PERFORMANCE EVALUATION OF SUBGRADE STABILIZATION WITH RECYCLED MATERIALS

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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.

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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.

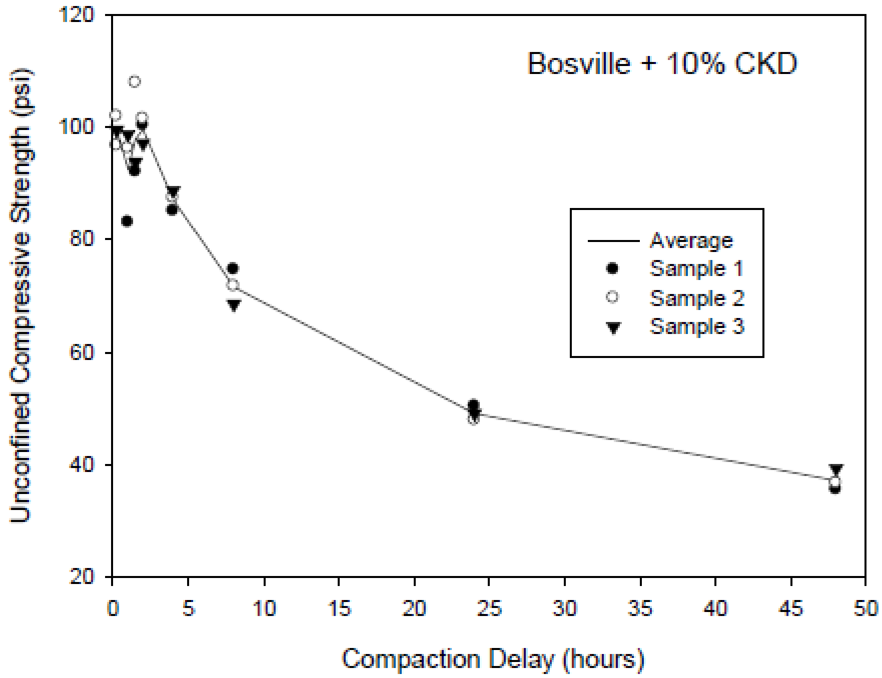


Figure 2.1: Reduction in Unconfined Compression Strength due to Compaction Delay (Cerato et al, 2011)

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.

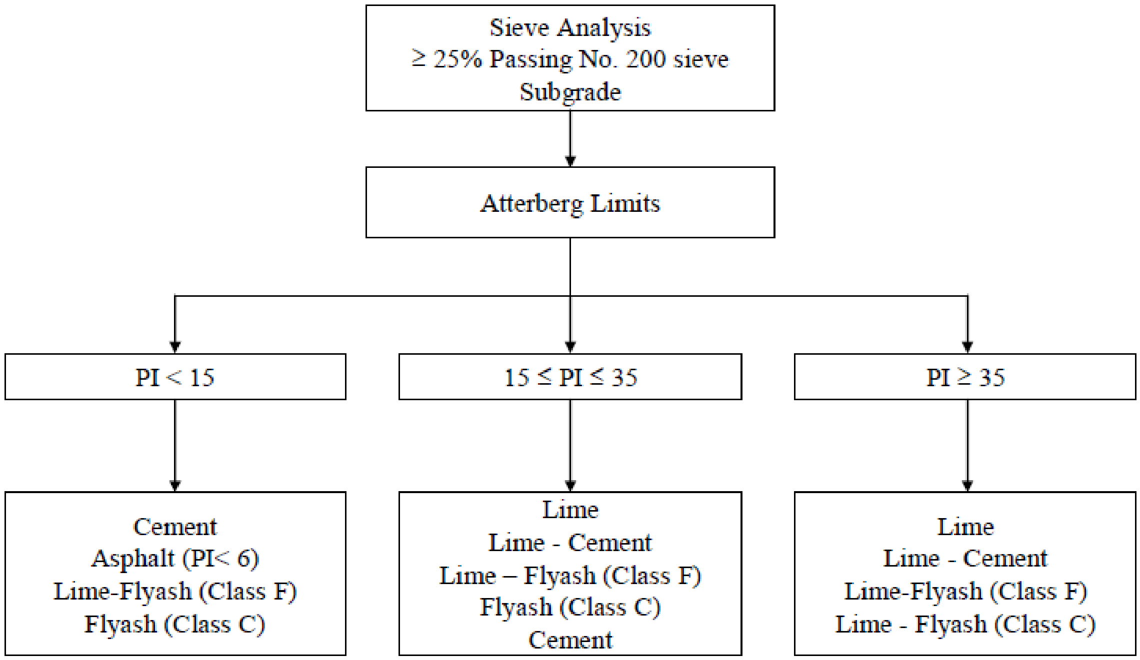


Figure 2.2: Decision Tree for Selecting Stabilizers for Use in Subgrade Soils (Little et al, 2009)

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.

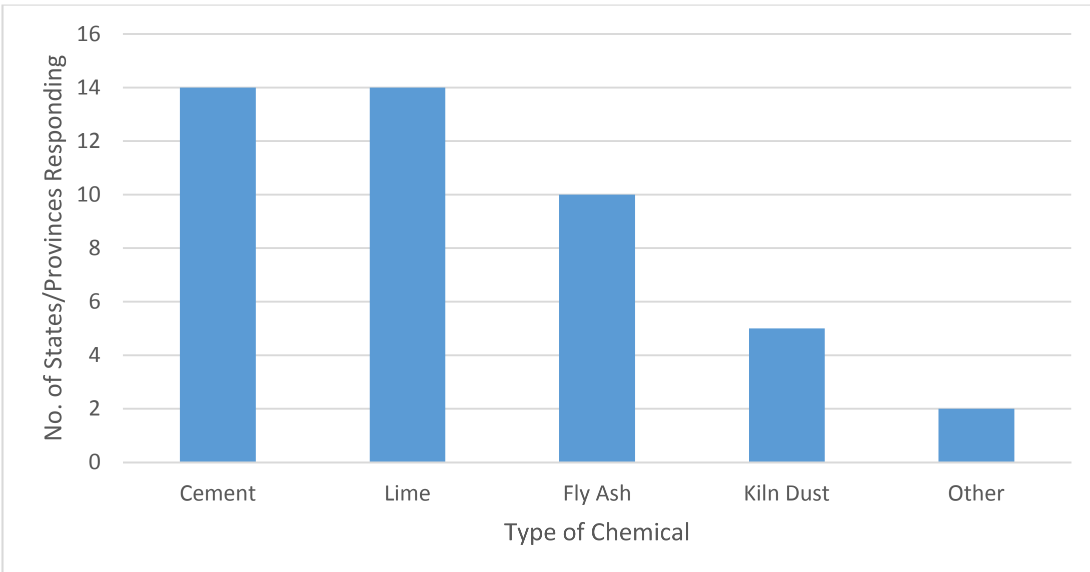


Figure 2.3: Types of Chemicals Used for Subgrade Stabilization by Various States (Sargand, 2014)

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.

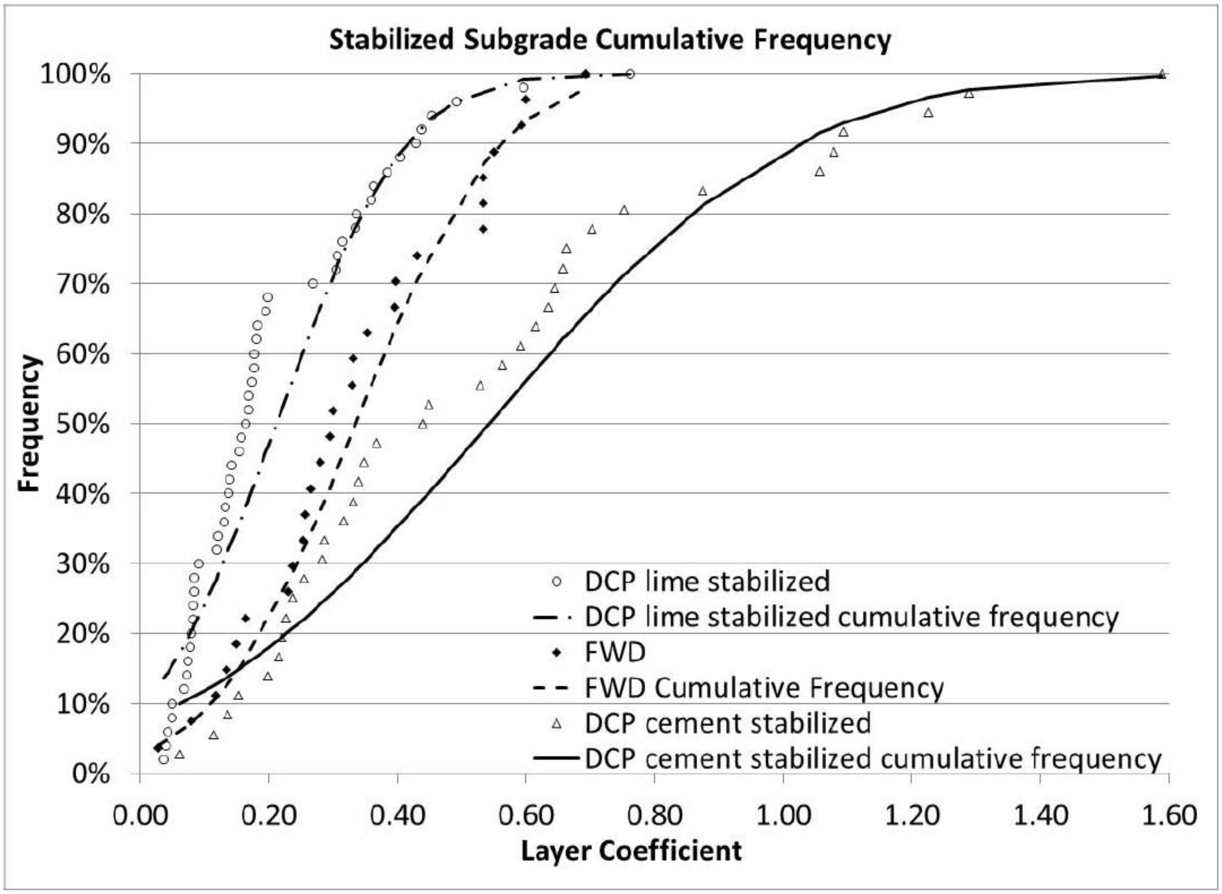


Figure 2.4: Cumulative Frequency of Stabilized Subgrade Layer Coeffcients – FWD vs. DCP (Sargand, 2014)

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).

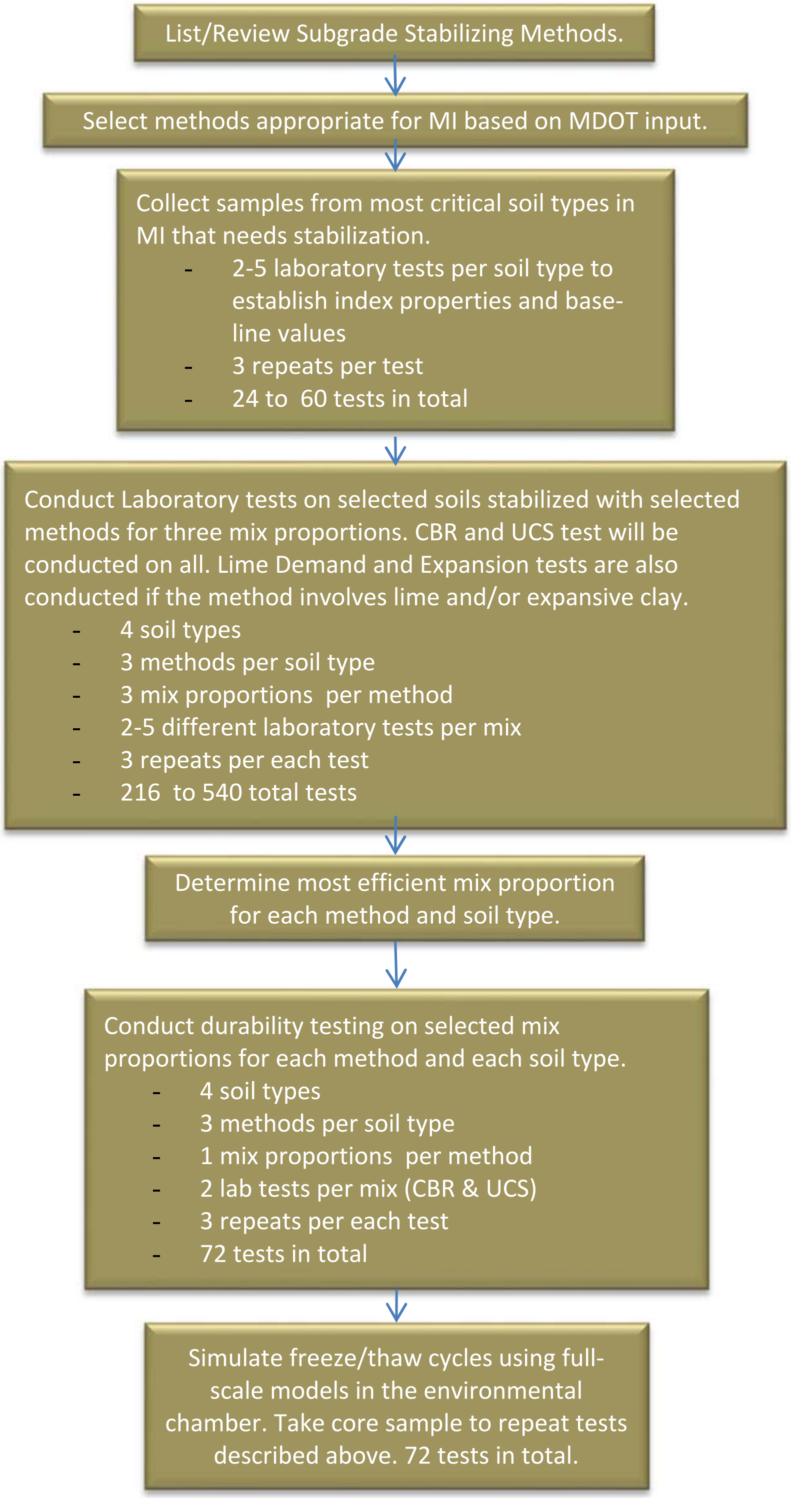


Figure 3.1: Summary of the Laboratory Test Program Process

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

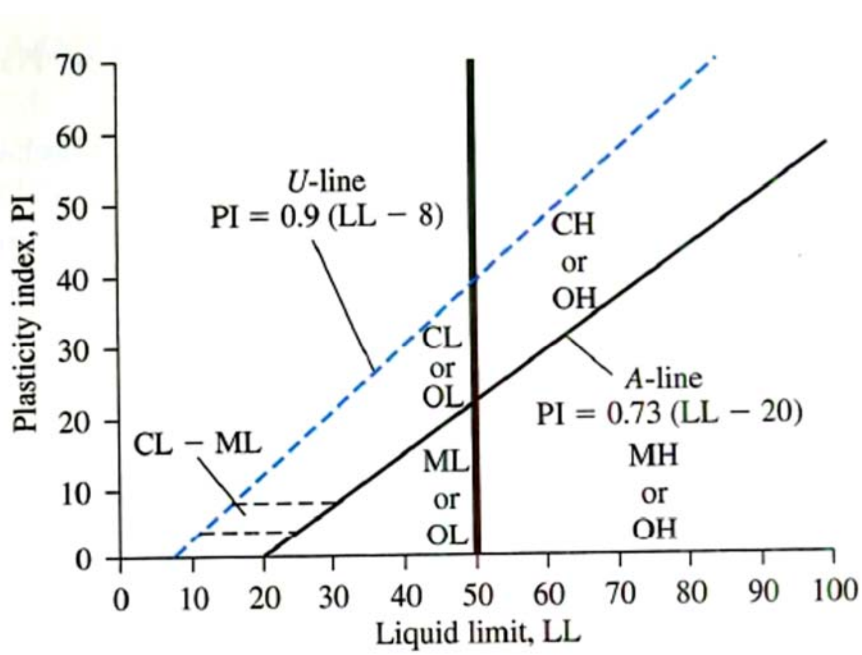
Figure 3.3: Cassagrande Apparatus for Liquid Limit Testing

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

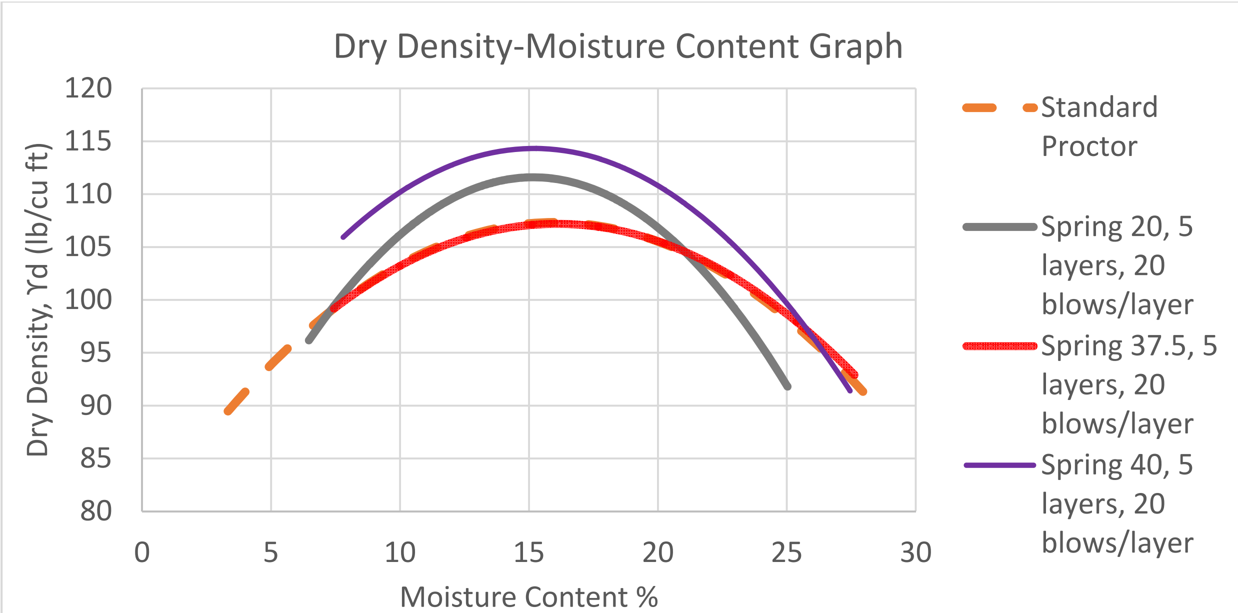


Figure 3.8: Calibration Graph of Harvard Miniature Compaction Apparatus

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.

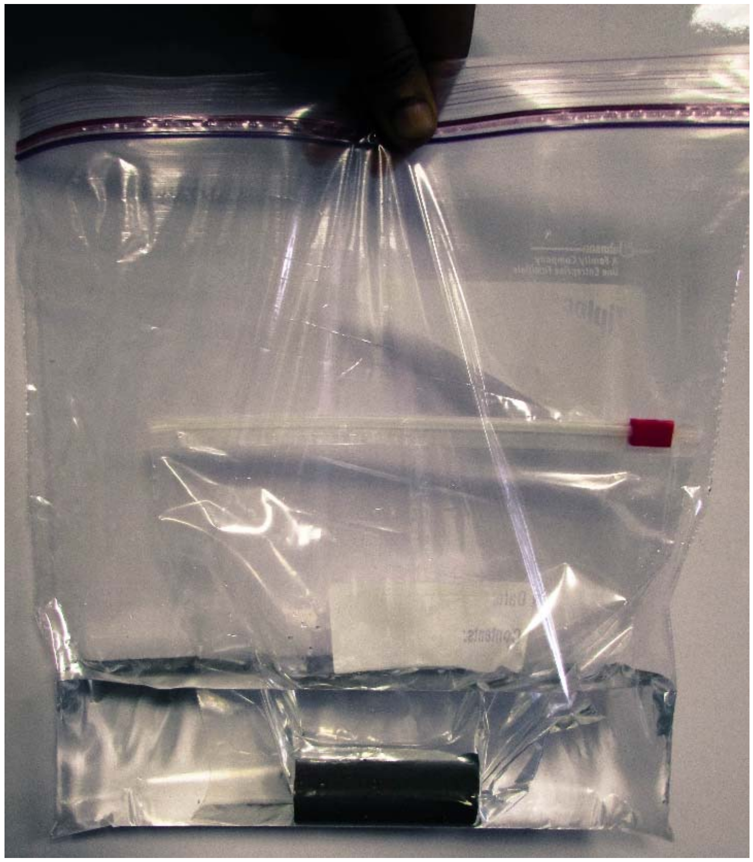


Figure 3.9: Sample Curing Procedure

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.

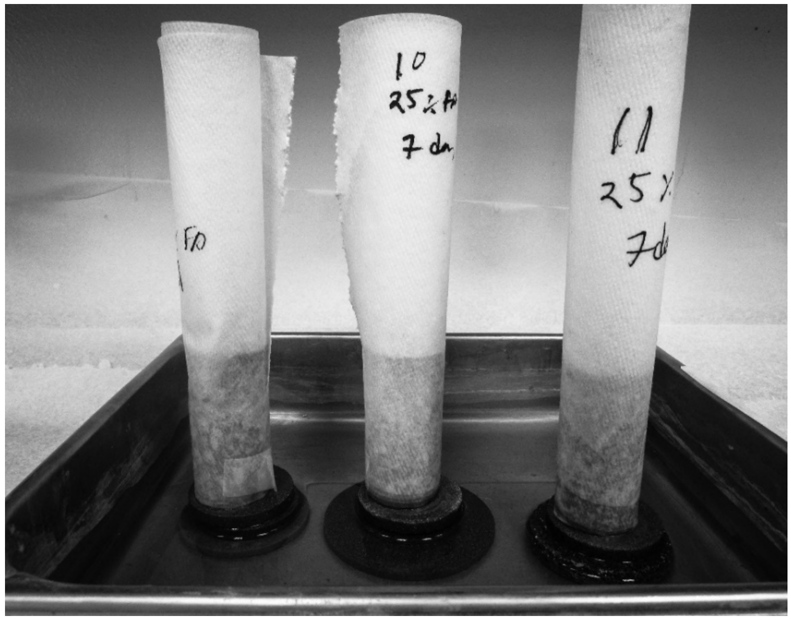


Figure 3.10: Capillary Soaking Procedure

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.

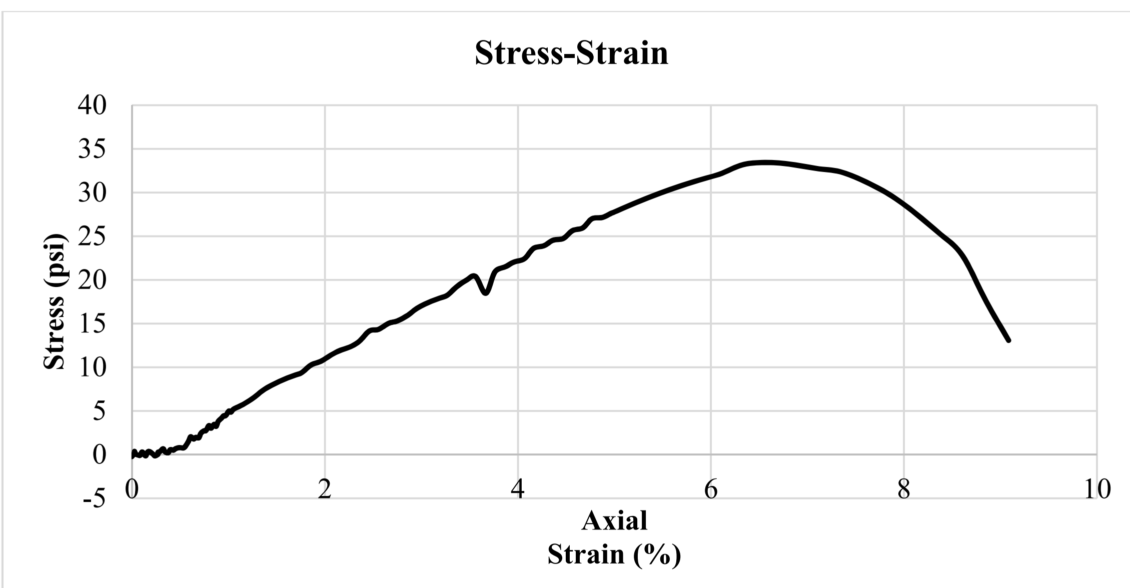


Figure 3.11: Stress-Strain Curve for the UCS Soil Sample-1, Specimen- 1 (Unsoaked)

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).

