can be used to determine the UCS of lime-soil-water mixtures at higher lime contents. For instance, if ASTM D 6276 testing (Step 2) indicates that 4% lime is needed, additional UCS testing could be done at 5% and 6% lime to ensure that the UCS criteria (Step 6) is also achieved.

Step 5 – Cure and Condition the Unconfined Compressive Strength (UCS) Specimens

Purpose: Approximate, in an accelerated manner, field curing and moisture conditions.


Soaking specimens

Procedure: Immediately following the fabrication of the test specimens, wrap the specimens in plastic wrap and seal in an airtight, moisture proof bag. Cure the specimens for 7 days at 40°C. Subject the specimens to a 24 hour capillary soak prior to testing.

The capillary soaking process should be done by removing the specimens from the airtight bag, then removing the plastic wrapping. The specimens are wrapped with wet absorptive fabric and placed on a porous stone. The water level should reach the top of the stone and be in contact with the fabric wrap throughout the capillary soak process, but the soil specimen should not come directly into contact with the water.

Desired Result: A minimum of two cured and moisture conditioned specimens for UCS testing.

Step 6 – Determine the Unconfined Compressive Strength (UCS) of the Cured and Moisture Conditioned Specimens

Purpose: To determine the UCS of the lime-stabilized soil to ensure adequate field performance in a cyclic freezing and thawing and an extended soaking environment.


Unconfined compressive strength test (ASTM D5102)

Procedure: Use ASTM D 5102 procedure B to determine the UCS of the cured and moisture conditioned specimens. The UCS is the average of the test results for a least two specimens.

Criteria: The minimum desired UCS depends on the intended use of the soil, the amount of cover material over the stabilized soil, exposure to soaking conditions, and the expected number of freezing and thawing cycles during the first winter of exposure. Suggested minimum UCS are shown in the following table.

Soil-Lime Mixture Unconfined Compressive Strength Recommendations [18]				
Anticipated Use	UCS Recommendations for Various Anticipated Service Conditions			
	Extended Soaking for 8 Days (psi)	Cyclic Freeze-Thaw[a]		
		3 Cycles (psi)	7 Cycles (psi)	10 Cycles (psi)
Subbase				
Rigid Pavement/ Floor Slabs/ Foundations	50	50	90	120
Flexible Pavement (> 10 in.)[b]	60	60	100	130
Flexible Pavement (8 in -10 in.)[b]	70	70	100	140
Flexible Pavement (5 in. – 8 in.)[b]	90	90	130	160
Base				
	130	130	170	200

Notes:

a – Number of freeze-thaw cycles expected in soil-lime layer during the 1st winter of exposure.

b – Total pavement thickness overlying the subbase.

Step 7 – Determine the Change in Expansion Characteristics [only for expansive soils]

Purpose: To evaluate the expansiveness of lime stabilized soils.

Procedure: Note the vertical and circumferential dimensions of the samples fabricated in Step 5 prior to performing the capillary soak. After soaking, perform new measurements using a caliper for the vertical dimension and a pi-tape for the circumference. Calculate the volume change between the initial (dry) condition and the soaked condition.

Criteria: Three-dimensional expansion of between 1 and 2% is commonly regarded as acceptable.

Additional Considerations: If the expansion exceeds the design parameter, fabricate additional samples increasing the lime content by 1 and 2% and repeat the test. If additional expansion, shrinkage, and uplift pressure data is desired, perform ASTM D3877 [19]. This step is applicable only to expansive soils.

Other Considerations

The procedures outlined in this document can be used to determine whether a soil can be stabilized with lime and, if so, to quantify the minimum amount of lime required to produce long-term strength, durability, and the other desired properties of a lime-stabilized soil. Typical construction specifications require 0.5 - 1.0 percent more lime than suggested by laboratory procedures, to account for differences between lab and field techniques (for example, field gradation vs. controlled lab pulverization) and field variability.

Other characteristics and properties of the soil, both untreated and lime-treated, may be important for engineering design, construction, and quality control. These characteristics and properties may include, for example: moisture content, moisture reduction, gradation, soil classification, Atterberg limits, organic content, soluble sulfate content, strength characteristics and indices such as CBR, modulus of resilience (Mr), modulus of subgrade reaction (k), R-value, shear strength, and bearing strength. The effect of lime to improve many of these soil properties and characteristics is often substantial, but beyond the scope of this document. They should however, be evaluated as required on a project by project basis.

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17. ASTM D5102 "Standard Test Method for Unconfined Compressive Strength of Compacted Soil-Lime Mixtures" (www.astm.org)

18. Excerpted from Reference 2, Table 3, page 36 (after Thompson, 1970).

19. ASTM D3877 "Standard Test Methods for One-Dimensional Expansion, Shrinkage, and Uplift Pressure of Soil-Lime Mixtures" (www.astm.org).

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