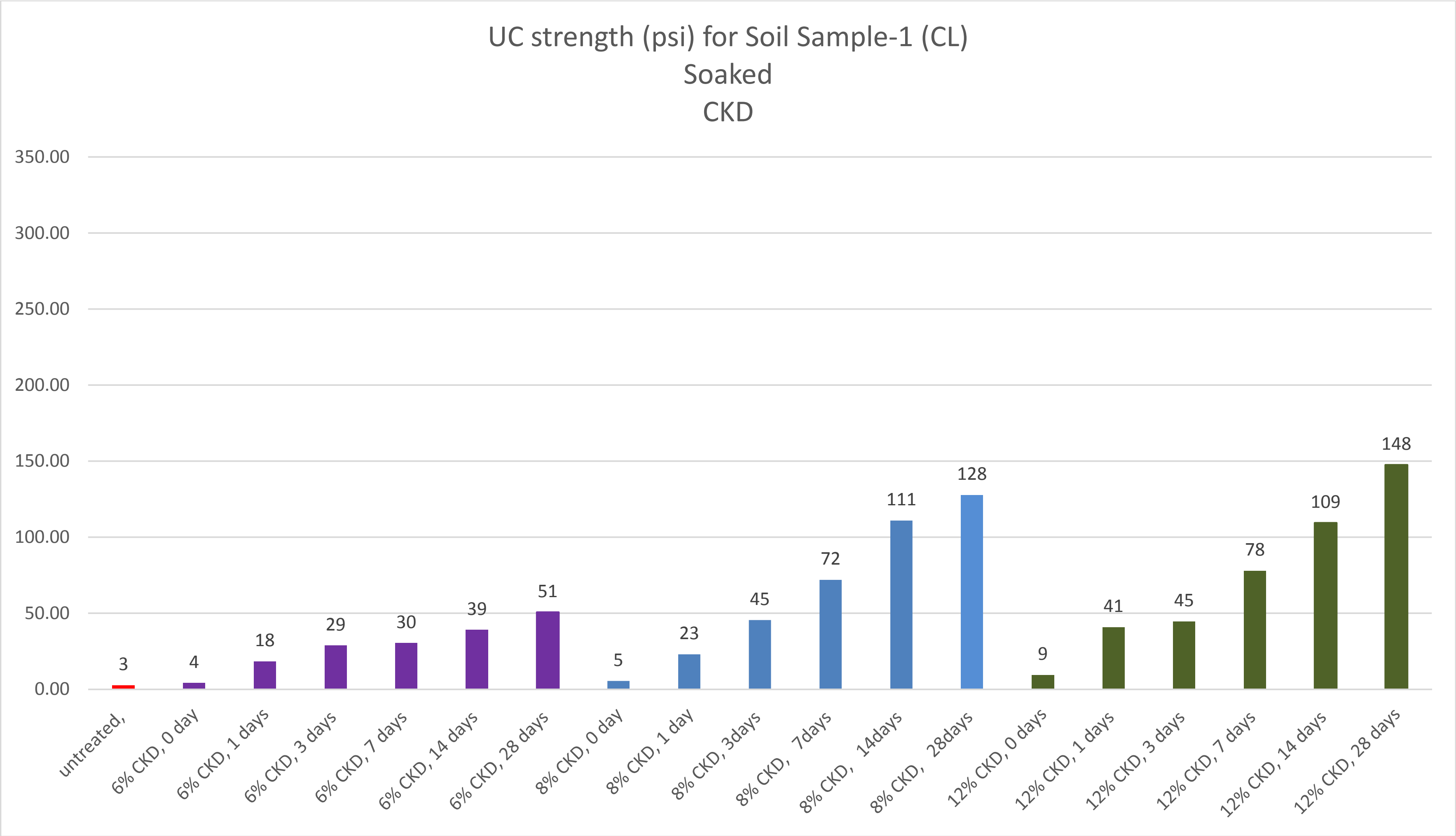


Figure 3.13: Comparison of Soaked UCS of Soil-1 (A-6) & CKD Mixes

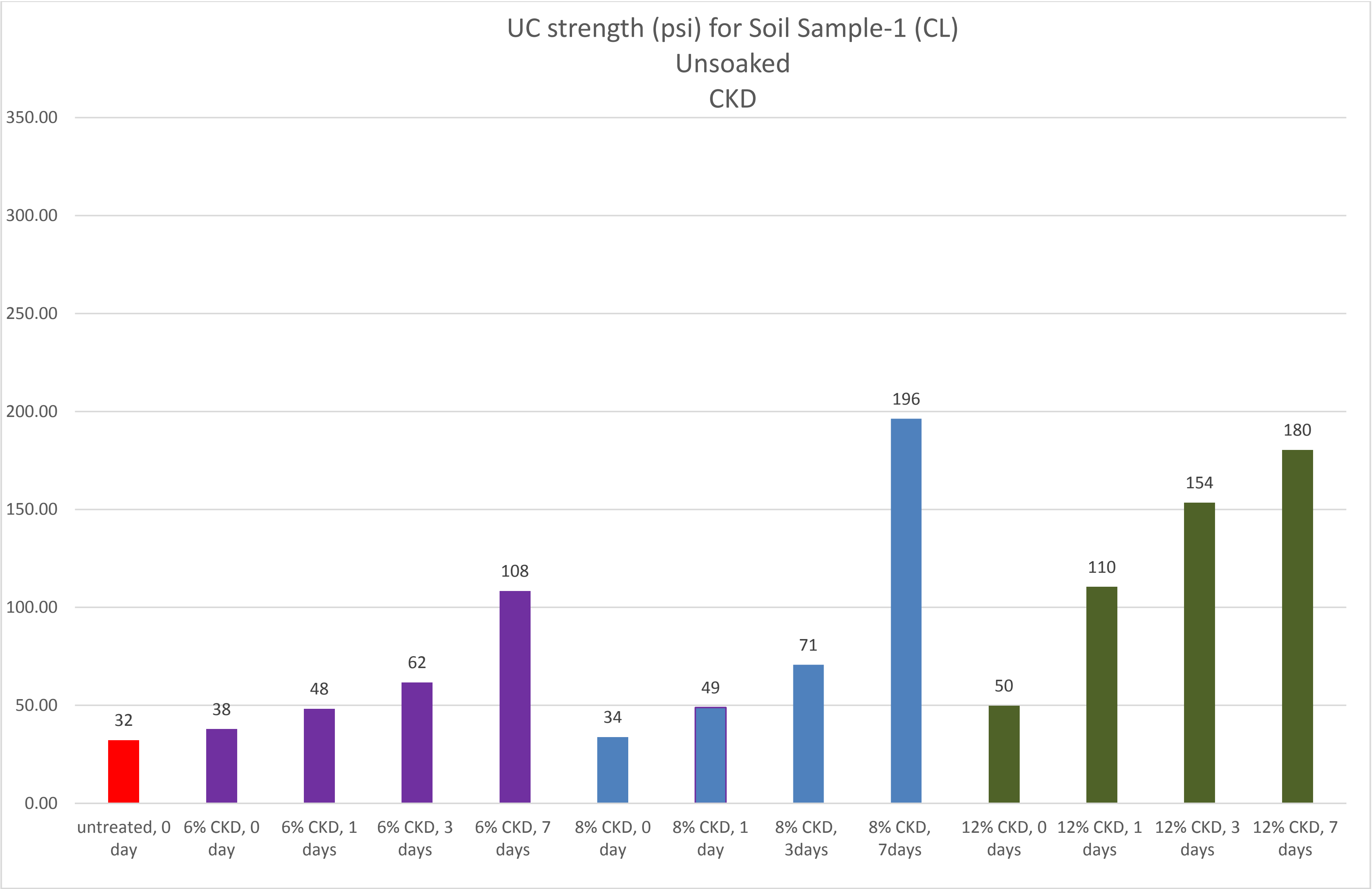


Figure 3.14: Comparison of Unsoaked UCS of Soil-1 (A-6) & CKD Mixes

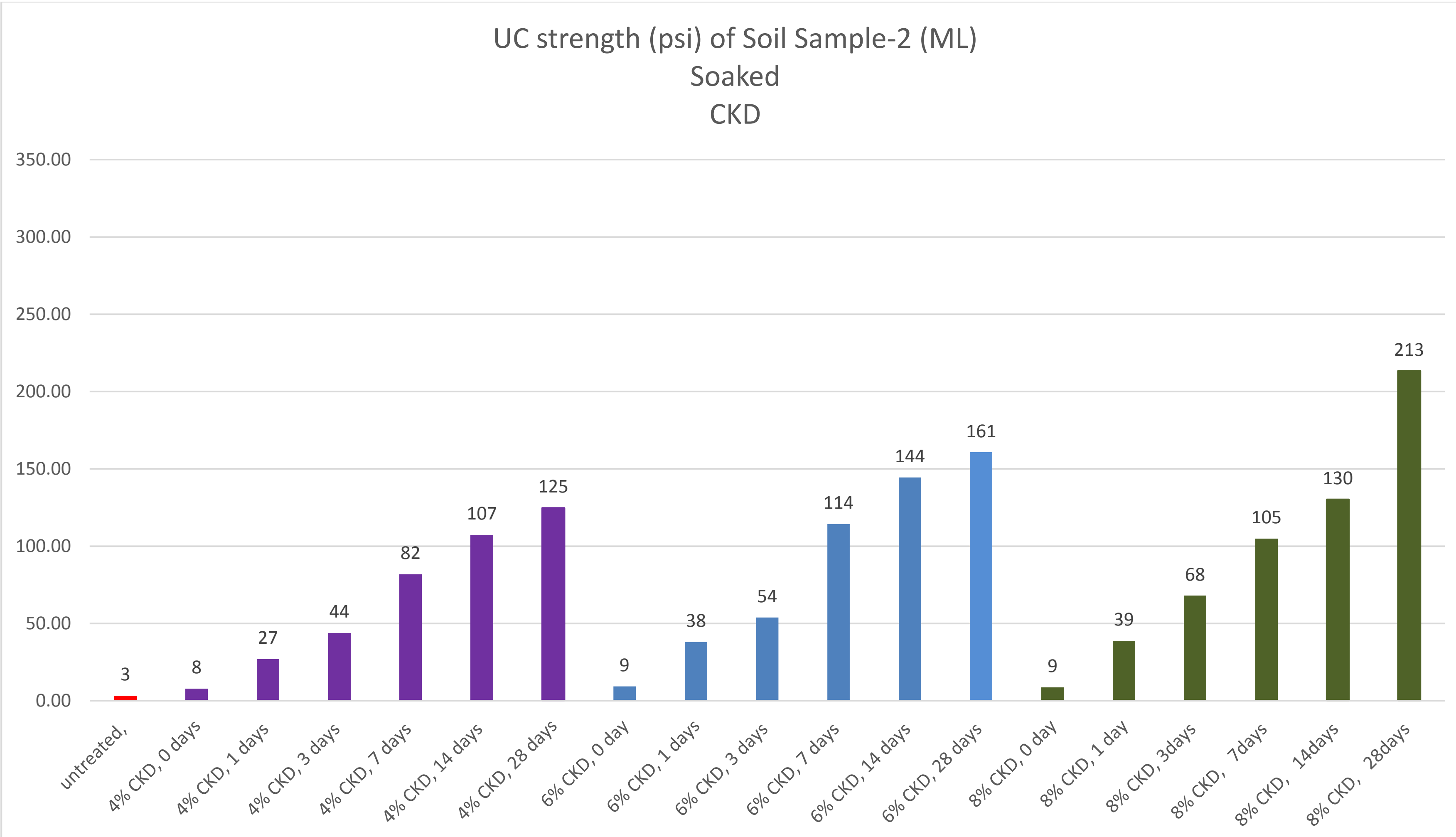


Figure 3.15: Comparison of Soaked UCS of Soil-2 (A-4) & CKD Mixes

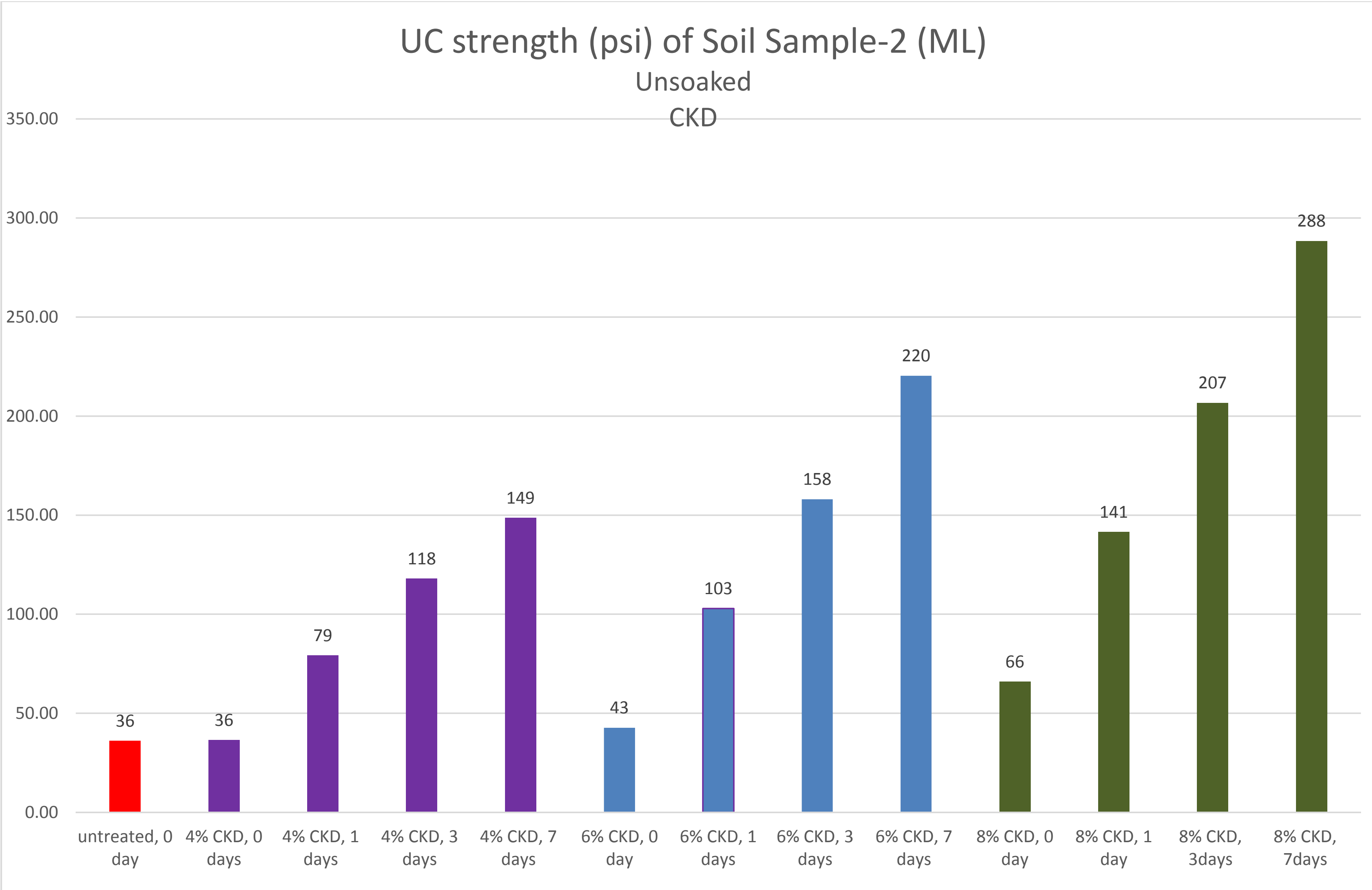


Figure 3.16: Comparison of Unsoaked UCS of Soil-2 (A-4) & CKD Mixes

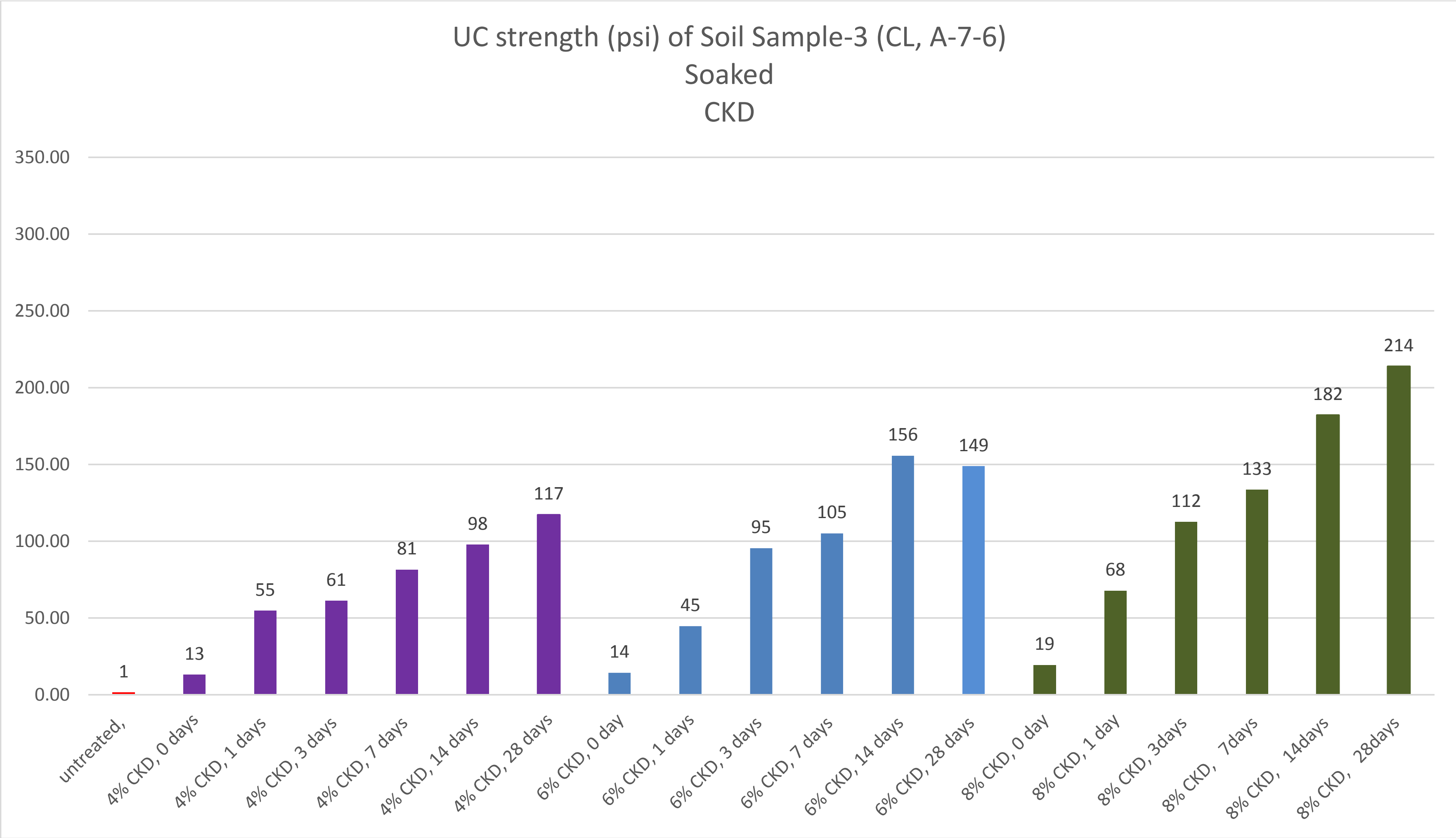


Figure 3.17: Comparison of Soaked UCS of Soil-3 (A-7-6) & CKD Mixes

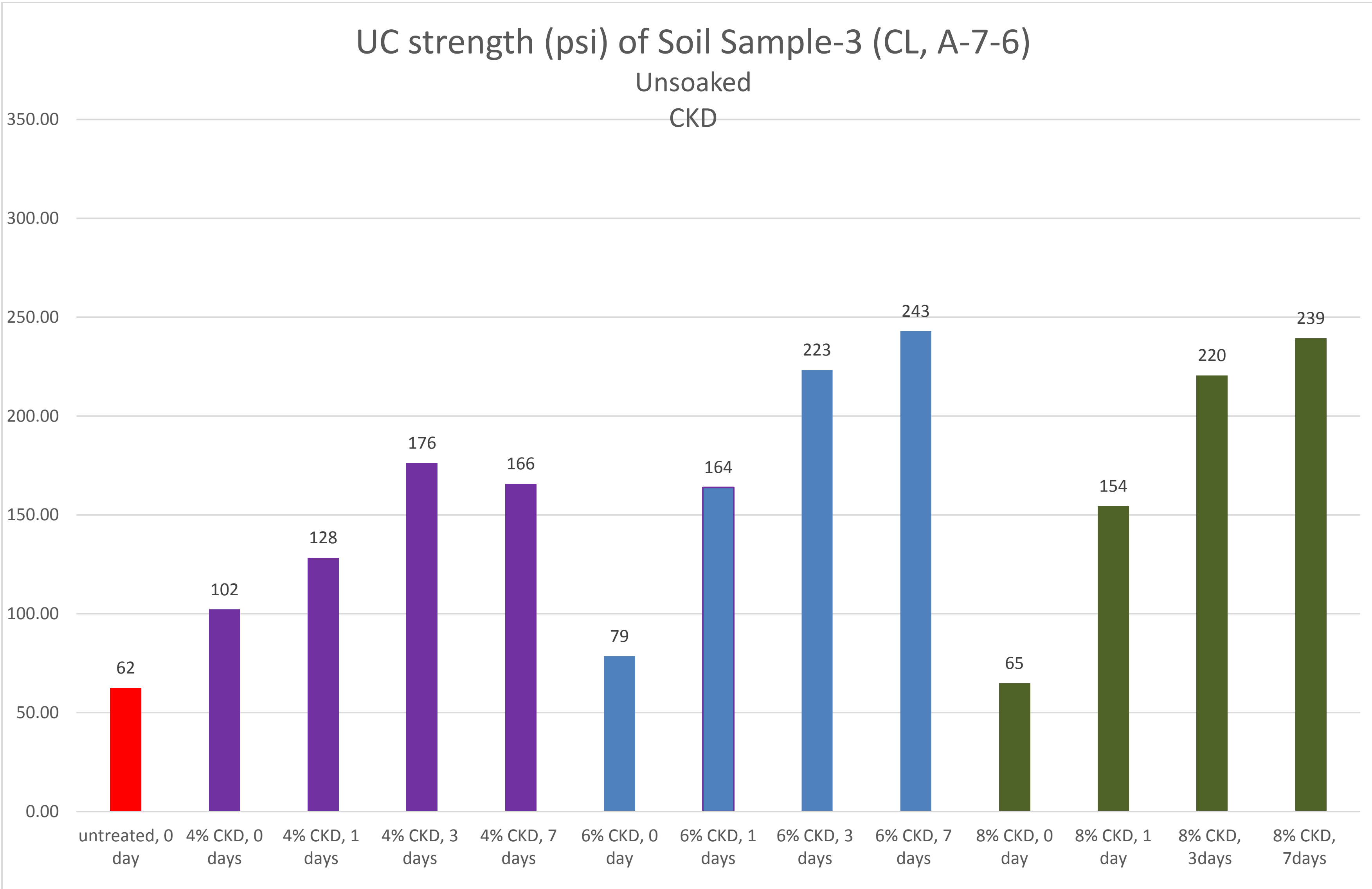


Figure 3.18: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & CKD Mixes

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.

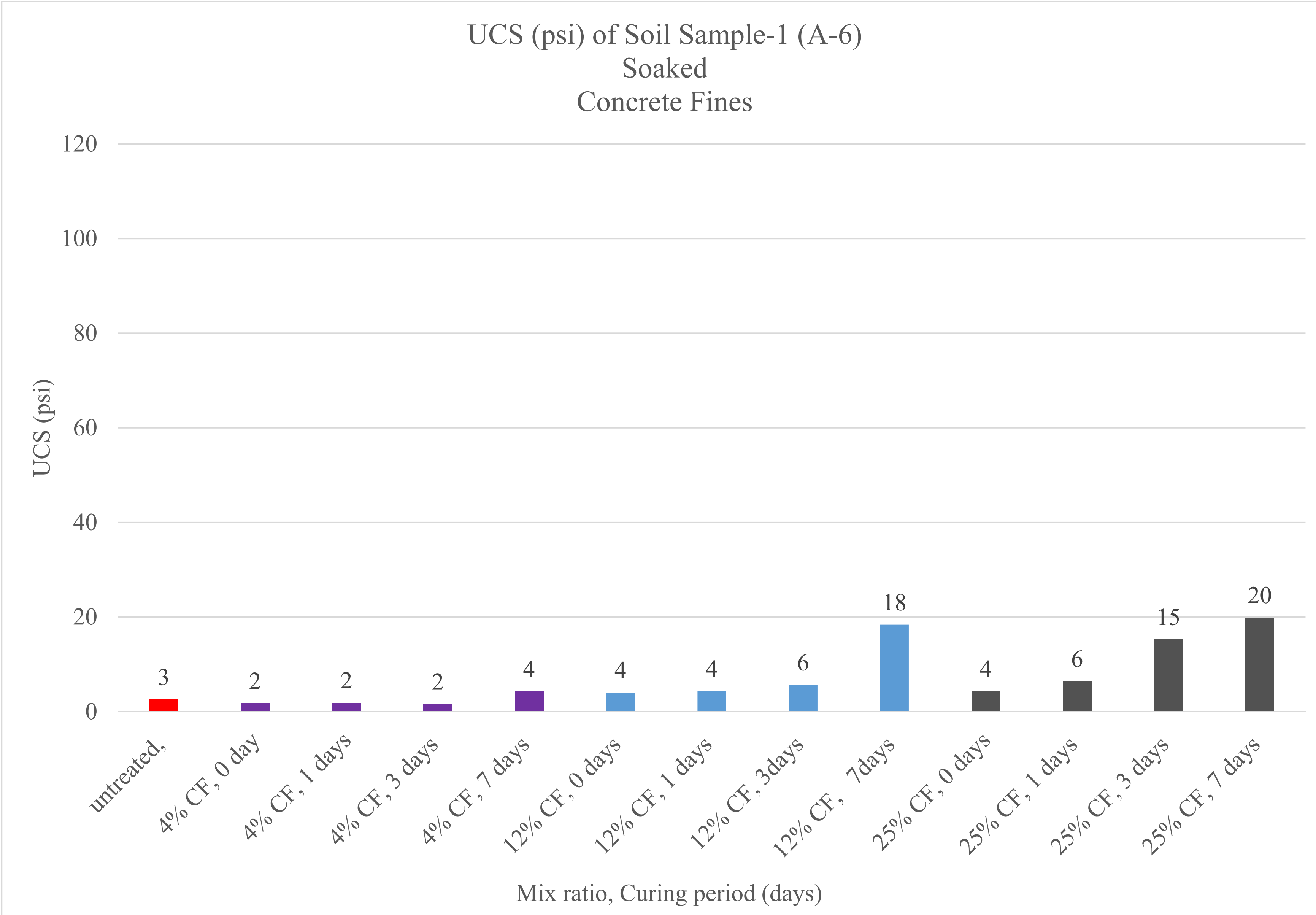


Figure 3.19: Comparison of Soaked UCS of Soil-1 (A-6) & CF Mixes

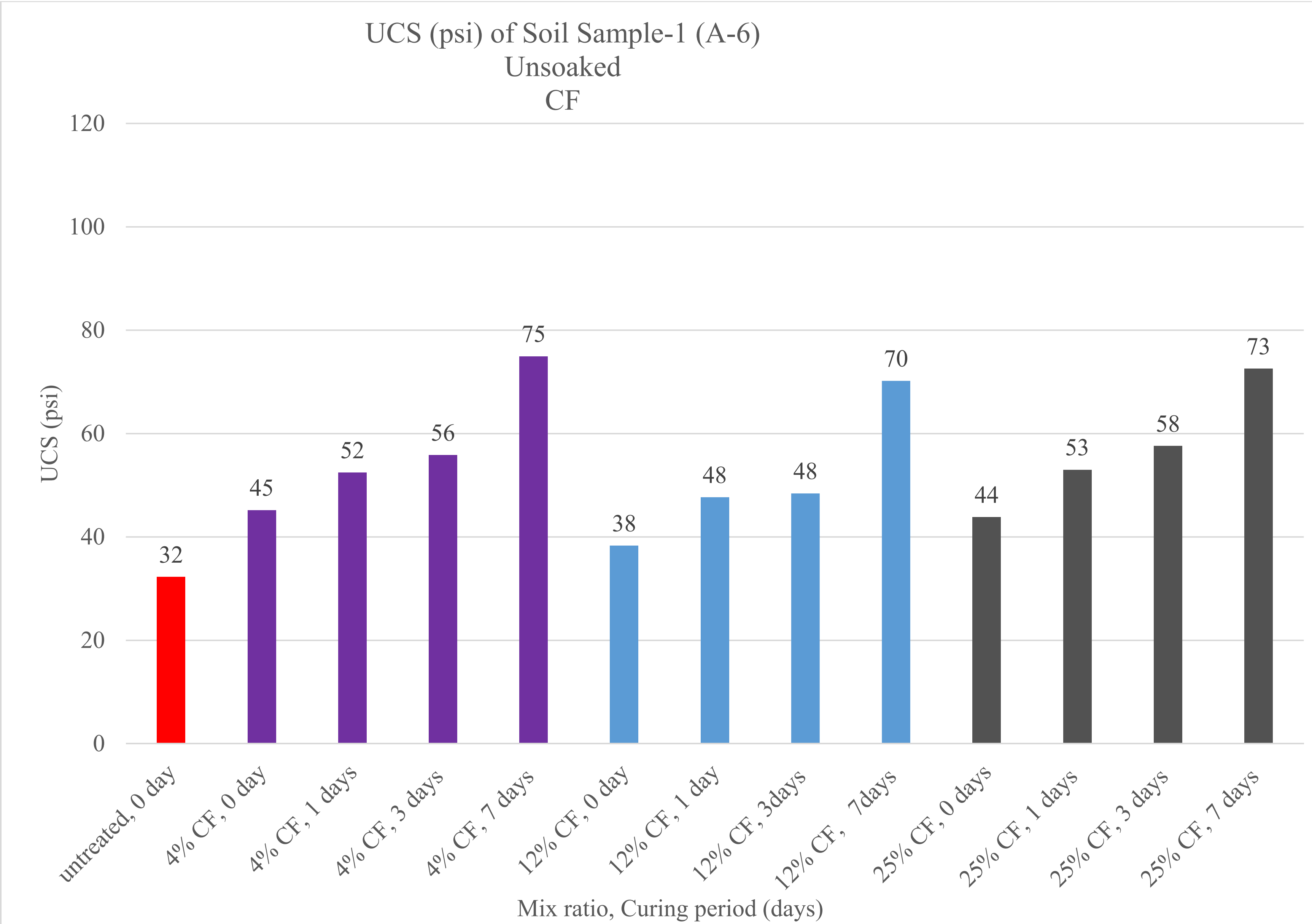


Figure 3.20: Comparison of Unsoaked UCS of Soil-1 (A-6) & CF Mixes

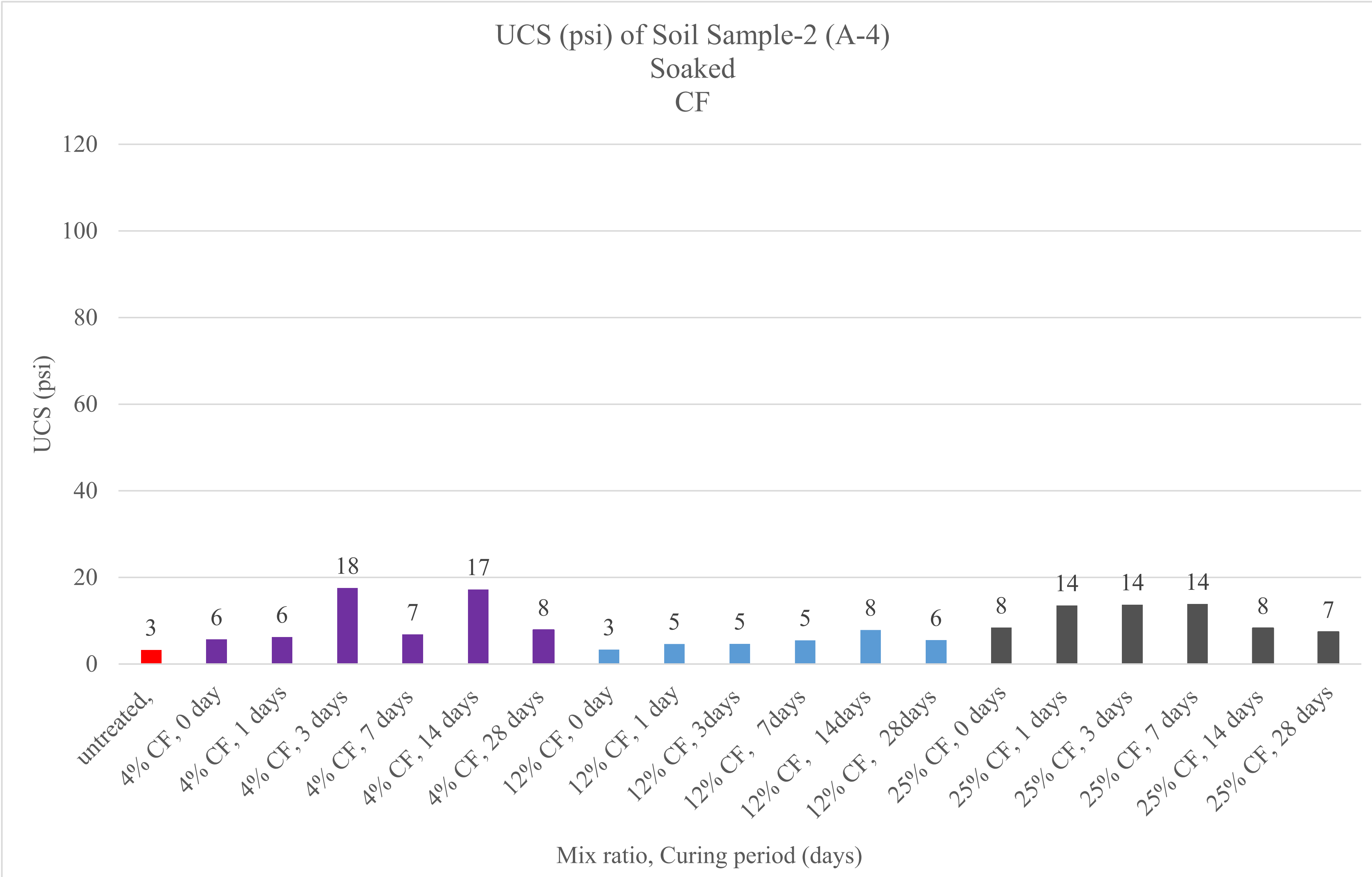


Figure 3.21: Comparison of Soaked UCS of Soil-2 (A-4) & CF Mixes

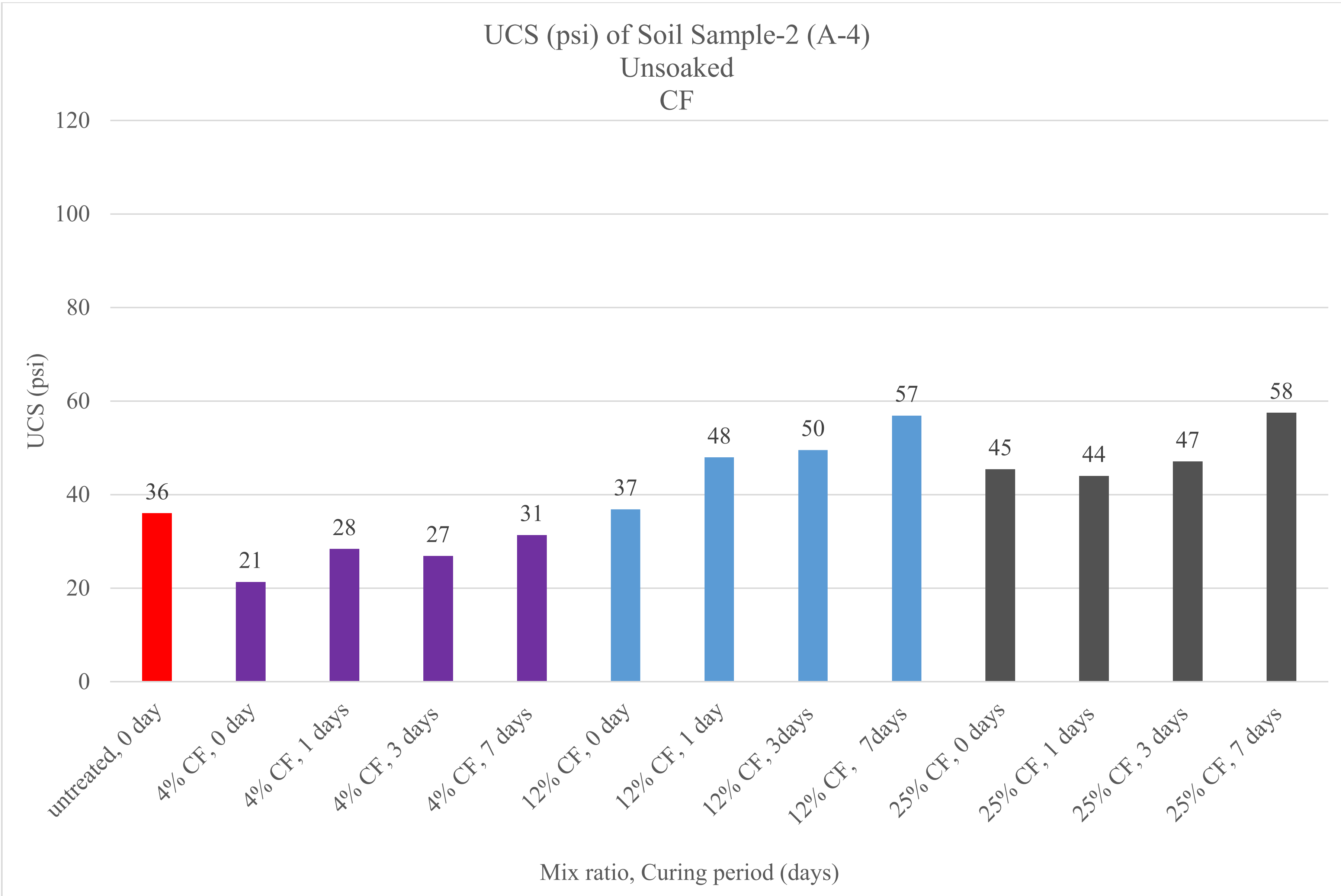


Figure 3.22: Comparison of Unsoaked UCS of Soil-2 (A-4) & CF Mixes

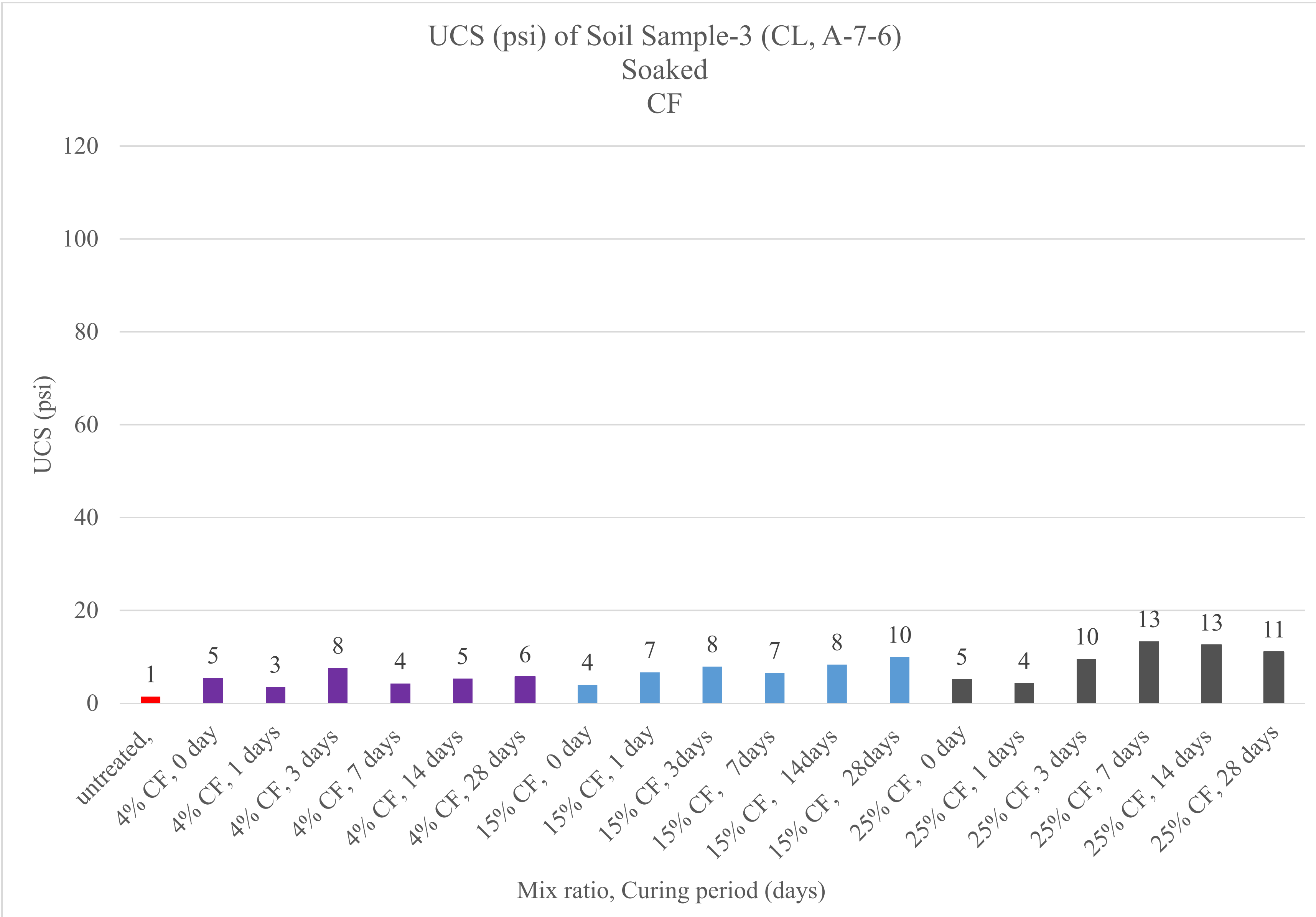


Figure 3.23: Comparison of Soaked UCS of Soil-3 (A-7-6) & CF Mixes

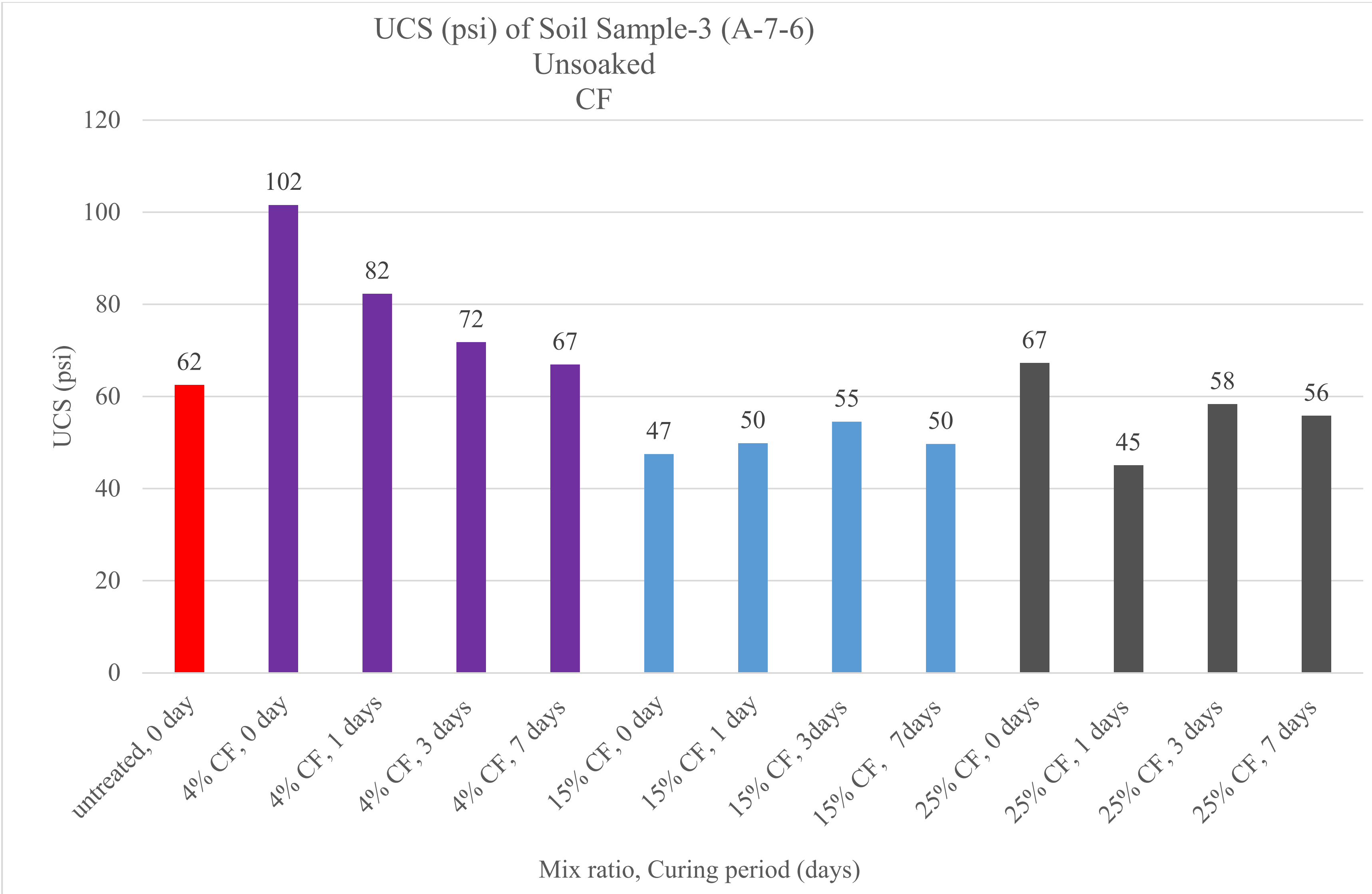


Figure 3.24: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & CF Mixes

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.

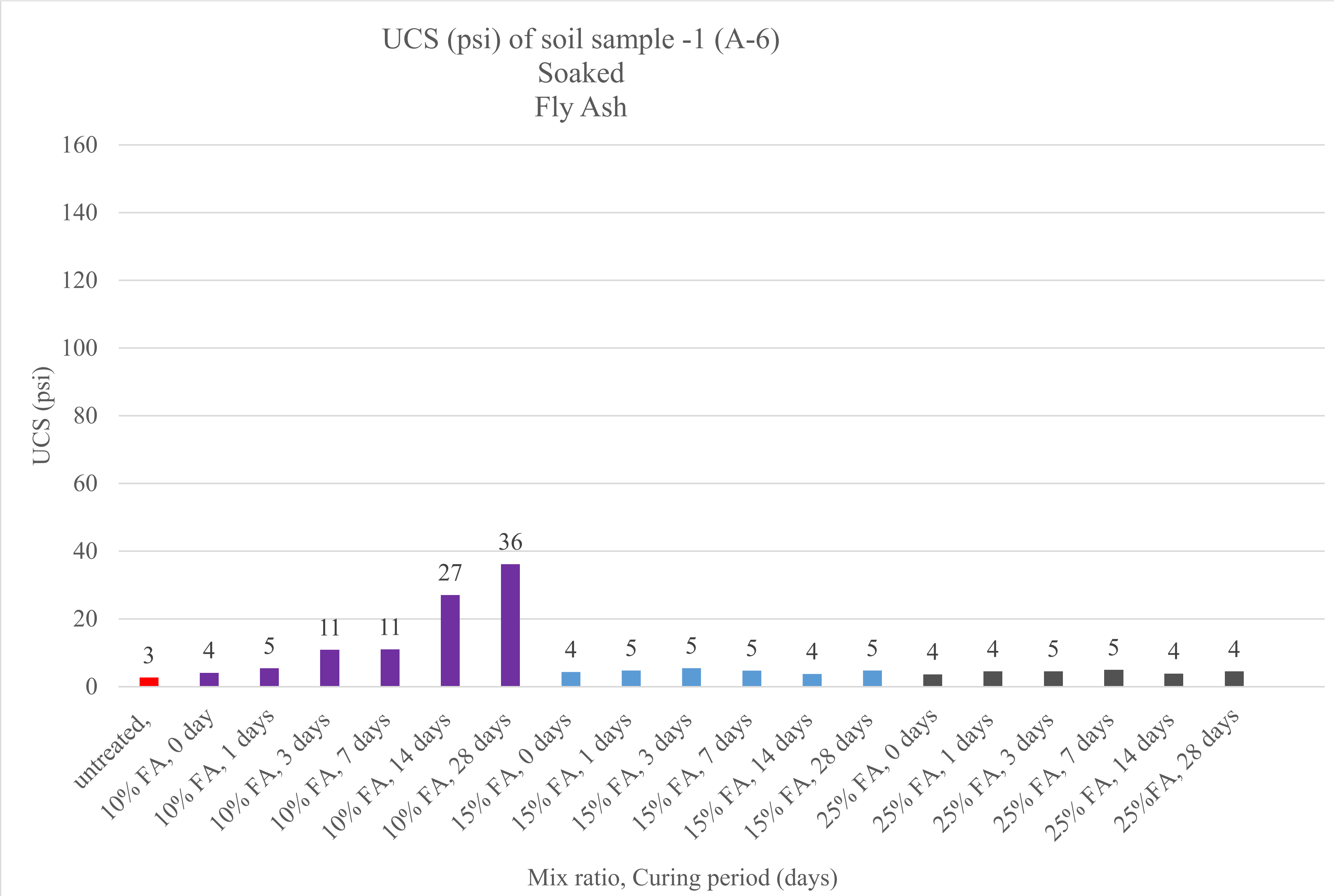


Figure 3.25: Comparison of Soaked UCS of Soil-1 (A-6) & FA Mixes

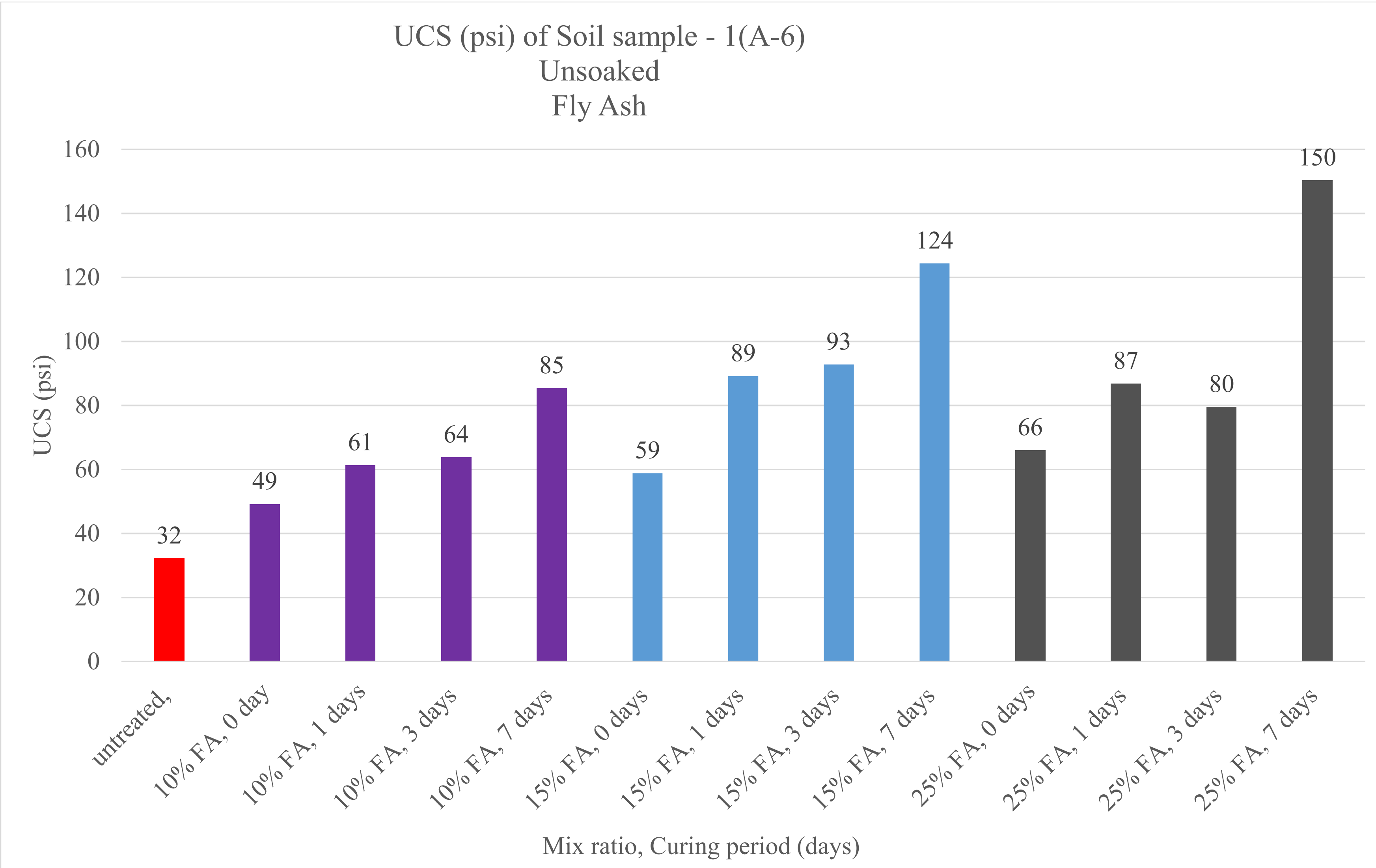


Figure 3.26: Comparison of Unsoaked UCS of Soil-1 (A-6) & FA Mixes

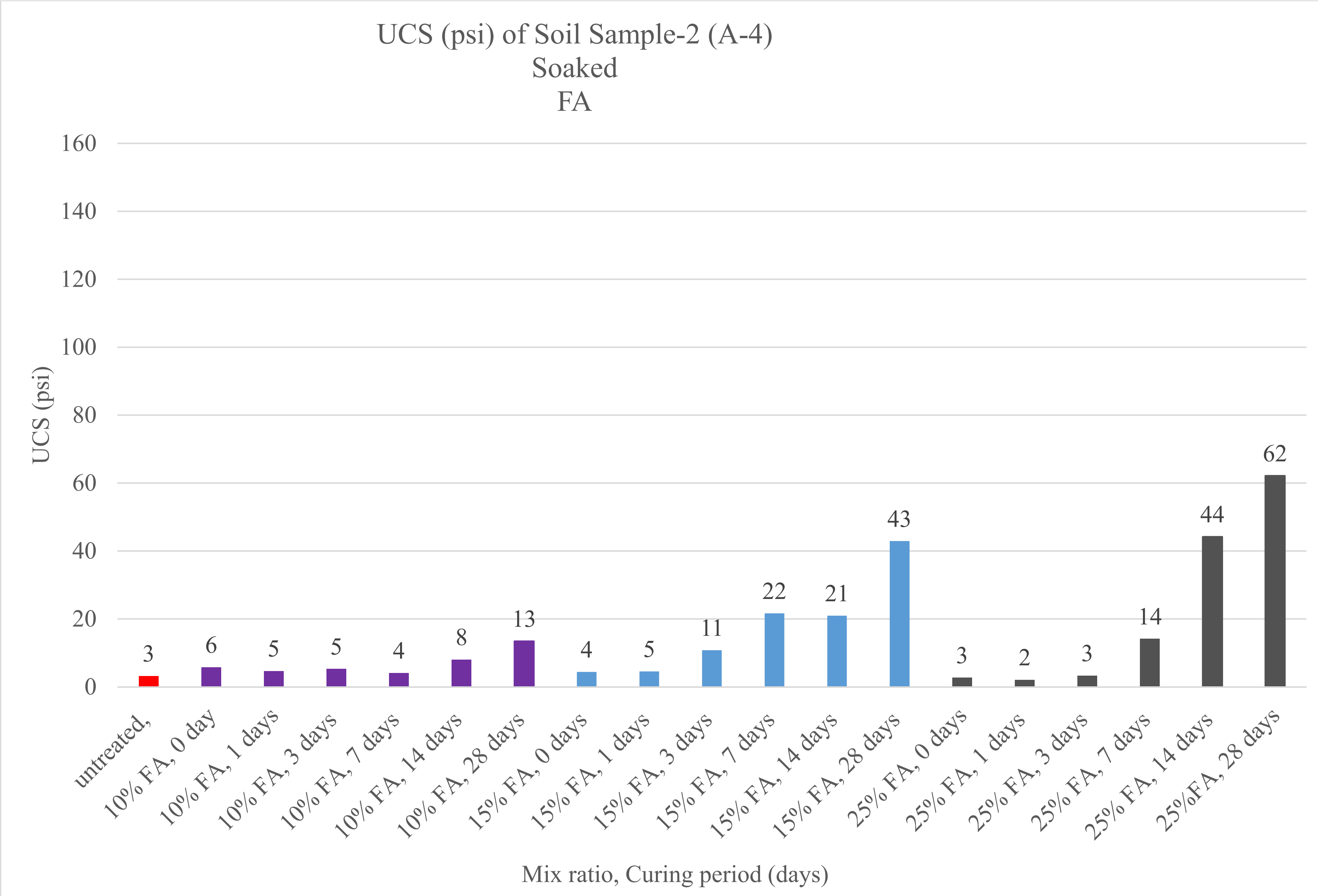


Figure 3.27: Comparison of Soaked UCS of Soil-2(A-4) & FA Mixes

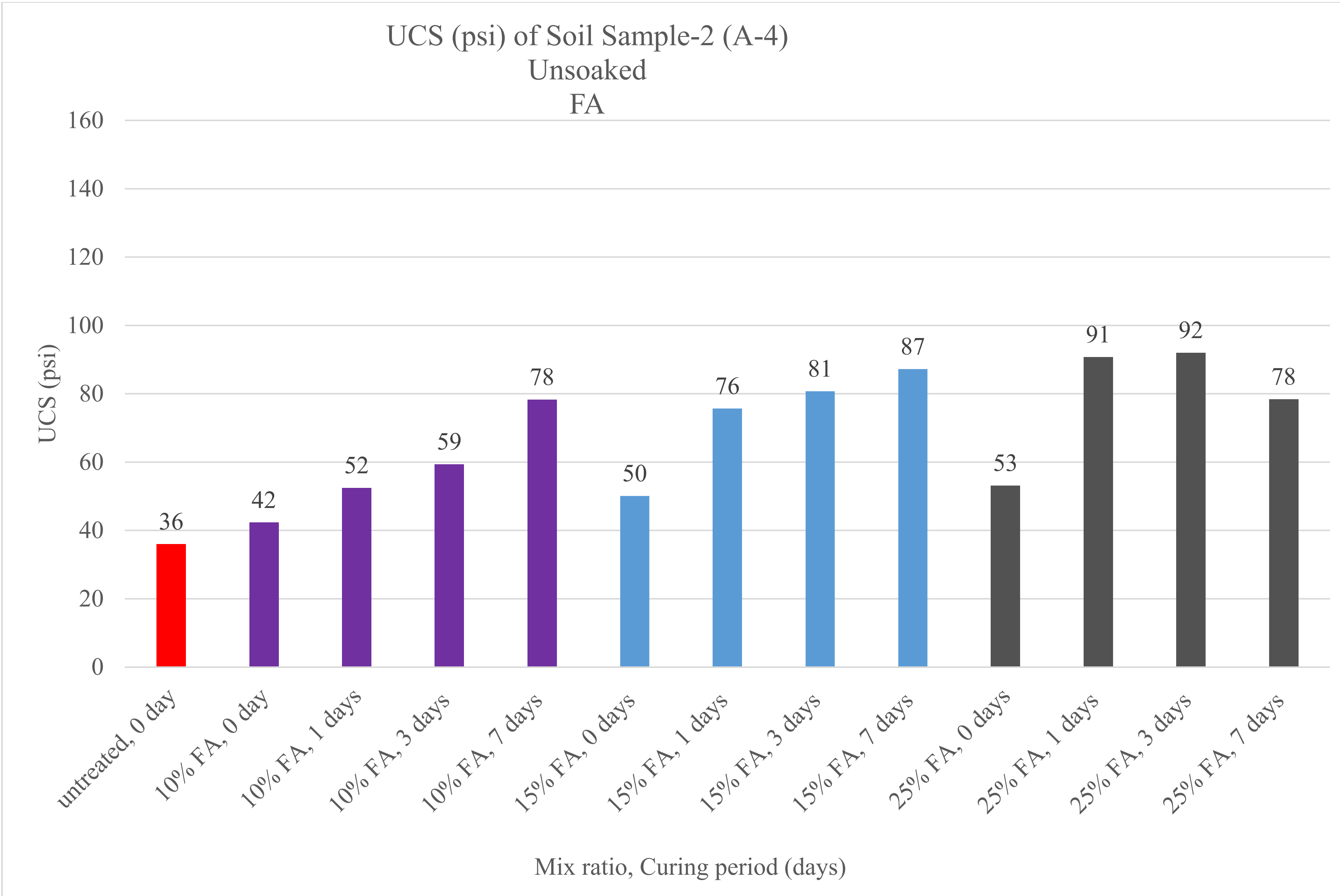


Figure 3.28: Comparison of Unsoaked UCS of Soil-2 (A-4) & FA Mixes

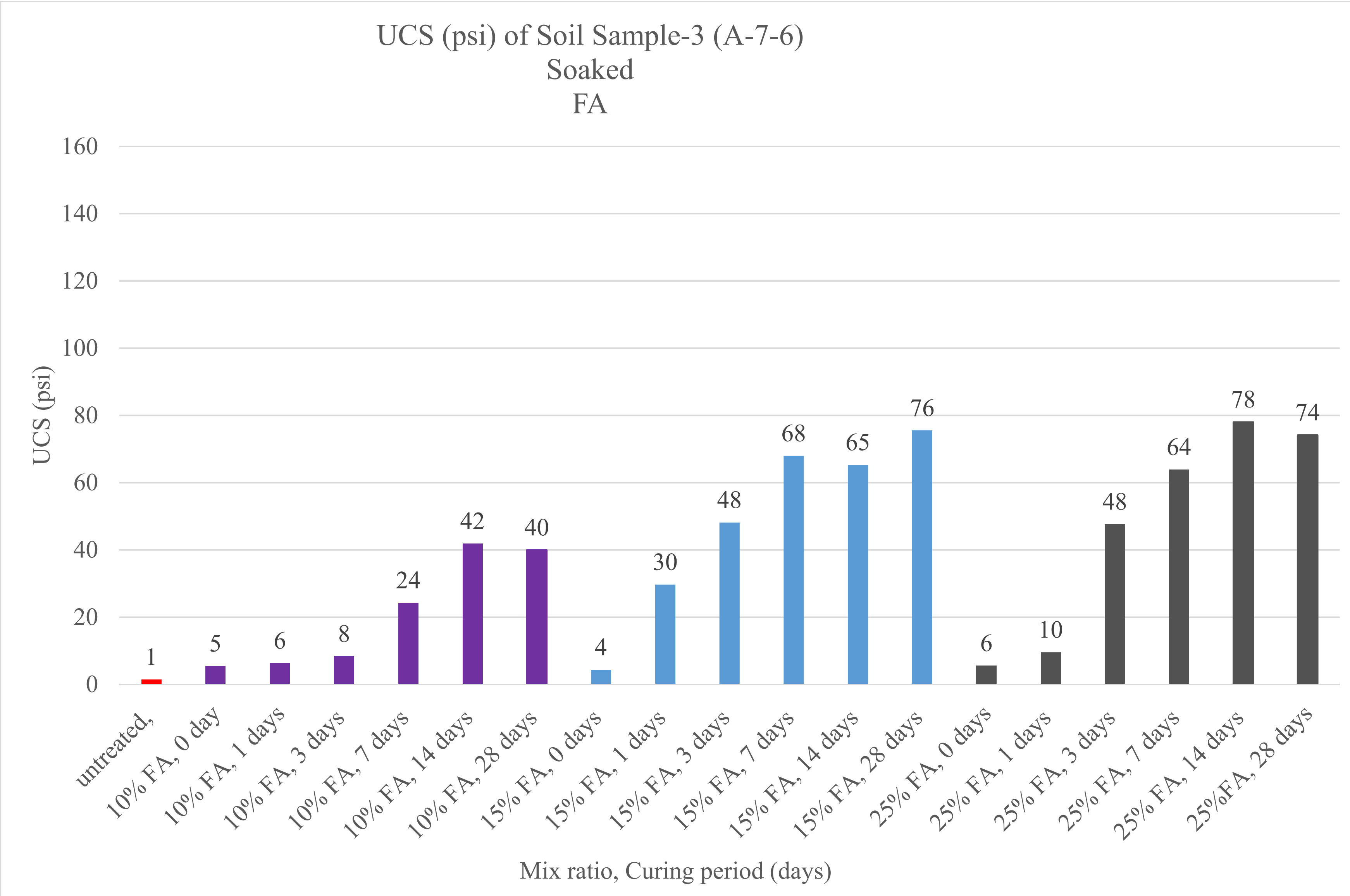


Figure 3.29: Comparison of Soaked UCS of Soil-3 (A-7-6) & FA Mixes

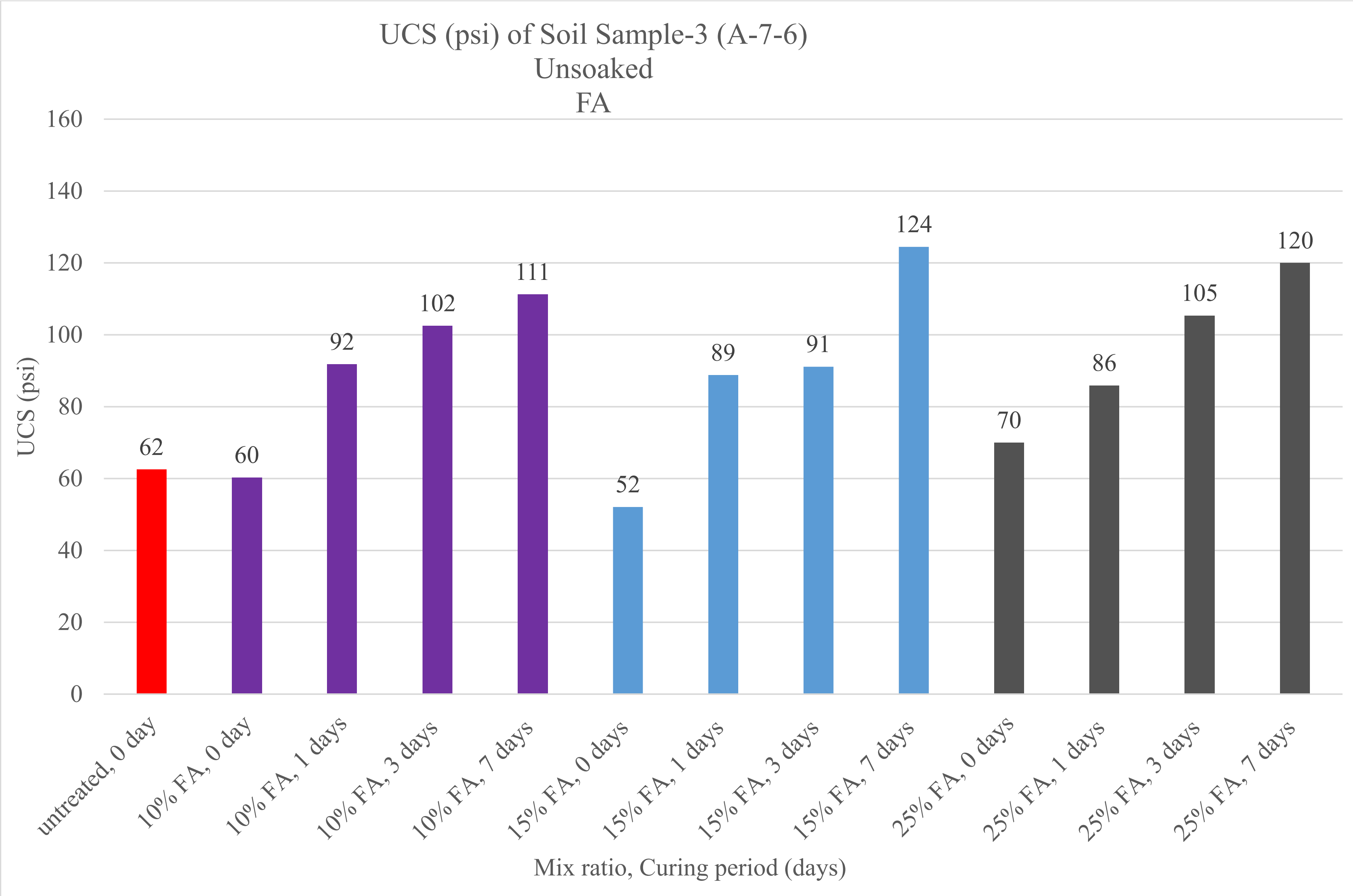


Figure 3.30: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & FA Mixes

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.

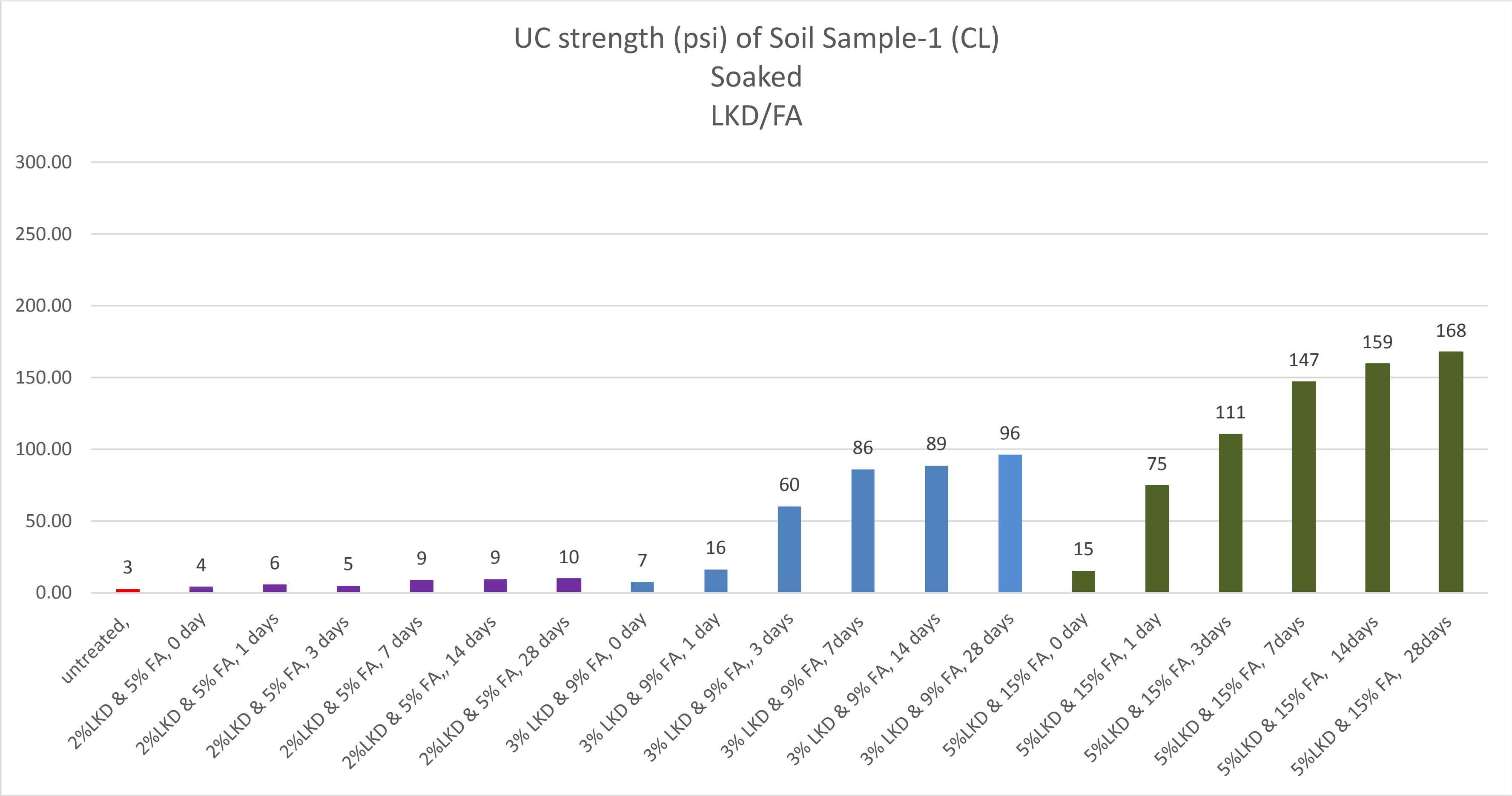


Figure 3.31: Comparison of Soaked UCS of Soil-1 (A-6) & LKD/FA Mixes

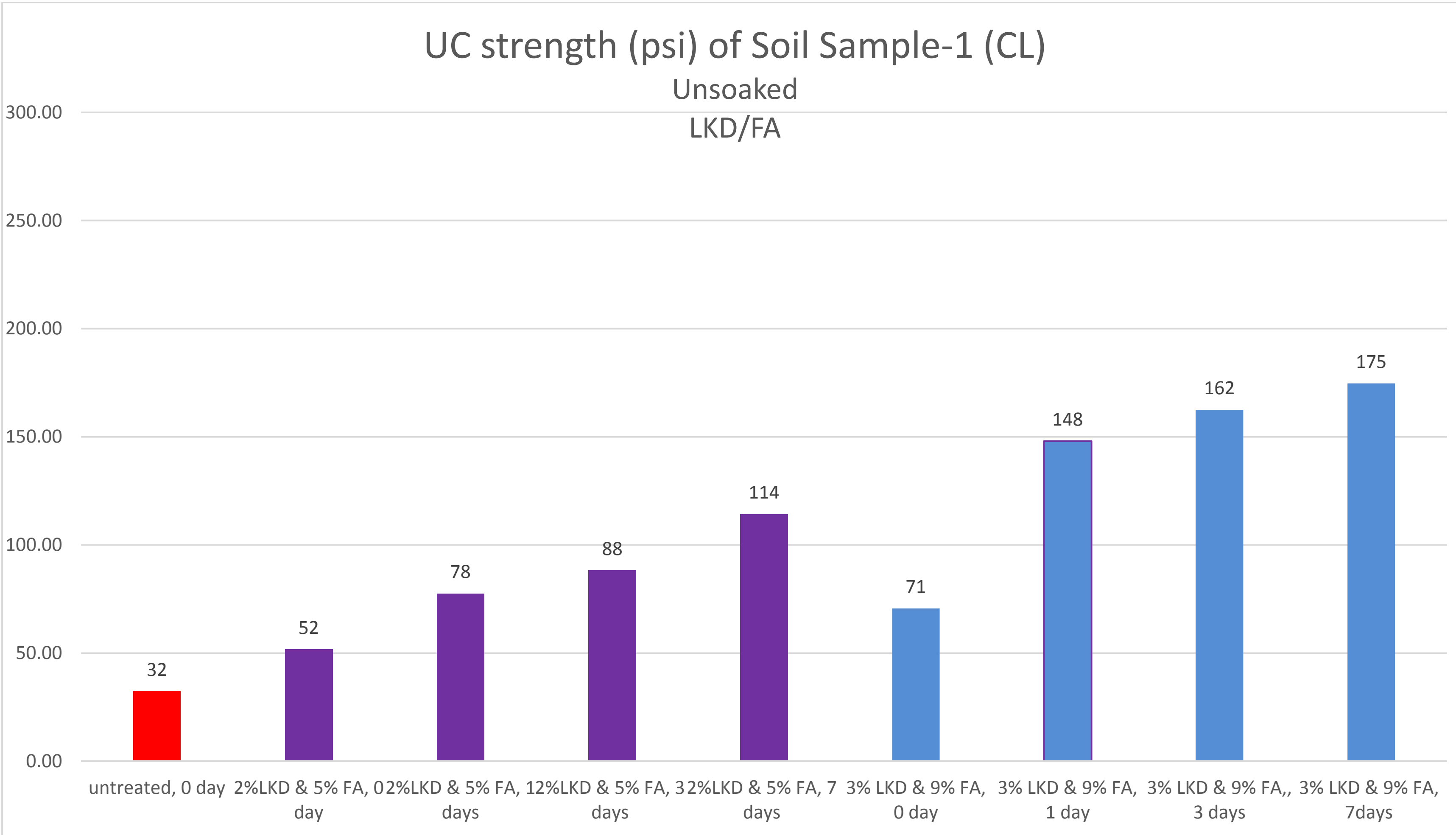


Figure 3.32: Comparison of Unsoaked UCS of Soil-1 (A-6) & LKD/FA Mixes

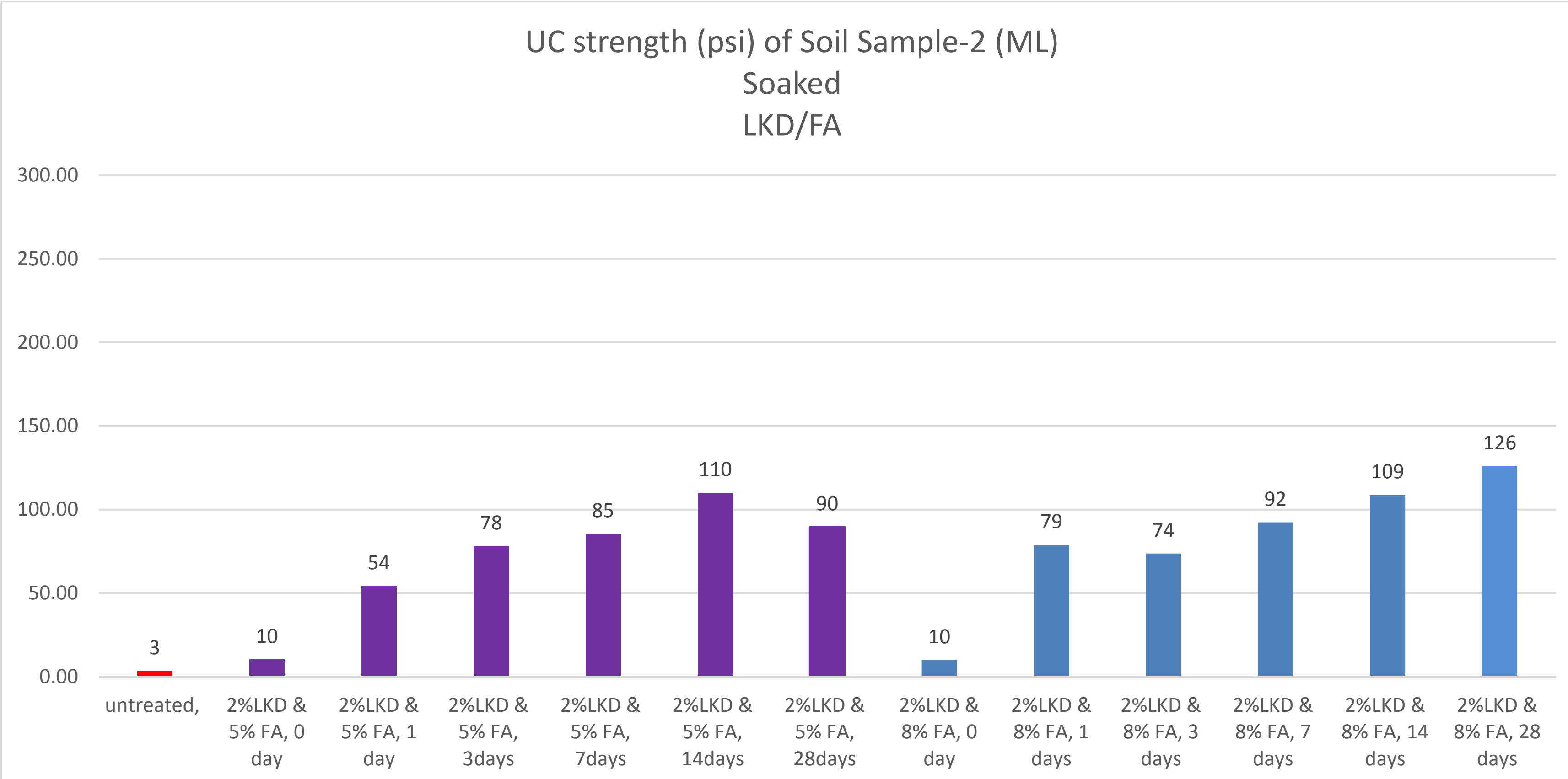


Figure 3.33: Comparison of Soaked UCS of Soil-2 (A-4) & LKD/FA Mixes

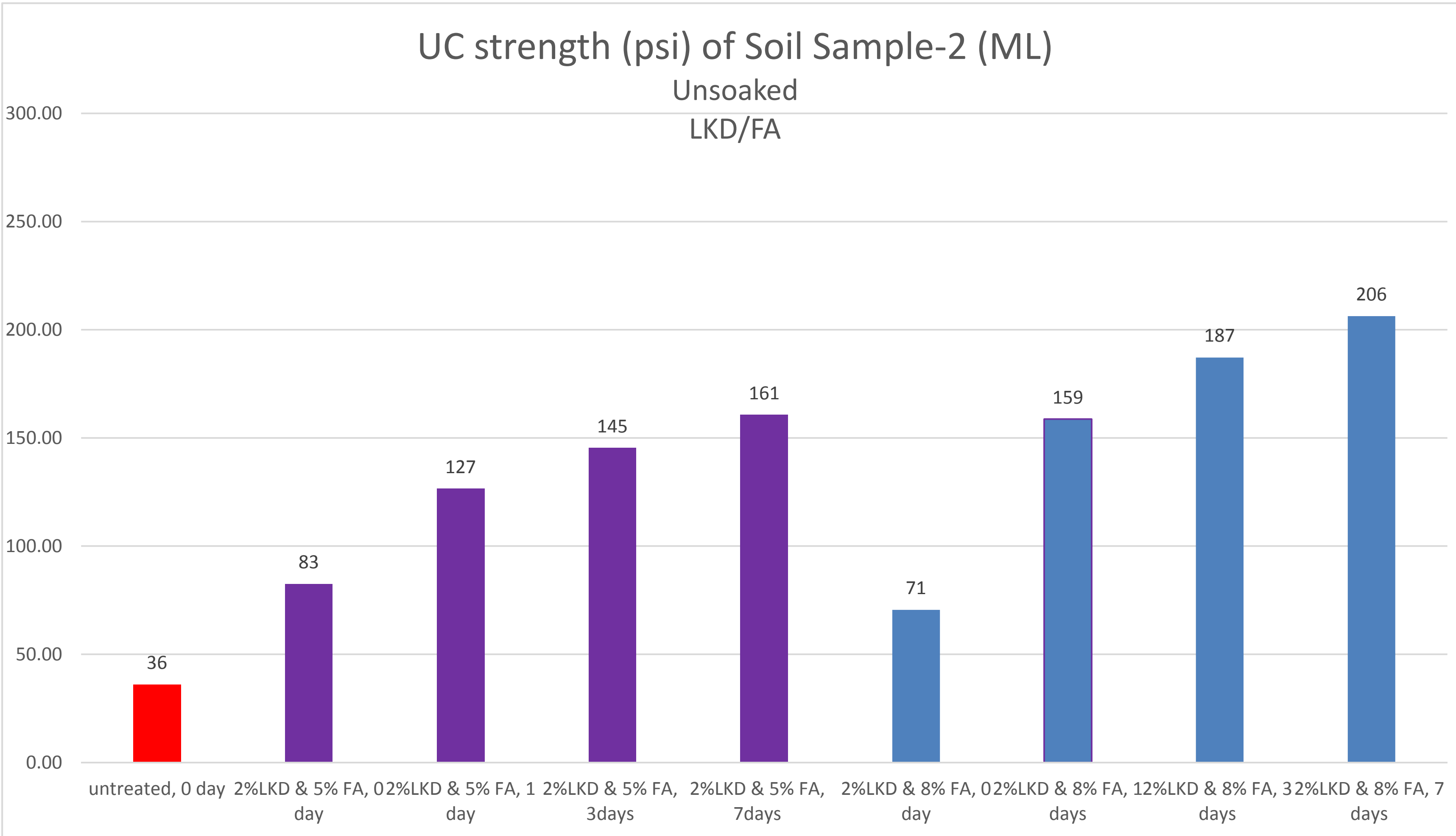


Figure 3.34: Comparison of Unsoaked UCS of Soil-2 (A-4) & LKD/FA Mixes

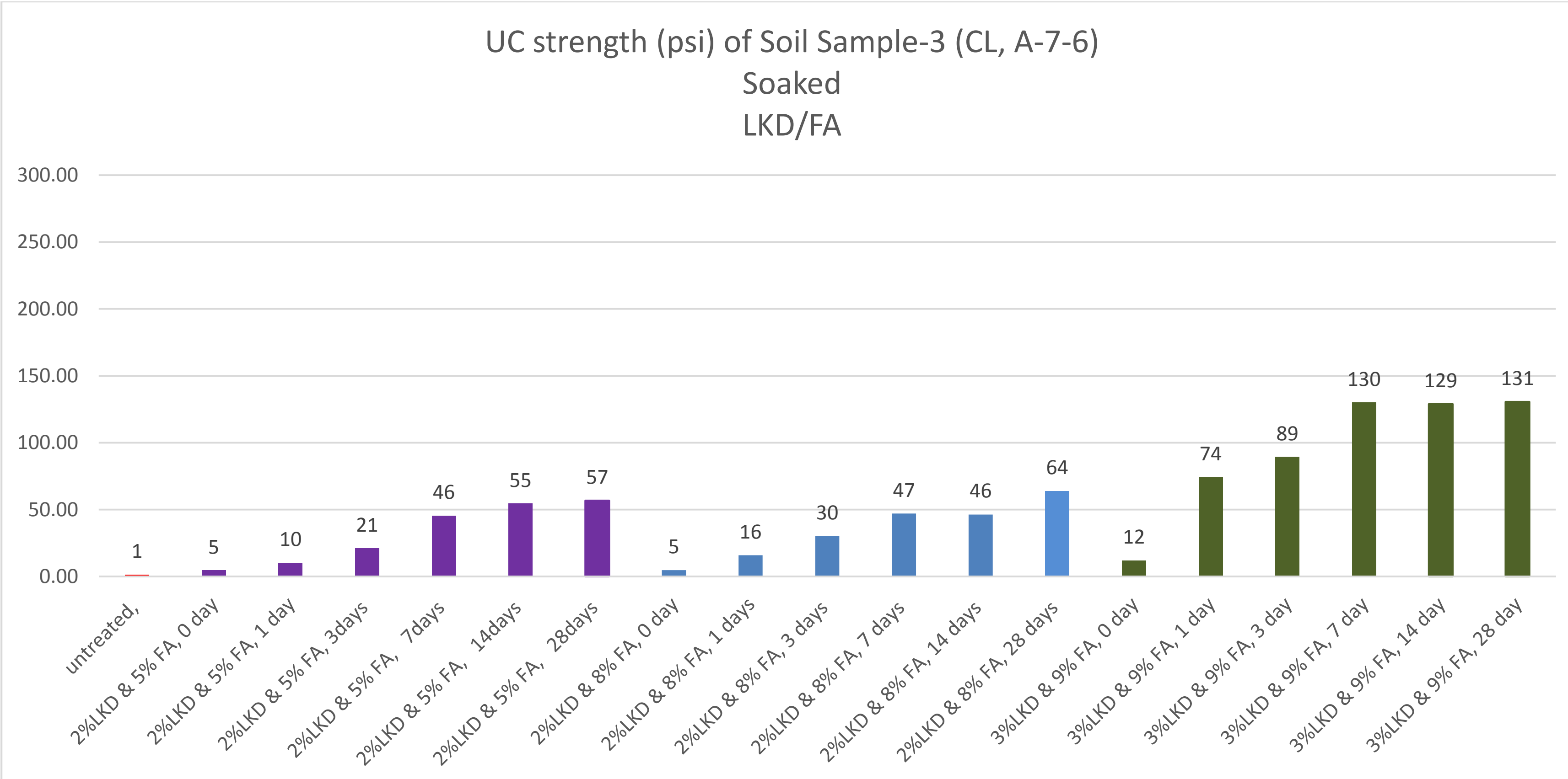


Figure 3.35: Comparison of Soaked UCS of Soil-3 (A-7-6) & LKD/FA Mixes

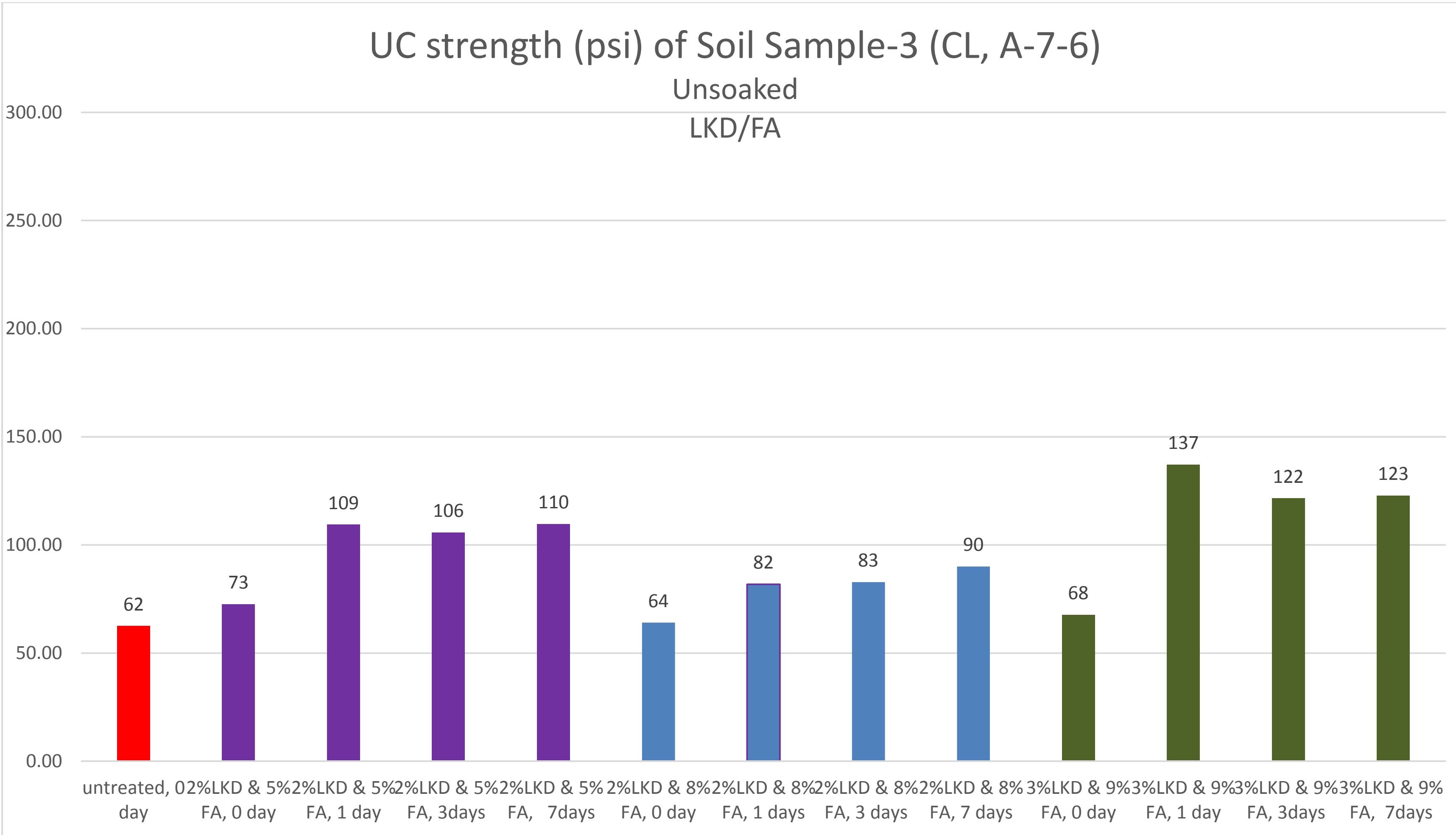


Figure 3.36: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & LKD/FA Mixes

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:

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

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.

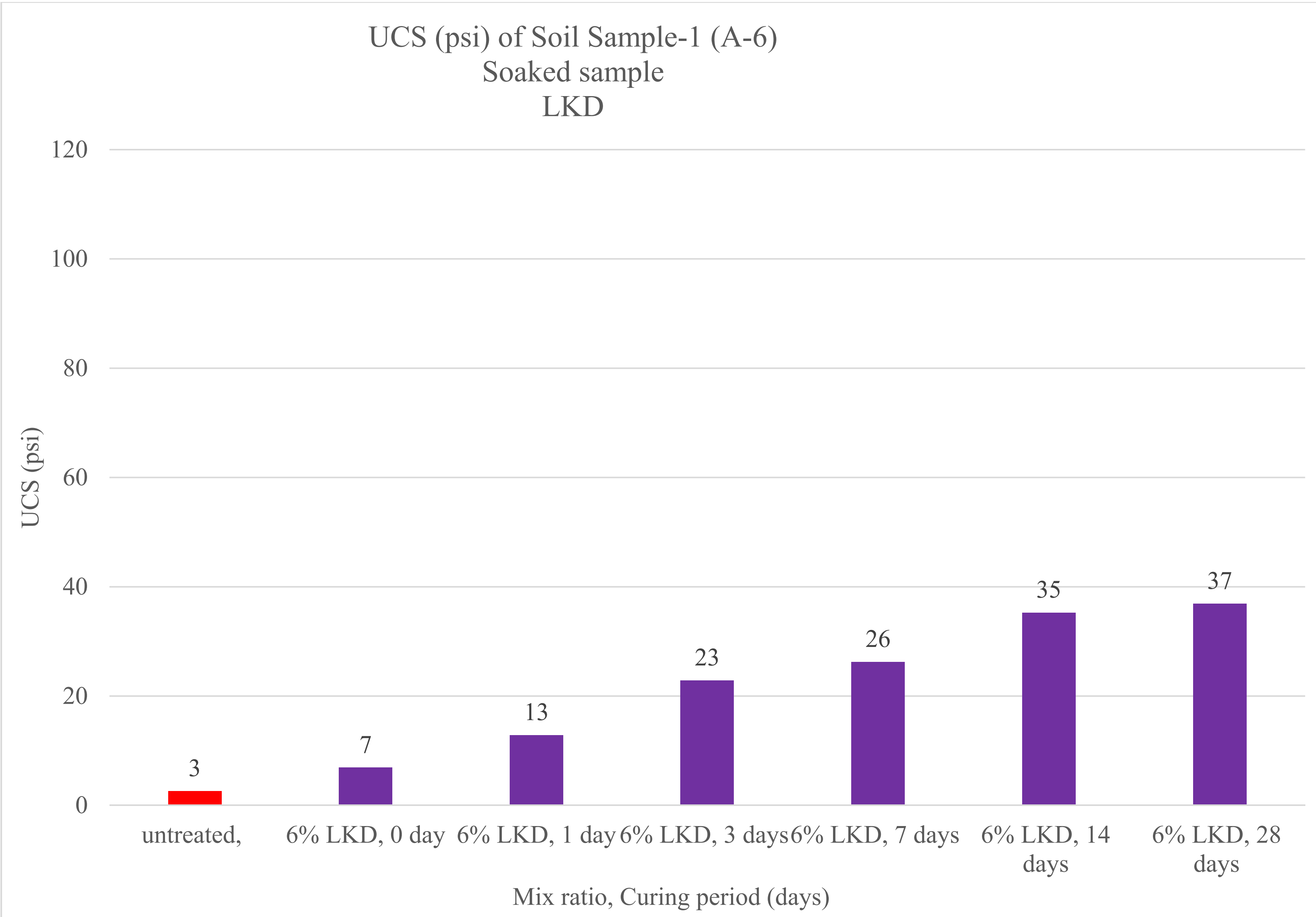


Figure 3.37: Comparison of Soaked UCS of Soil-1 (A-6) & 6% LKD Mix

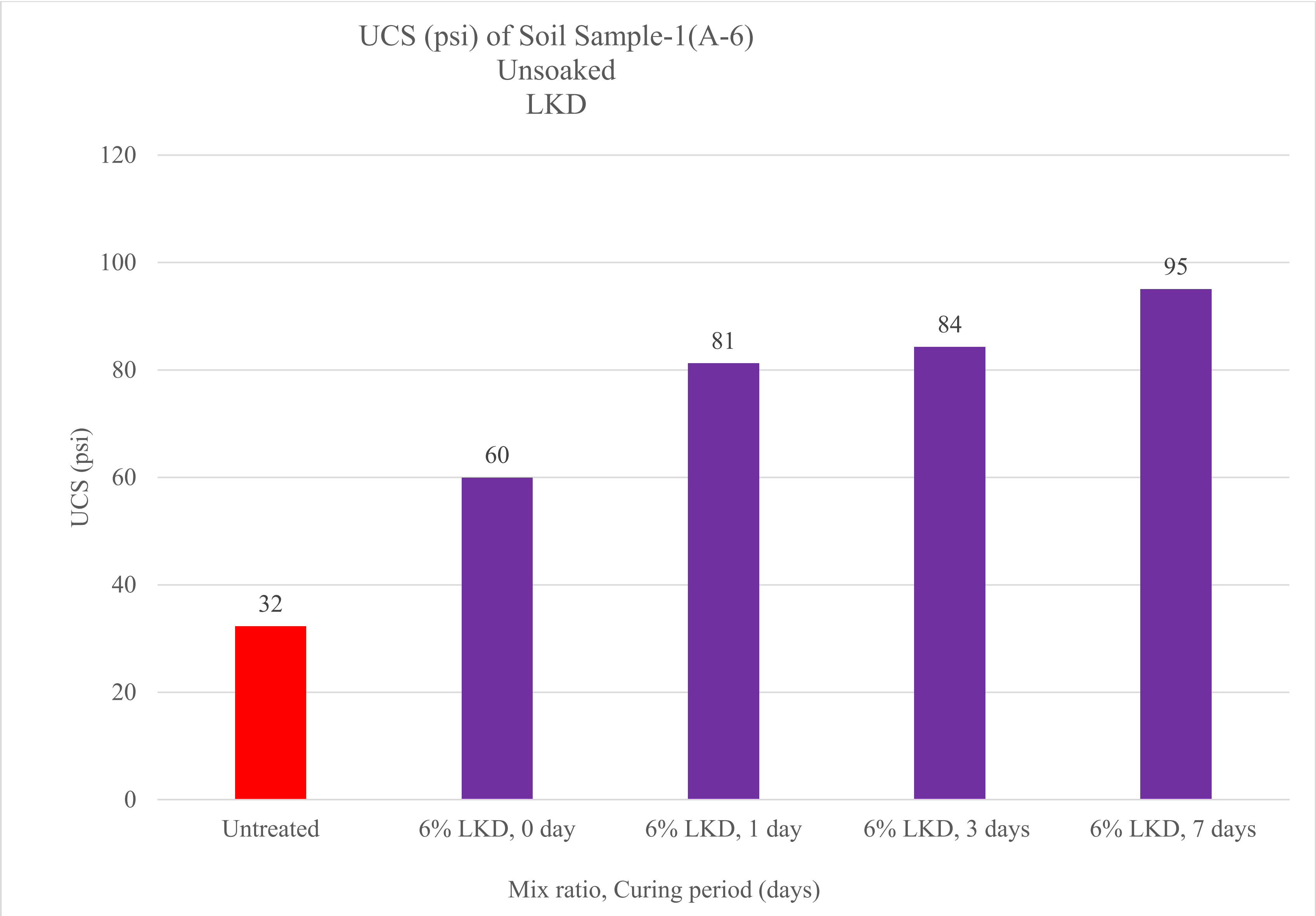


Figure 3.38: Comparison of Unsoaked UCS of Soil-1 (A-6) & 6% LKD Mix

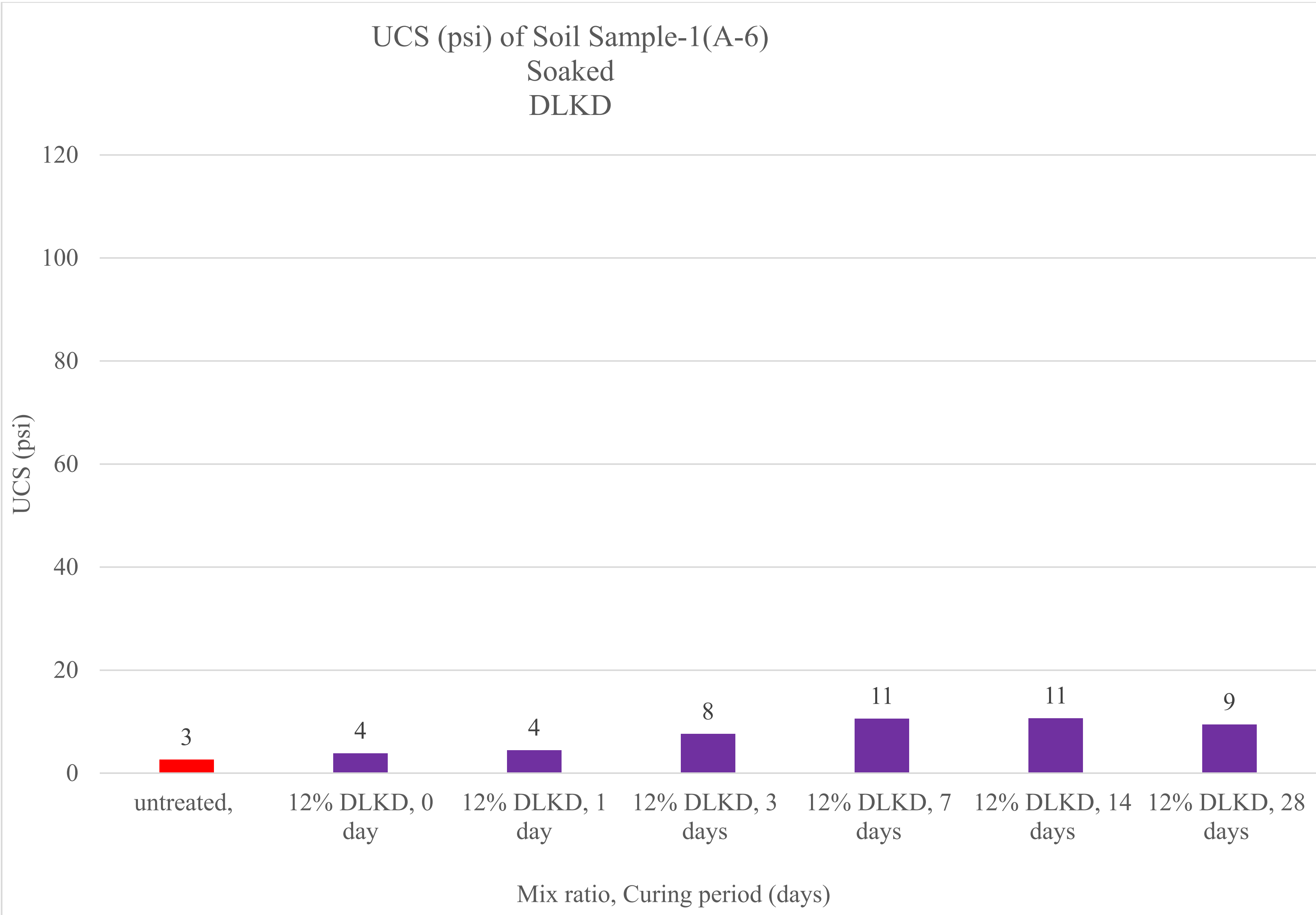


Figure 3.39: Comparison of Soaked UCS of Soil-1 (A-6) & 12% DLKD Mix

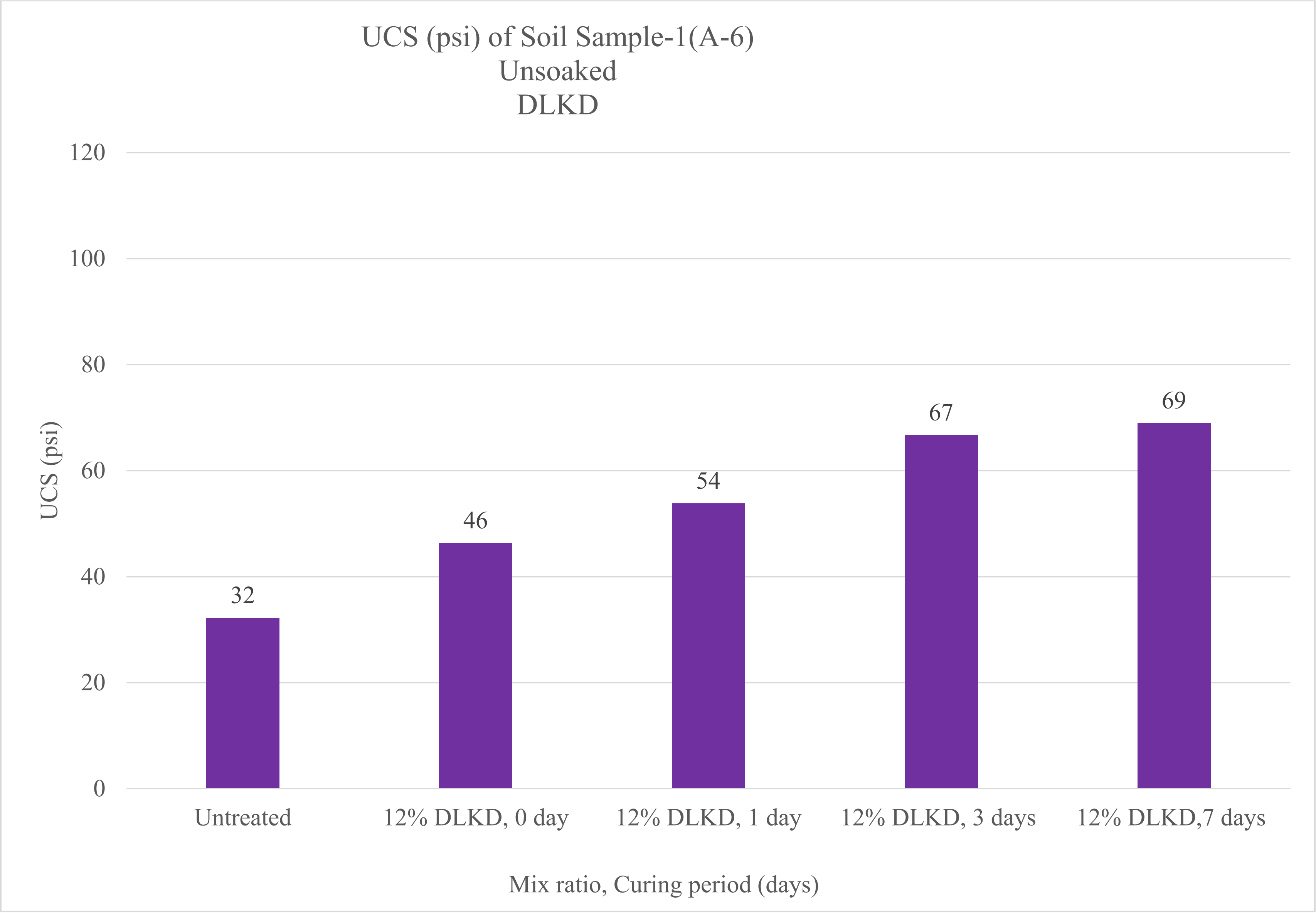


Figure 3.40: Comparison of Unsoaked UCS of Soil-1 (A-6) & 12% DLKD Mix

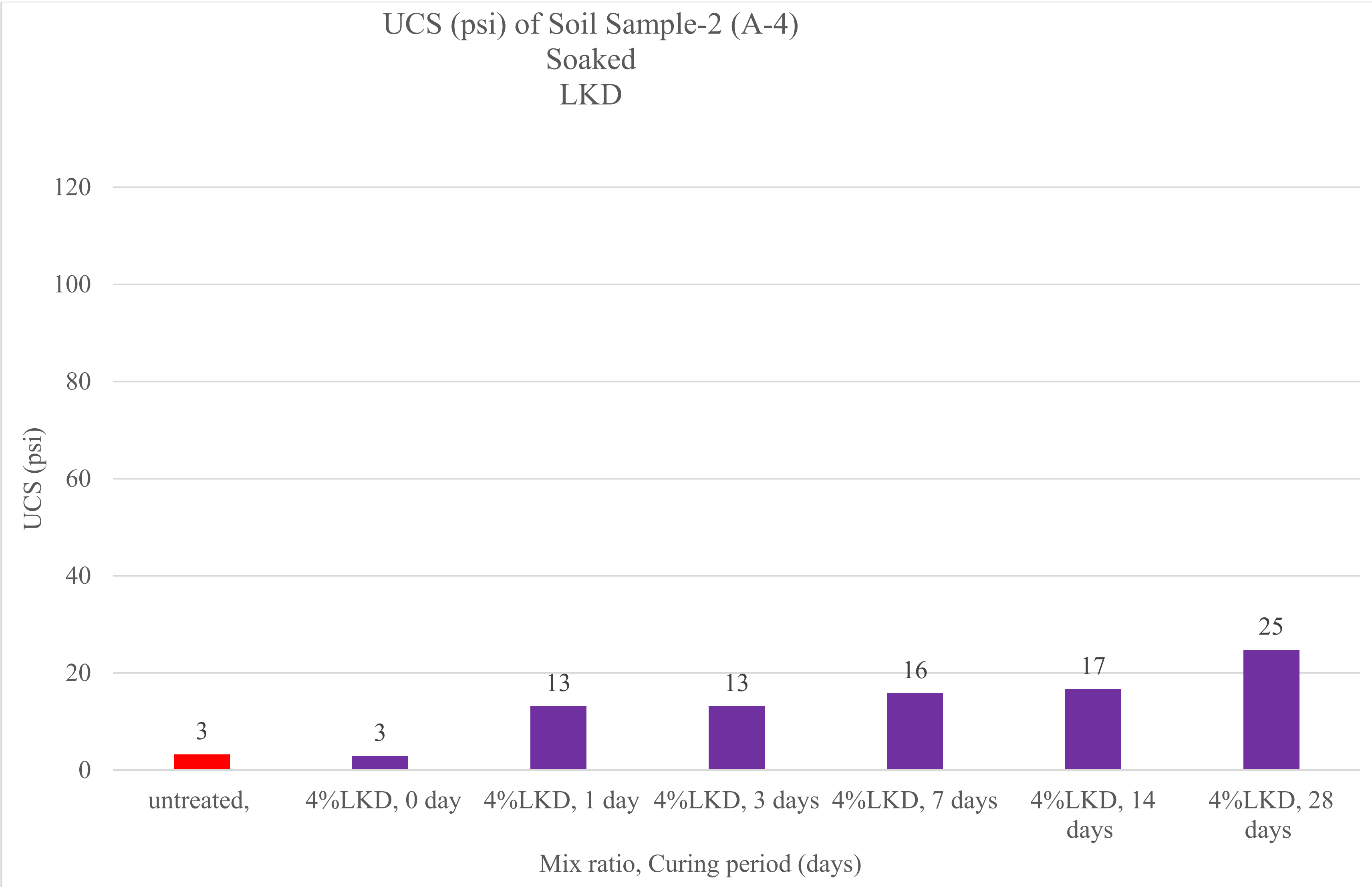


Figure 3.41: Comparison of Soaked UCS of Soil-2 (A-4) & 4% LKD Mix

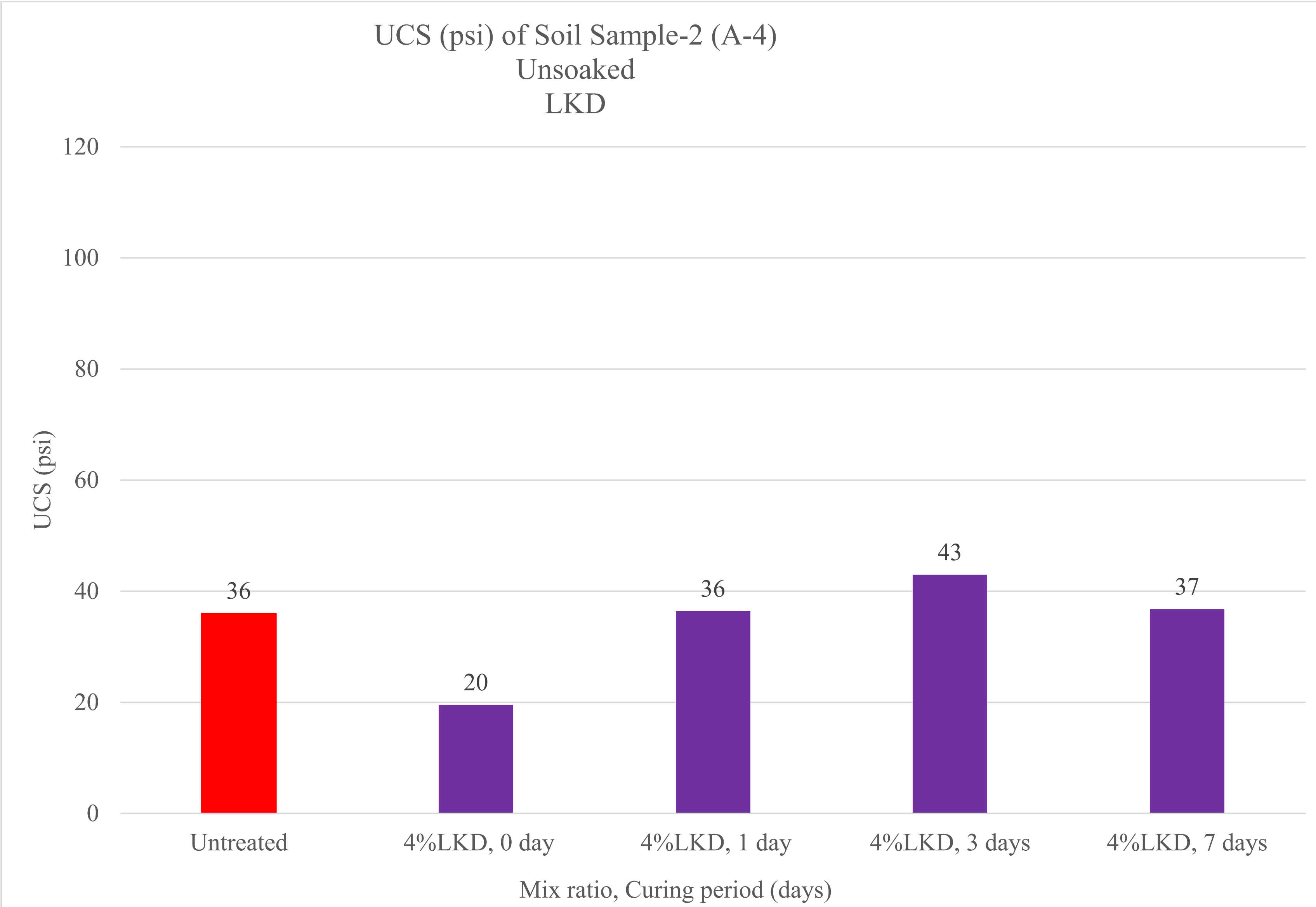


Figure 3.42: Comparison of Unsoaked UCS of Soil-2 (A-4) & 4% LKD Mix

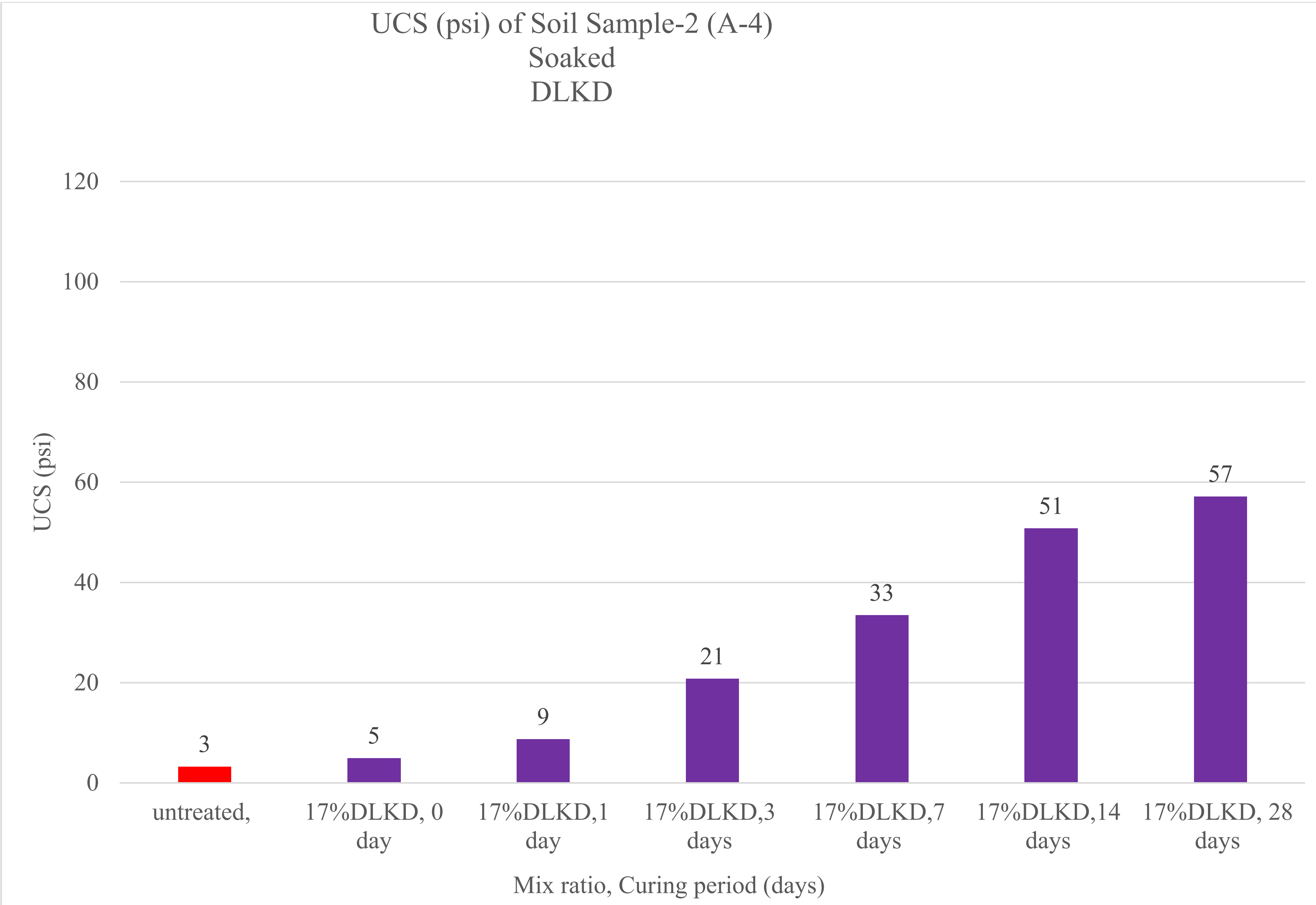


Figure 3.43: Comparison of Soaked UCS of Soil-2 (A-4) & 17% DLKD Mix

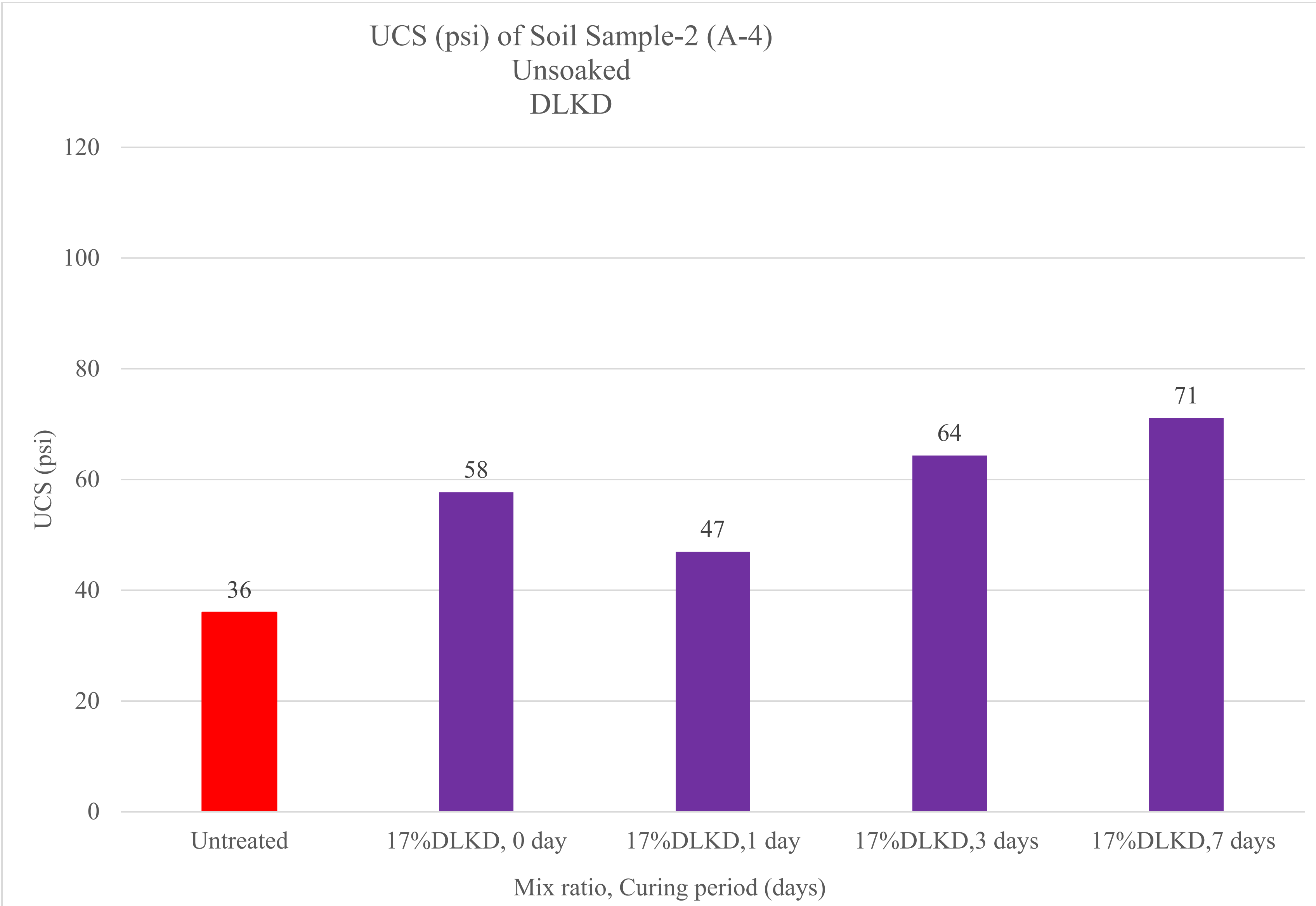


Figure 3.44: Comparison of Unsoaked UCS of Soil-2 (A-4) & 17% DLKD Mix

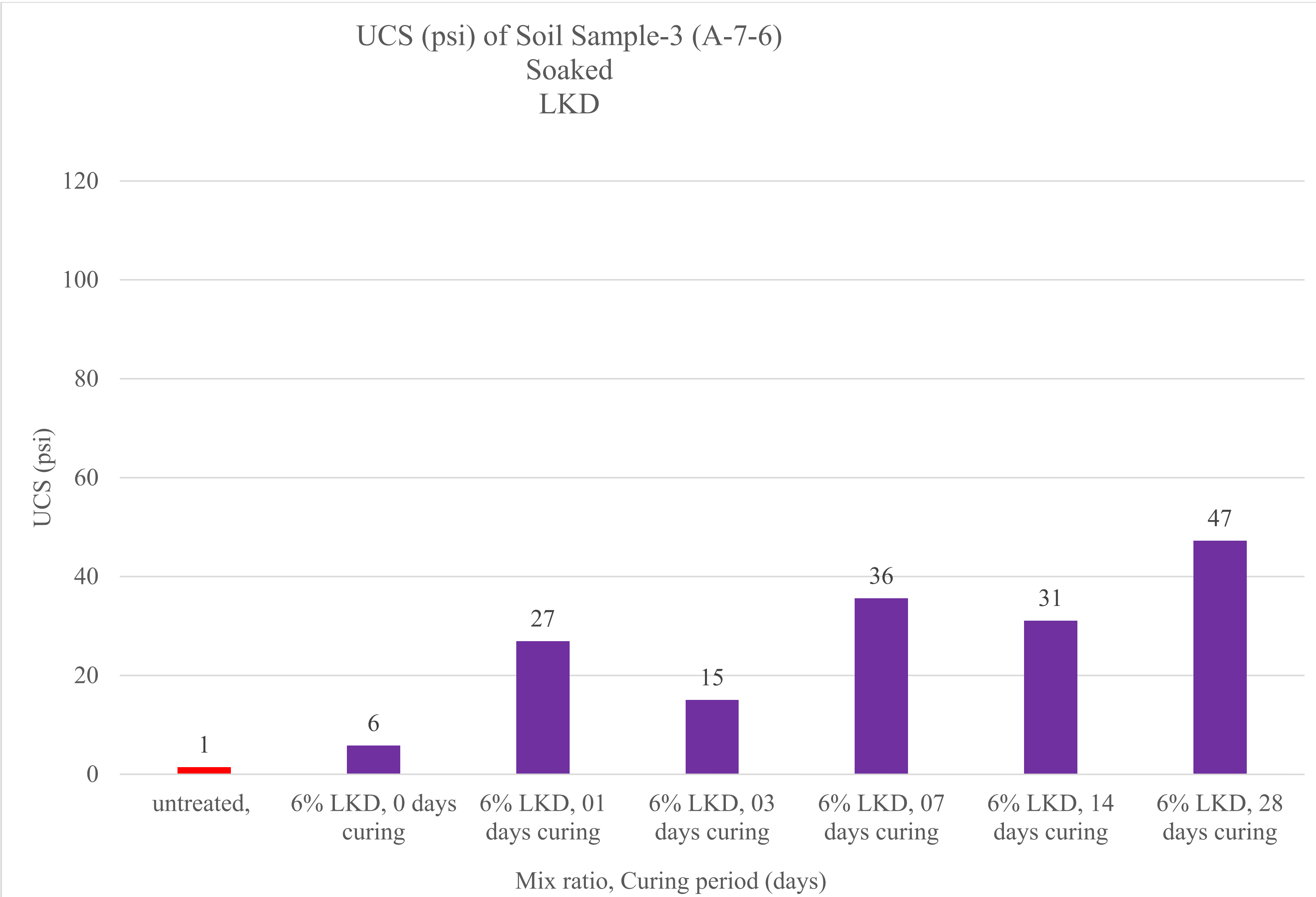


Figure 3.45: Comparison of Soaked UCS of Soil-3 (A-7-6) & LKD Mix

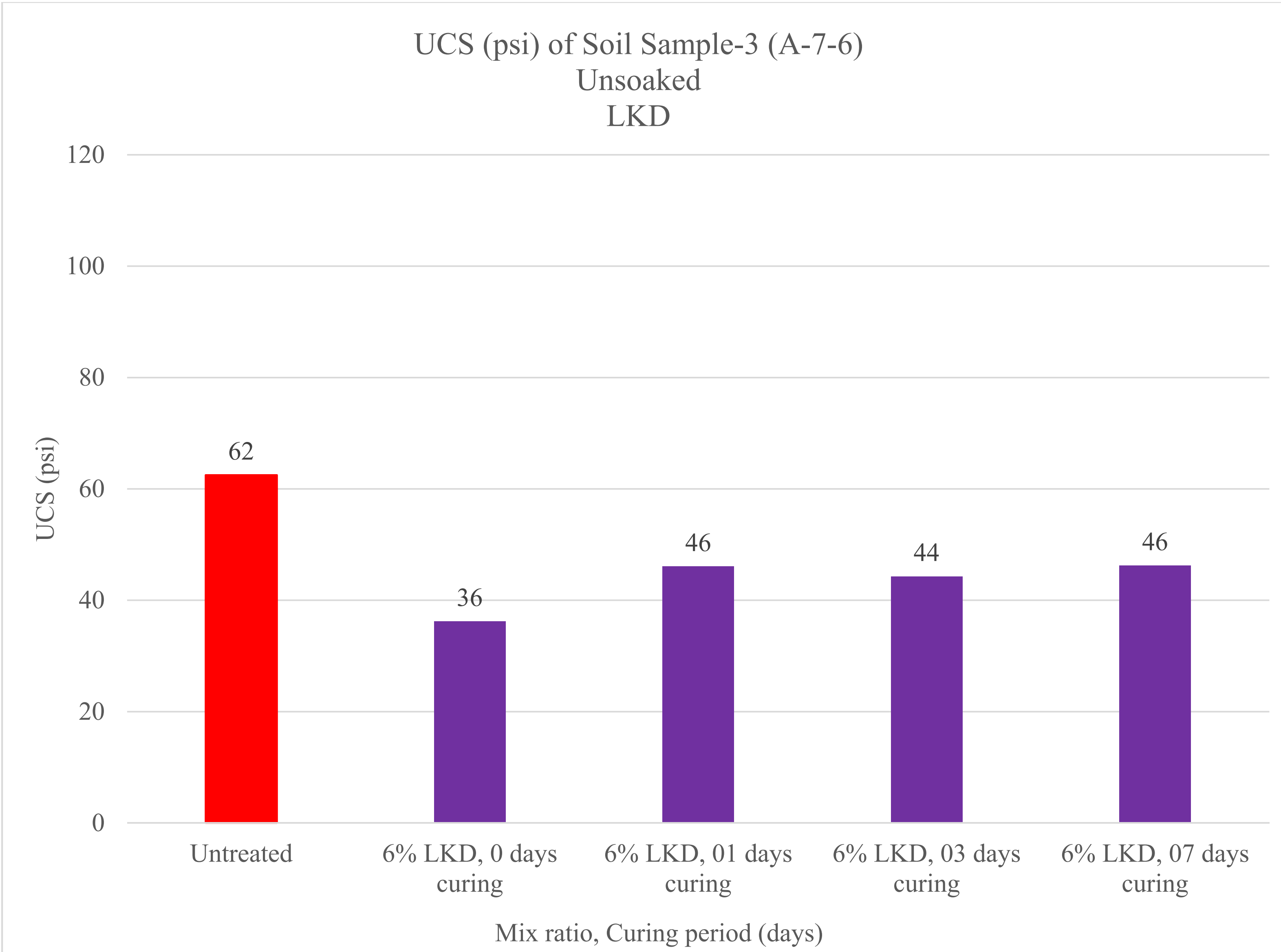


Figure 3.46: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & LKD Mix

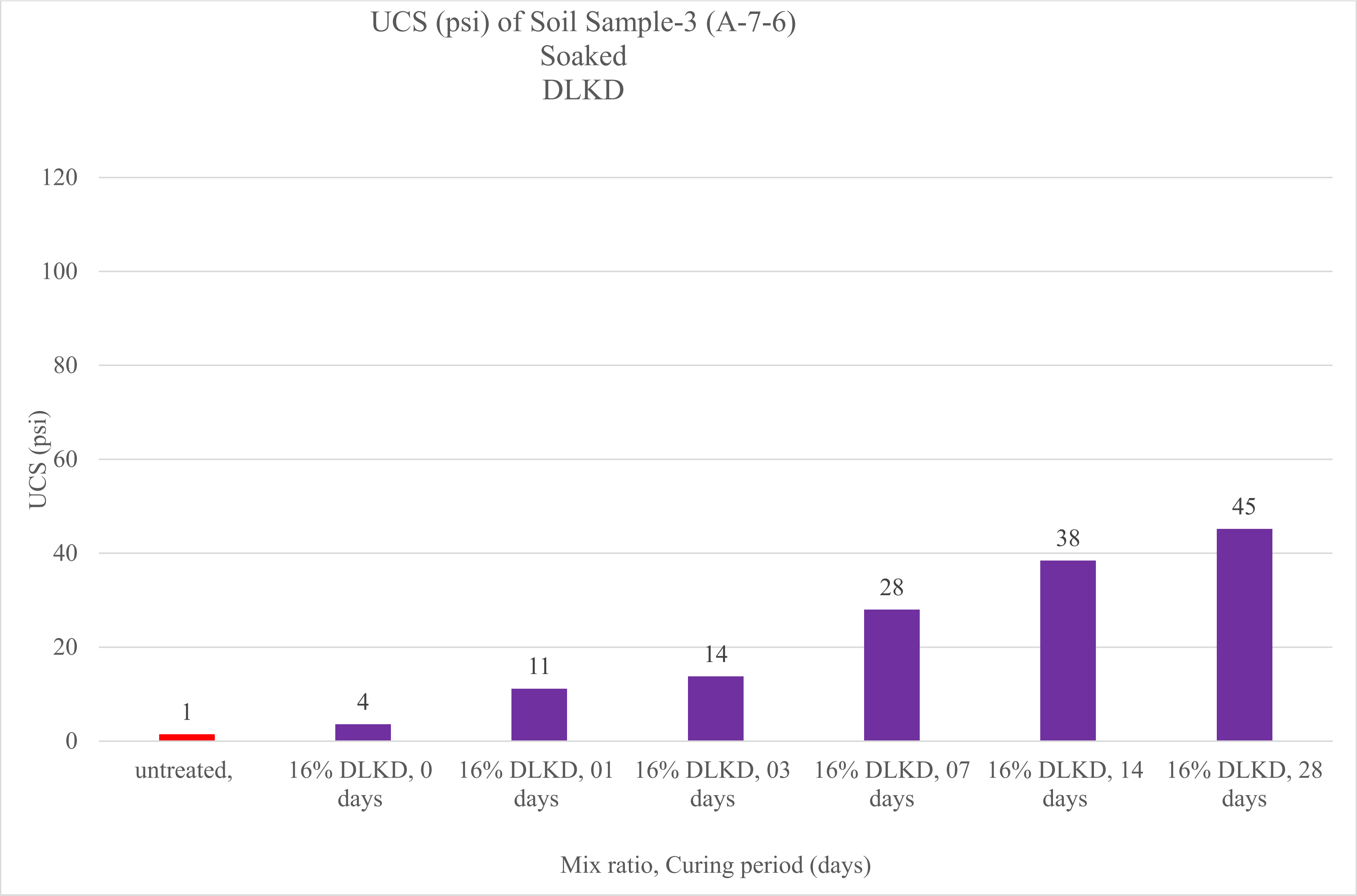


Figure 3.47: Comparison of Soaked UCS of Soil-3 (A-7-6) & DLKD Mix

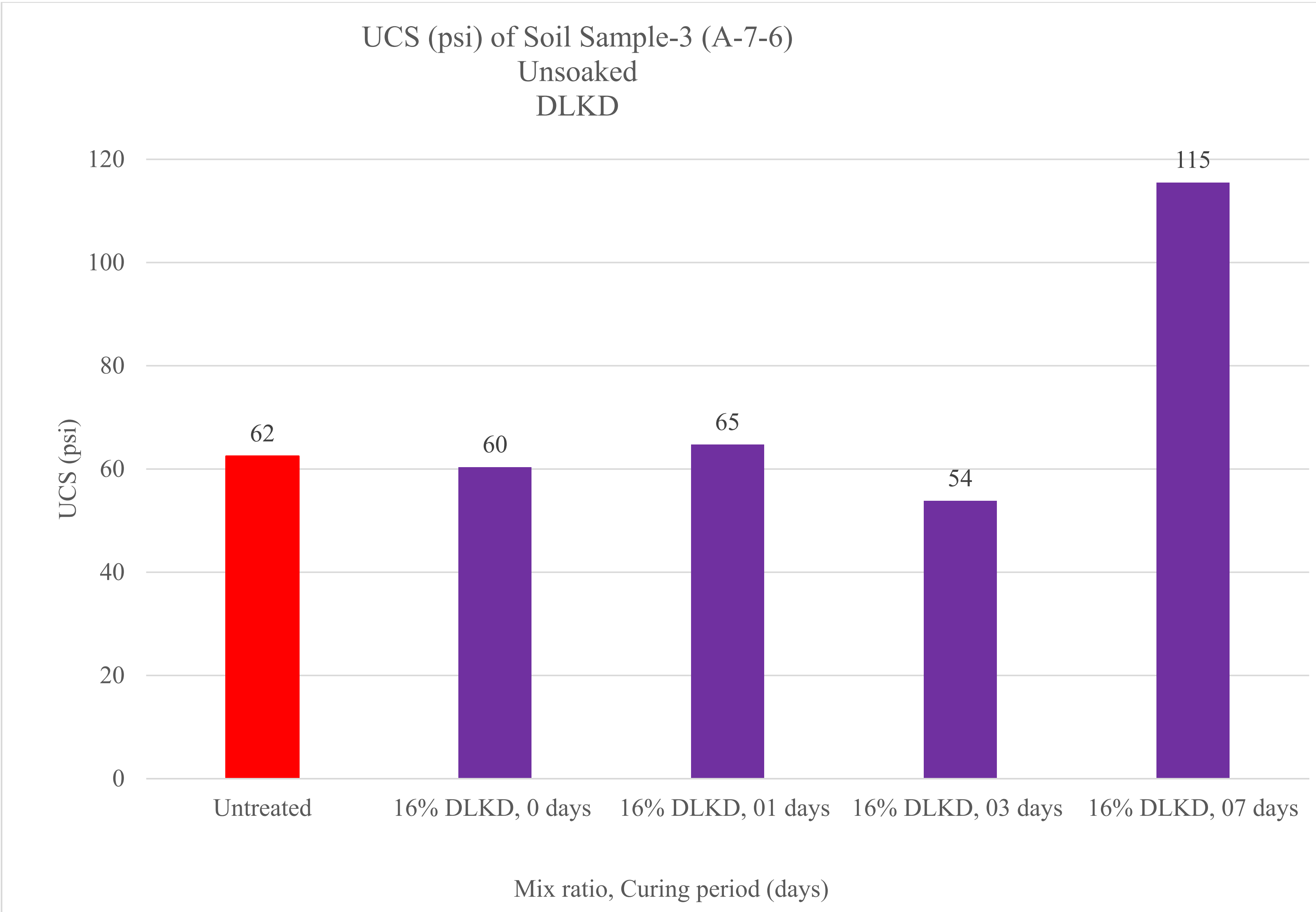


Figure 3.48: Comparison of Unsoaked UCS of Soil-3 (A-7-6) & DLKD Mix

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:

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

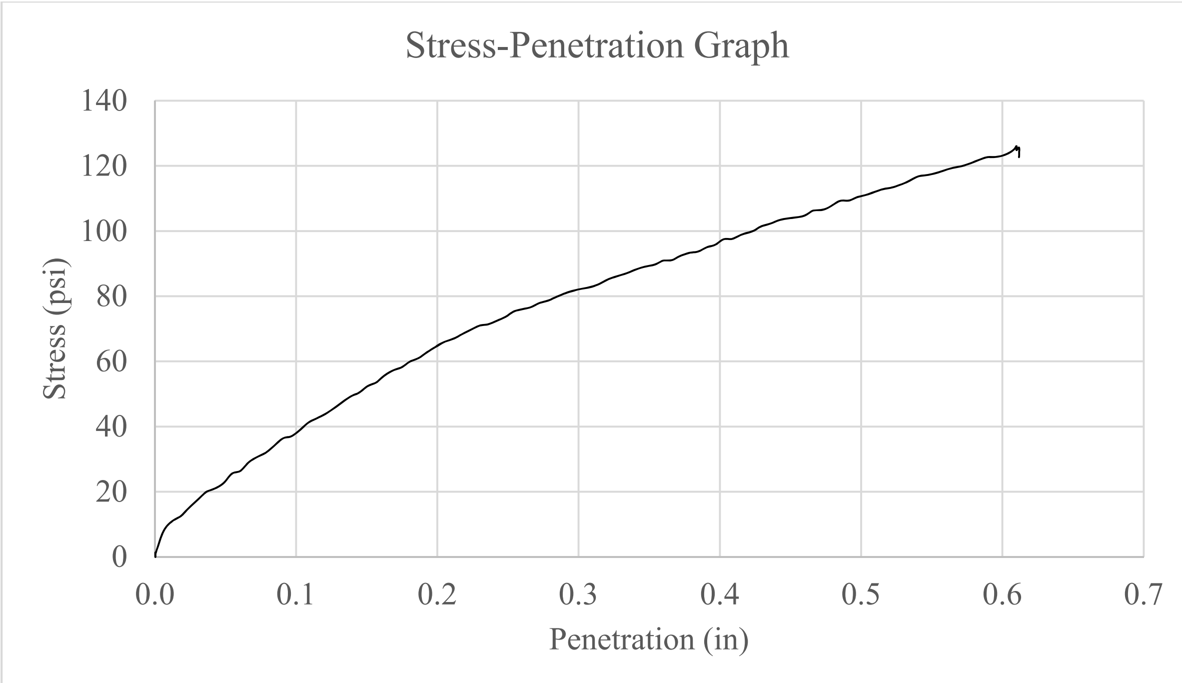


Figure 3.49: Example Stress-Penetration Graph of CBR Test (Soaked CBR of Untreated Soil-1, Specimen-1)

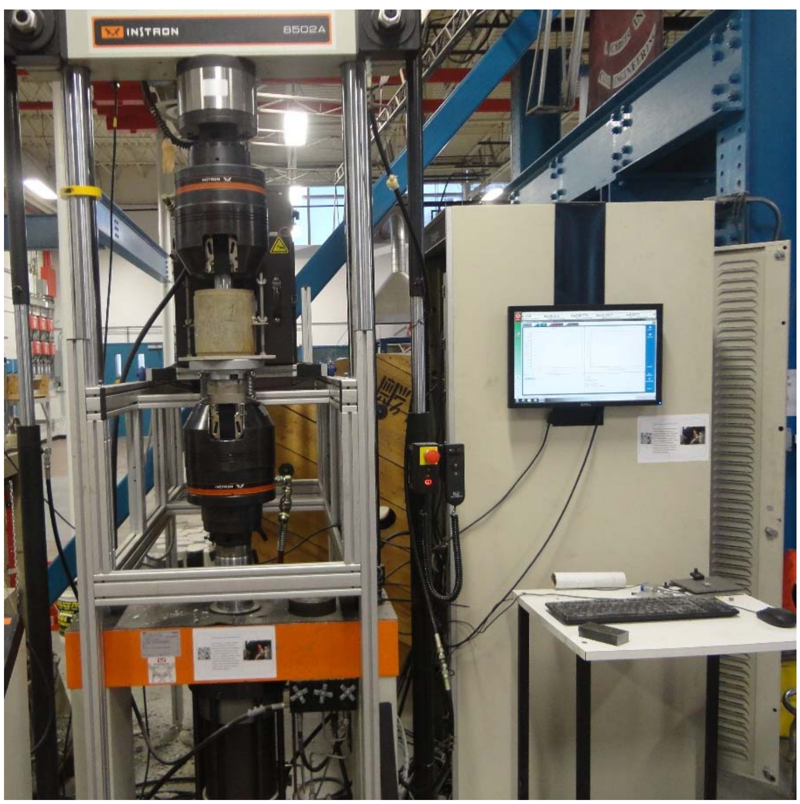


Figure 3.50: CBR Test Using Instron

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)

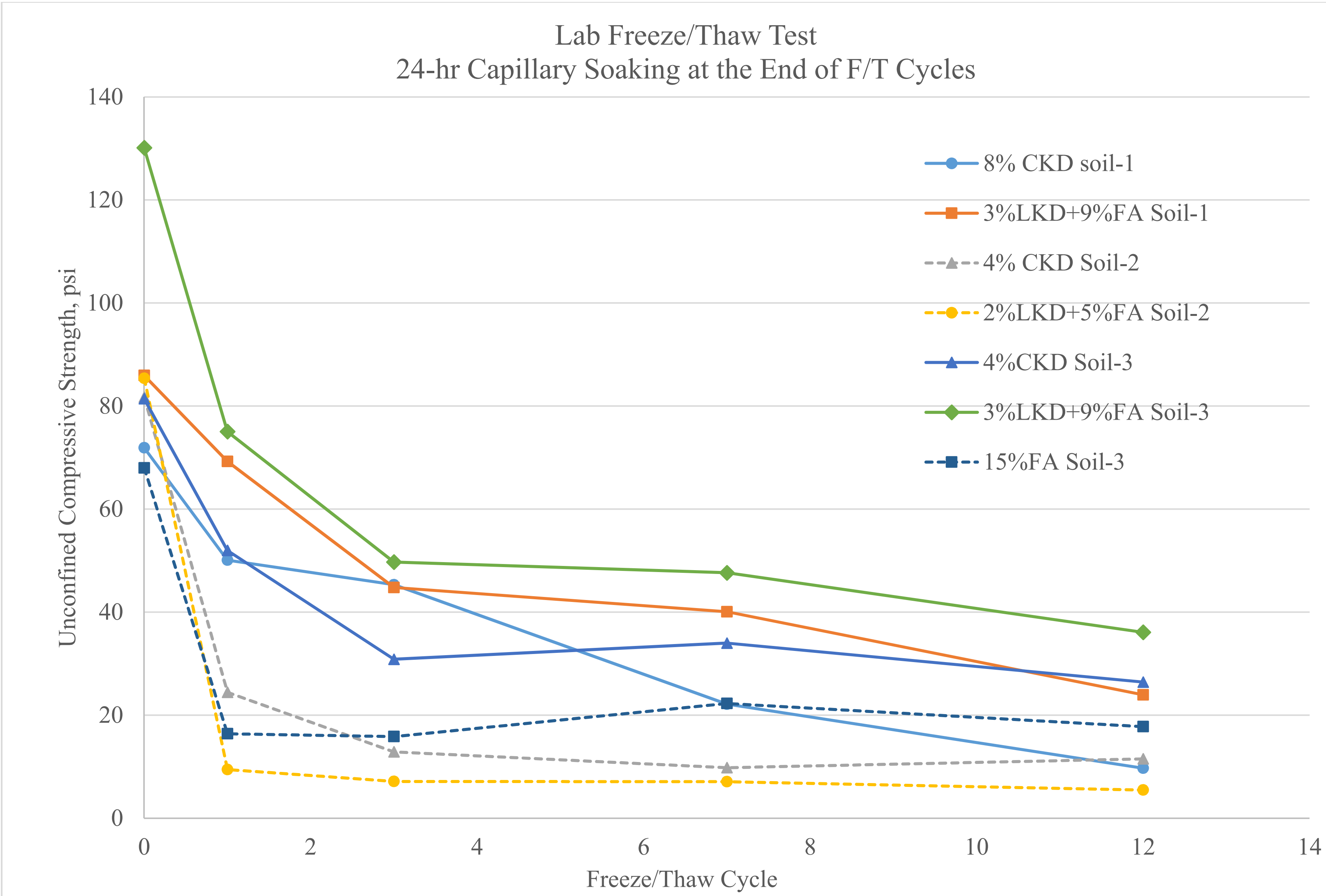


Figure 3.52: Reduction of UCS with Freeze/Thaw Cycles (24-hr Capillary Soaking at the End of Cycles)

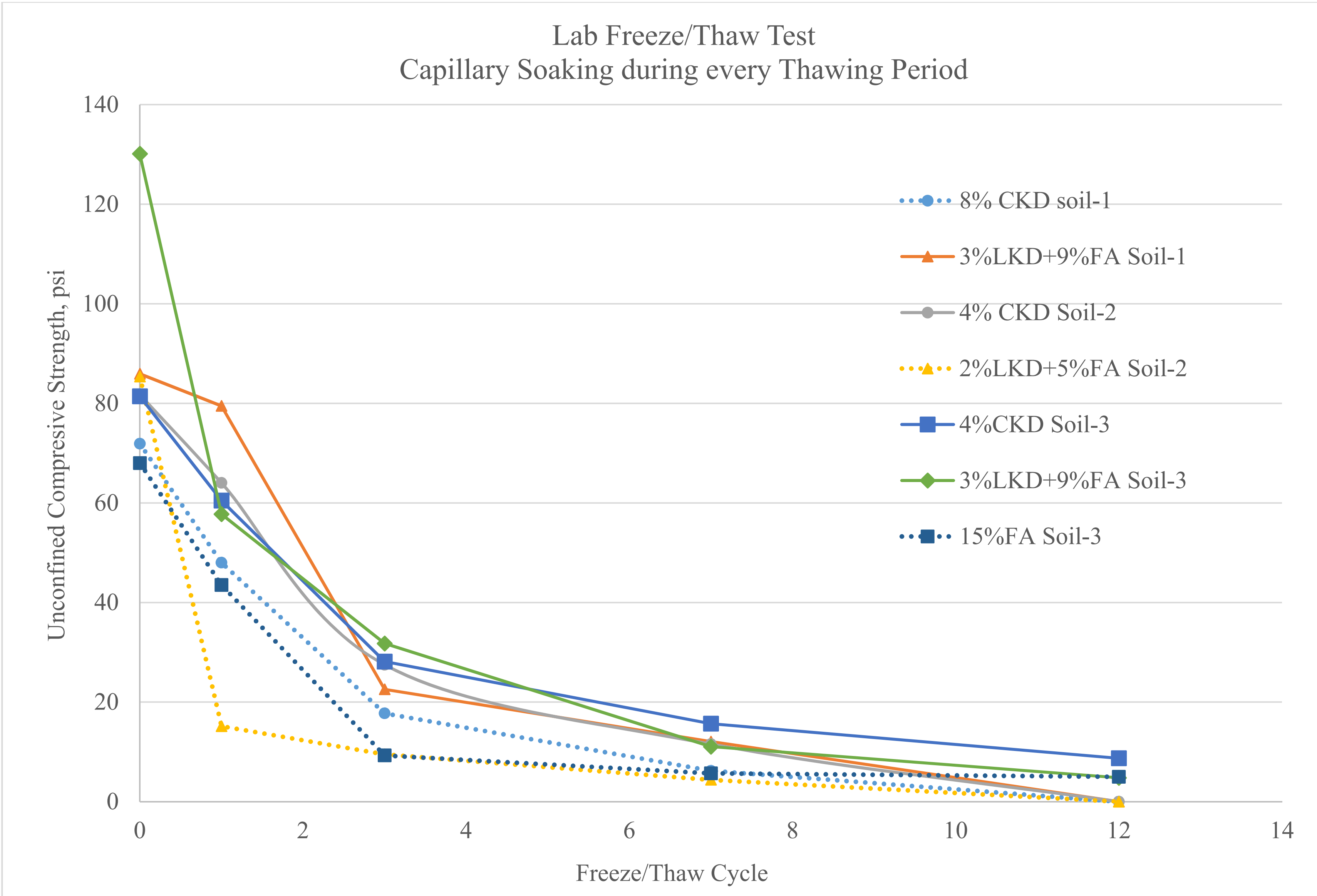


Figure 3.53: Reduction of UCS with Freeze/Thaw Cycles (24-hr Capillary Soaking during every Thawing Period)

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.

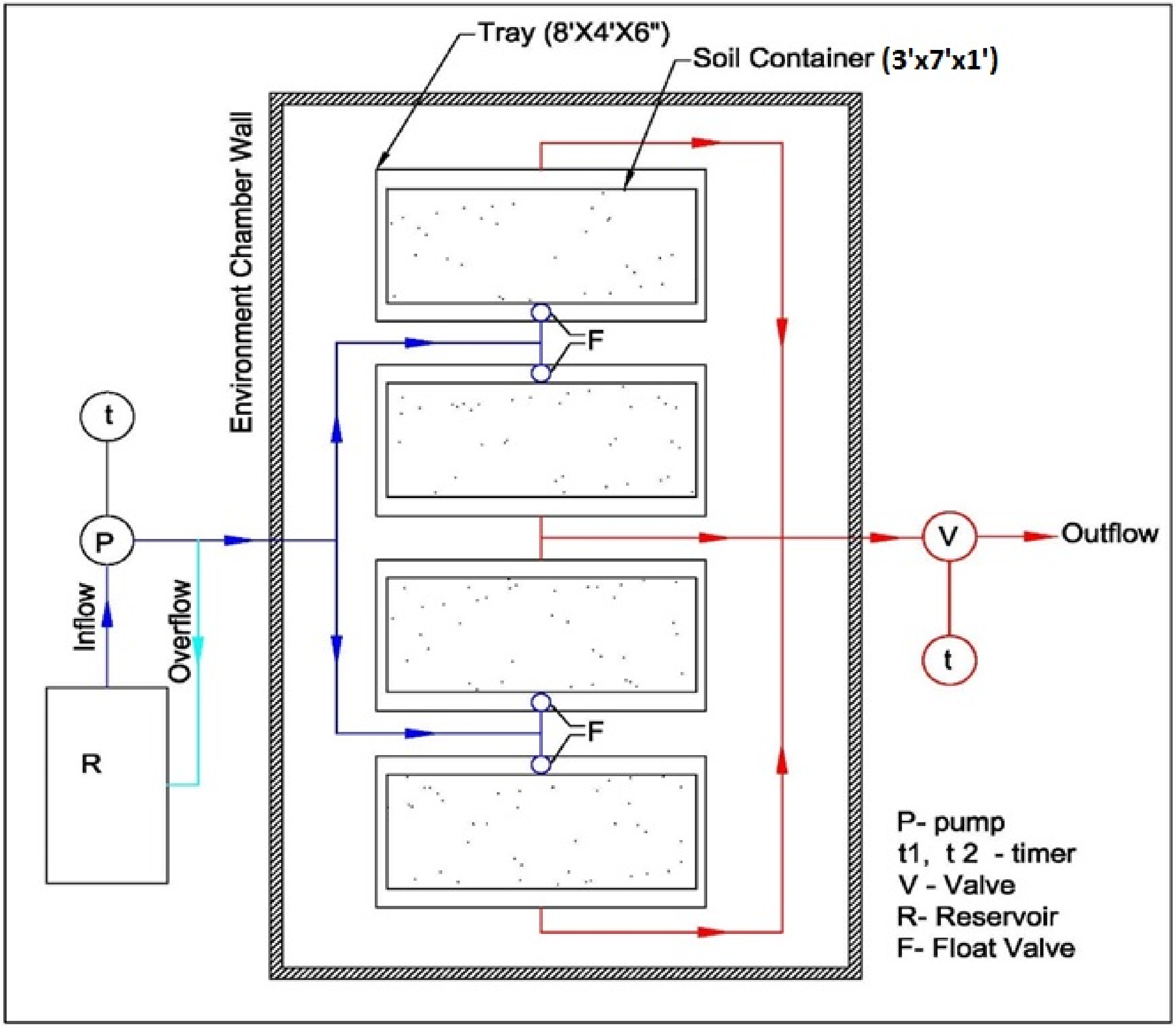


Figure 3.57: Schematic Diagram of Water Flow

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.

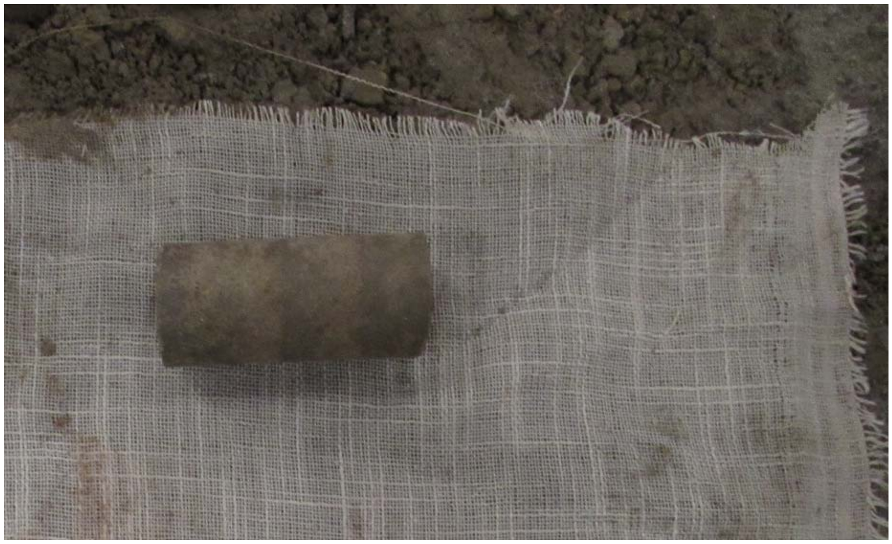


Figure 3.58: Wrapping & Placing Samples

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

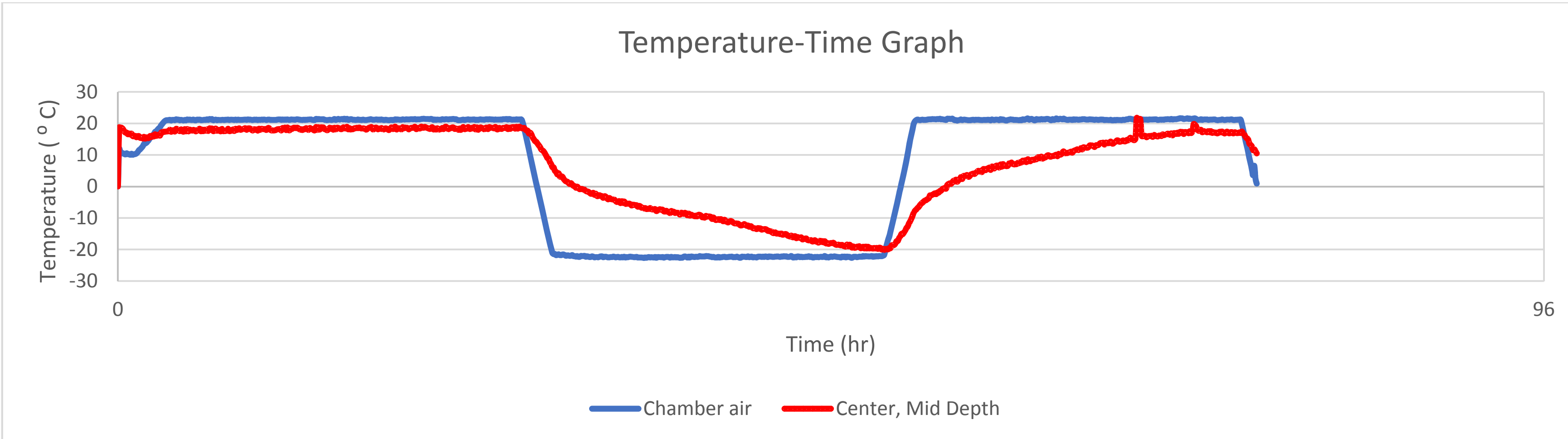


Figure 3.60: Soil Slab and Air Temperature

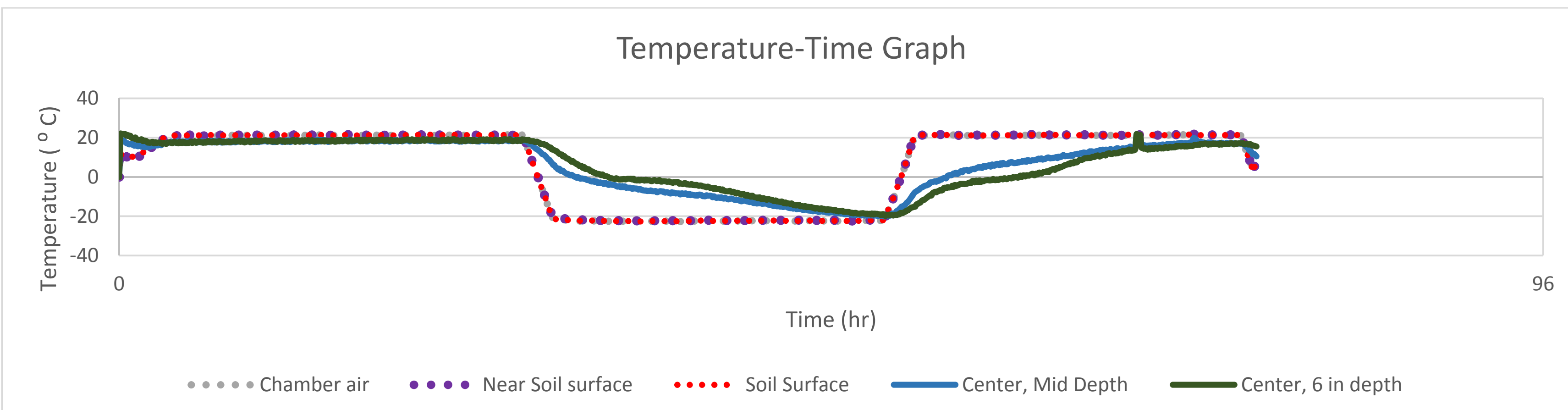


Figure 3.61: Soil Slab Temperature Variation with Depth

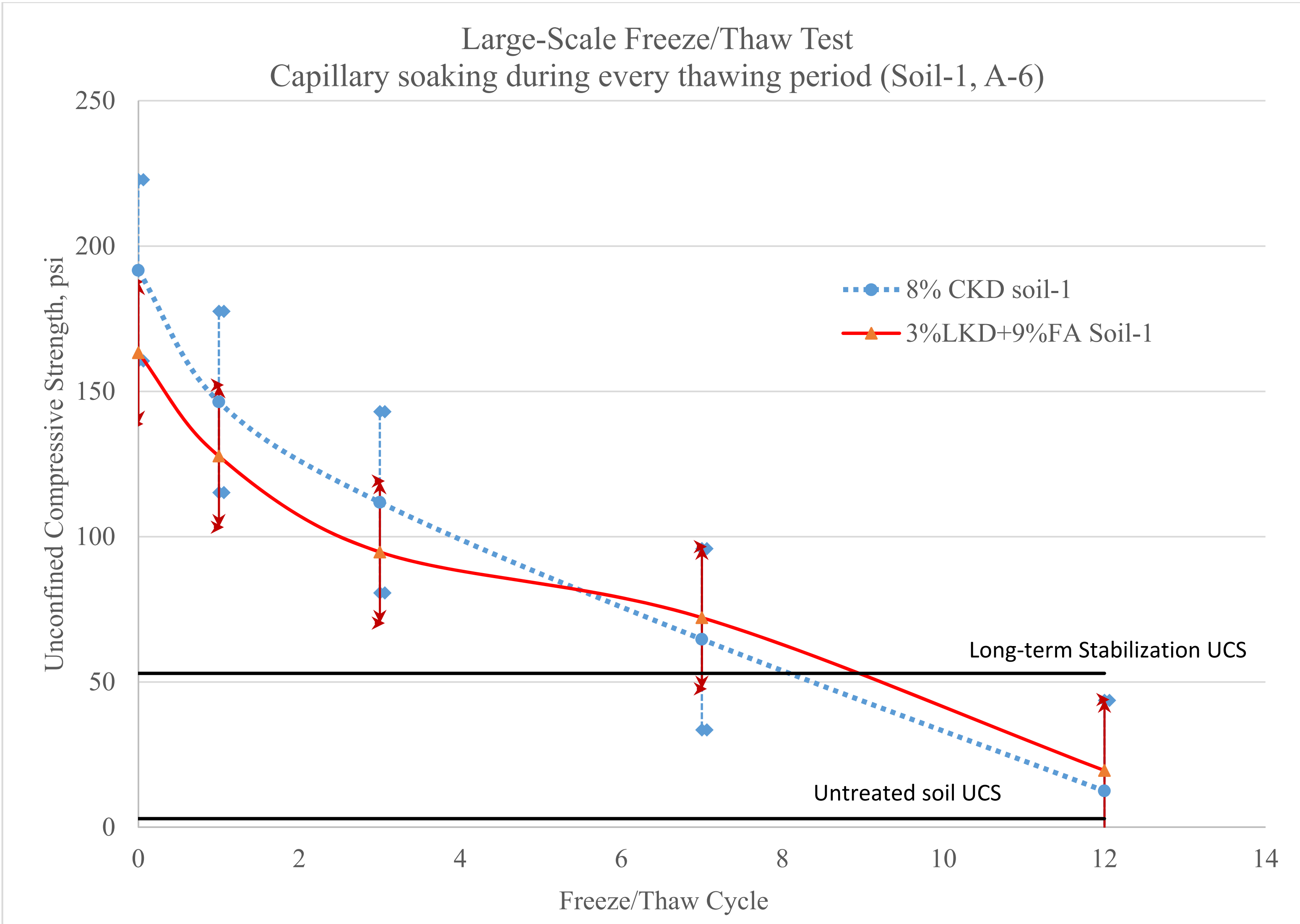


Figure 3.62: Reduction of UCS with Large-Scale Freeze/Thaw Cycles (Soil-1, A-6)

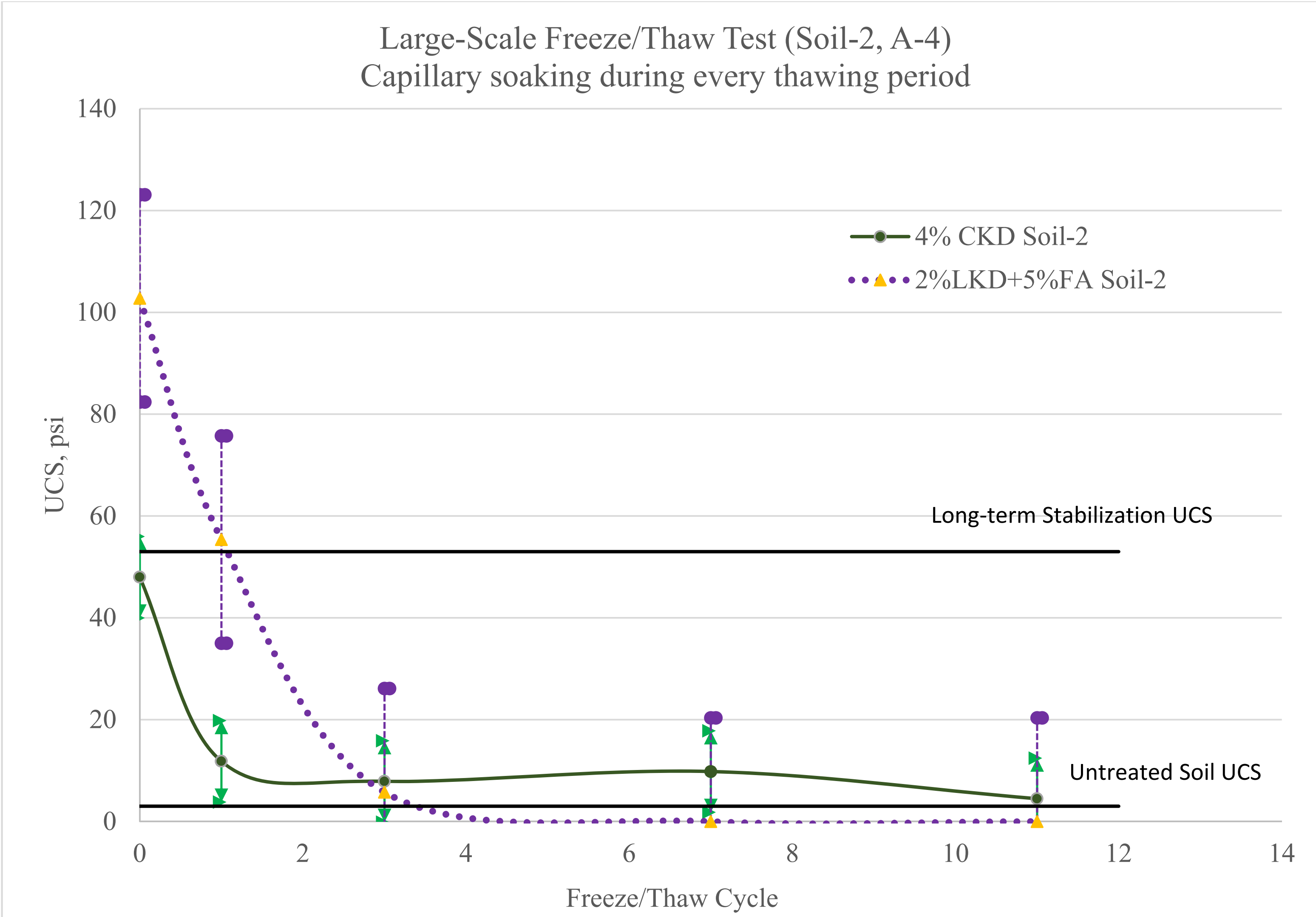


Figure 3.63: Reduction of UCS with Large Scale Freeze/Thaw Cycles (Soil-2, A-4)

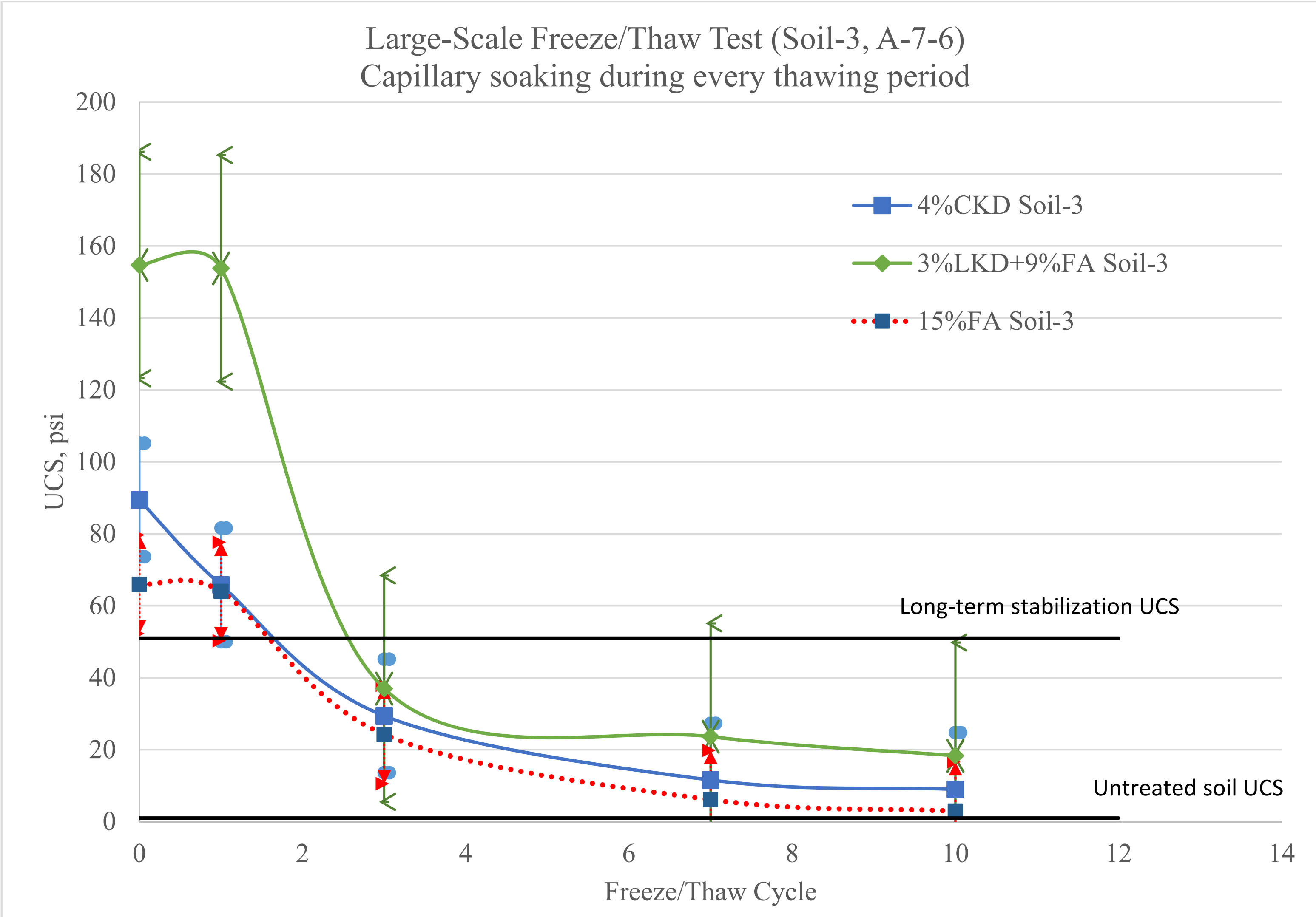


Figure 3.64: Reduction of UCS with Large-Scale Freeze/Thaw Cycles (Soil-3, A-7-6)

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.

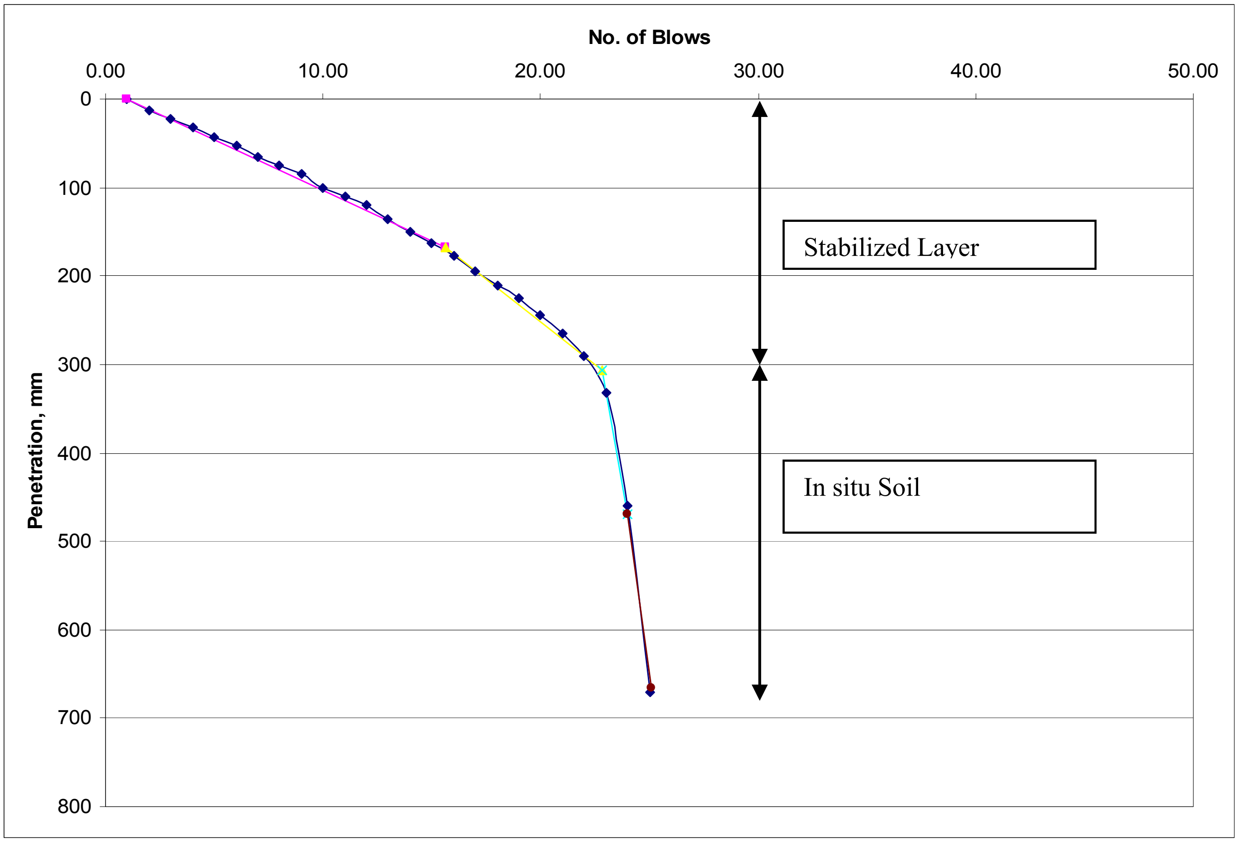


Figure 4.1: Typical DCP Results Plot for a Stabilized Subgrade

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.

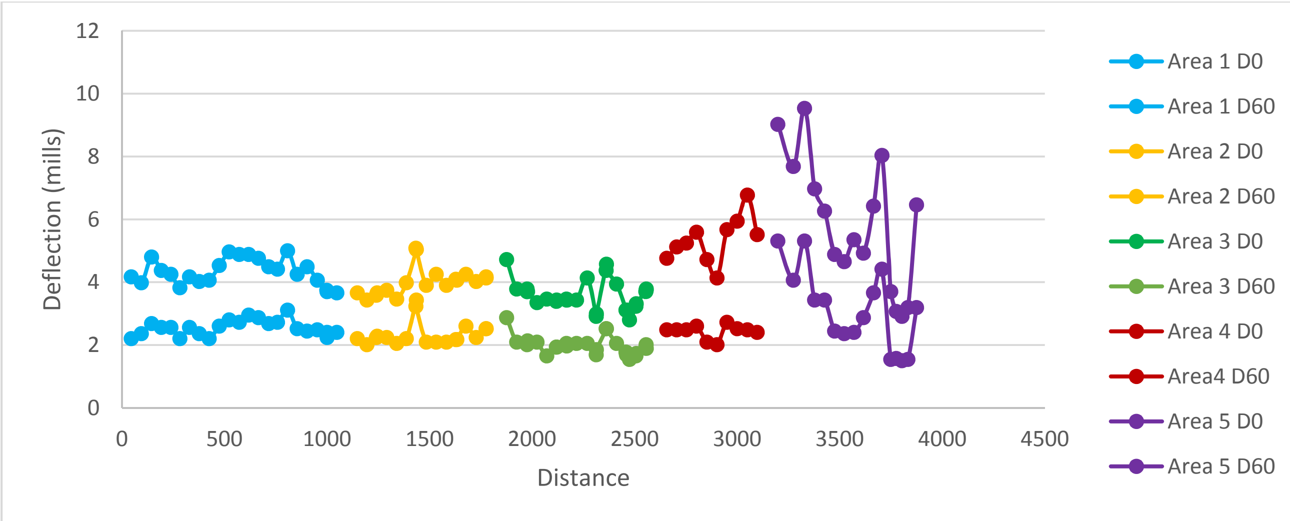


Figure 4.4: Deflection Plots for Inside Lane of I-75/I-96

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.

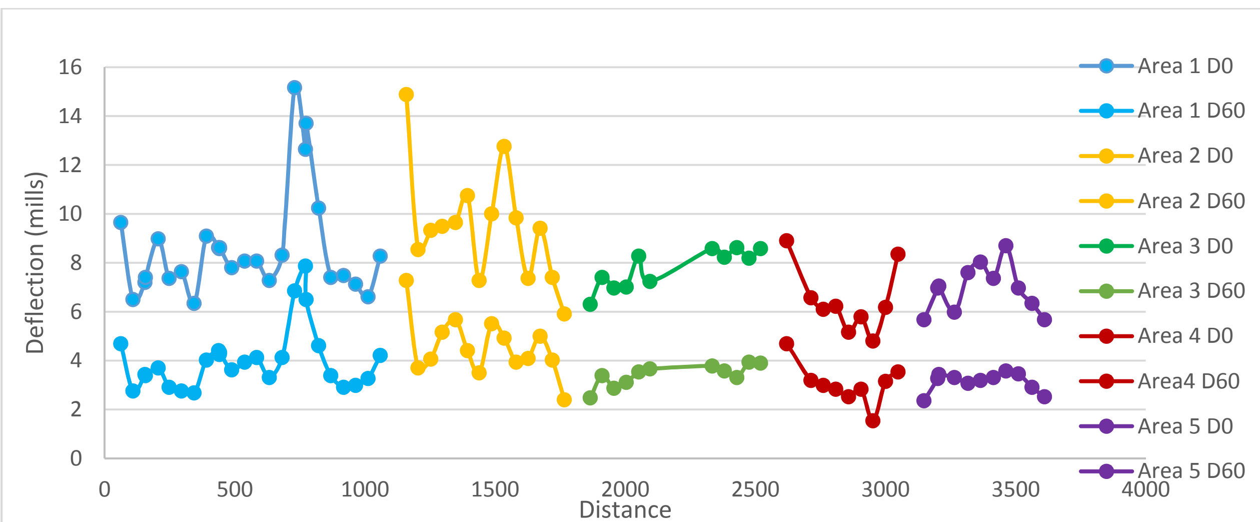


Figure 4.5: Deflection Plots for Shoulder Lane of I-75/I-96

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

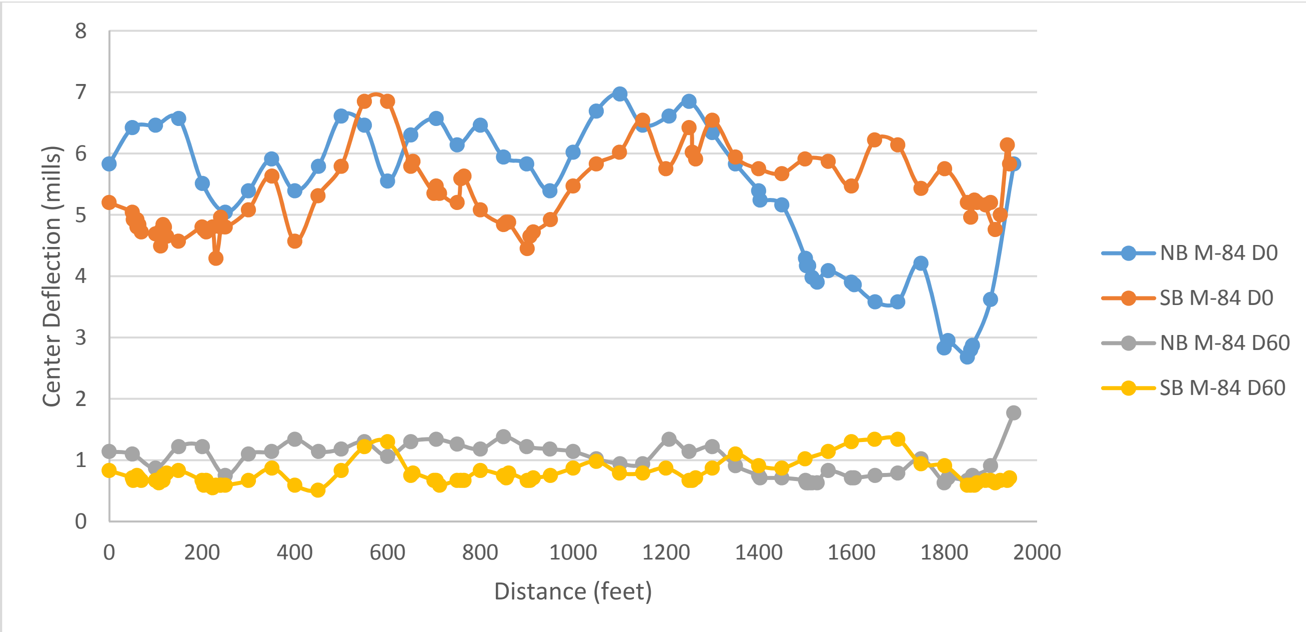


Figure 4.7: Deflection Plots for M-84

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.

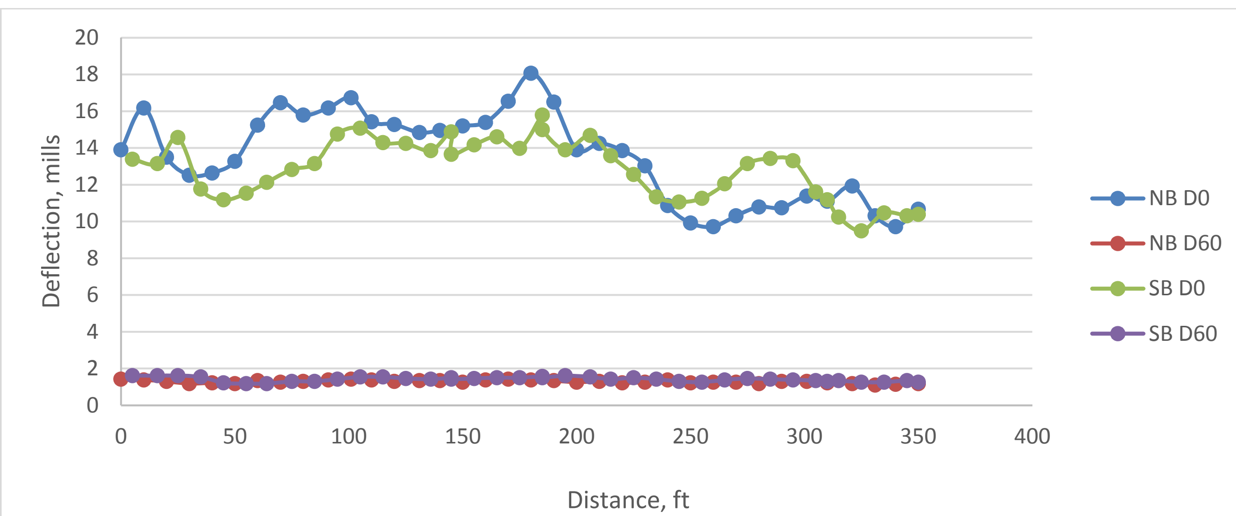


Figure 4.9: Deflection Plots for Waverly Road

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.

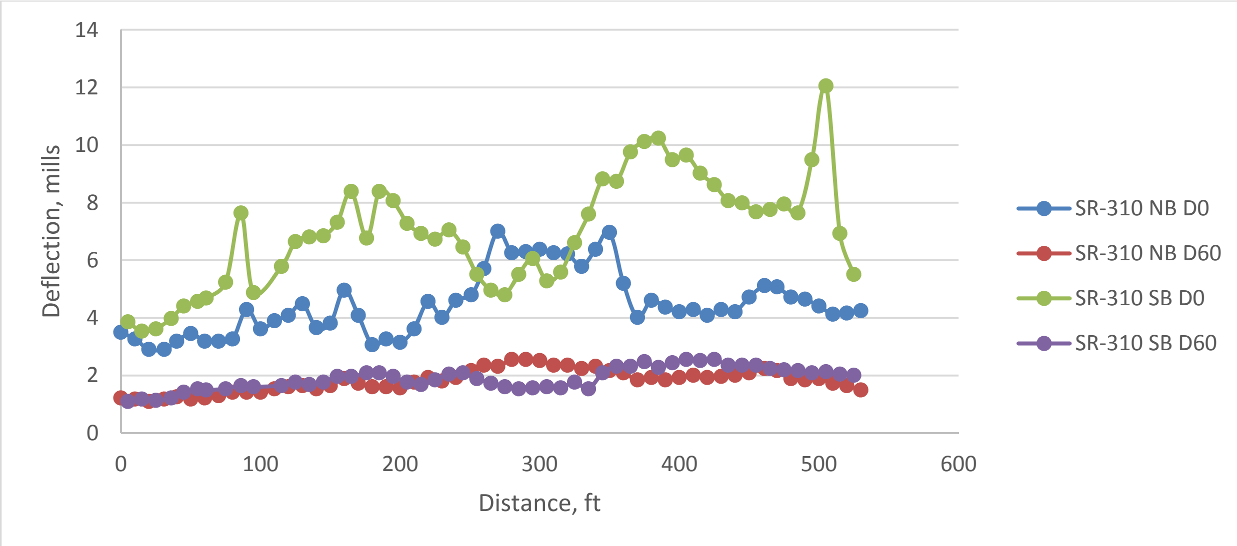


Figure 4.11: FWD Deflection Plots for SR 310

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:

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

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.

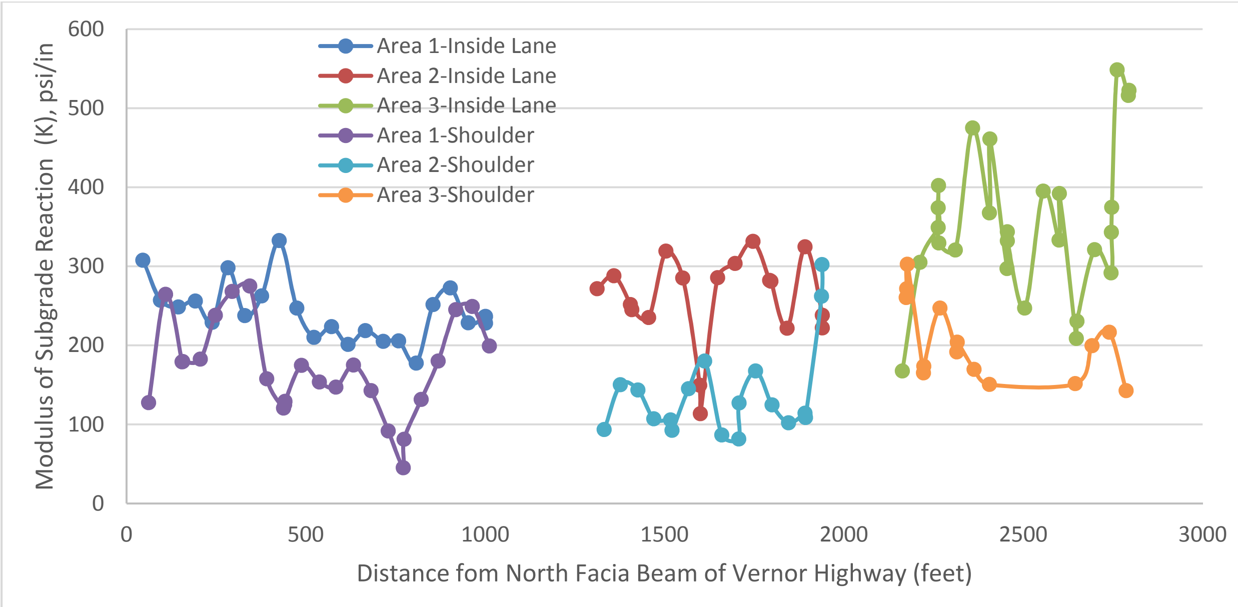


Figure 4.12: Modulus of Subgrade Reaction Values for Areas 1-3

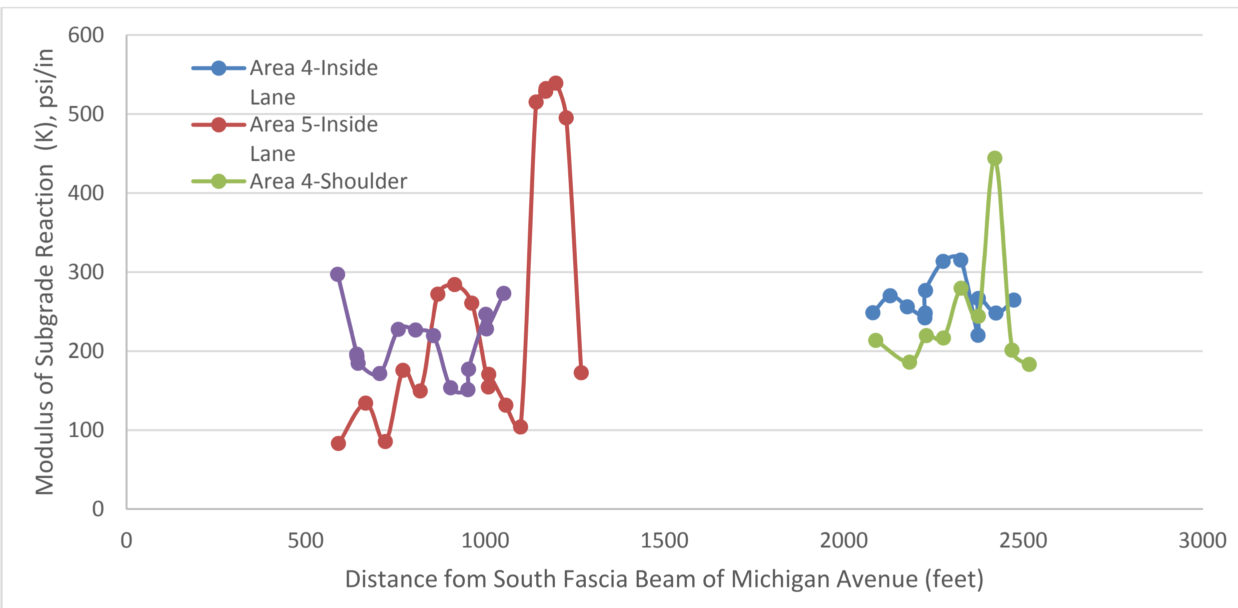


Figure 4.13: Modulus of Subgrade Reaction Values for Areas 4-3

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 *i*th layer

BR_i = DCP blow rate for the *i*th layer (blows/inch)

T_i = Thickness of the *i*th layer

The structural number (SN) of the *i*th 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.

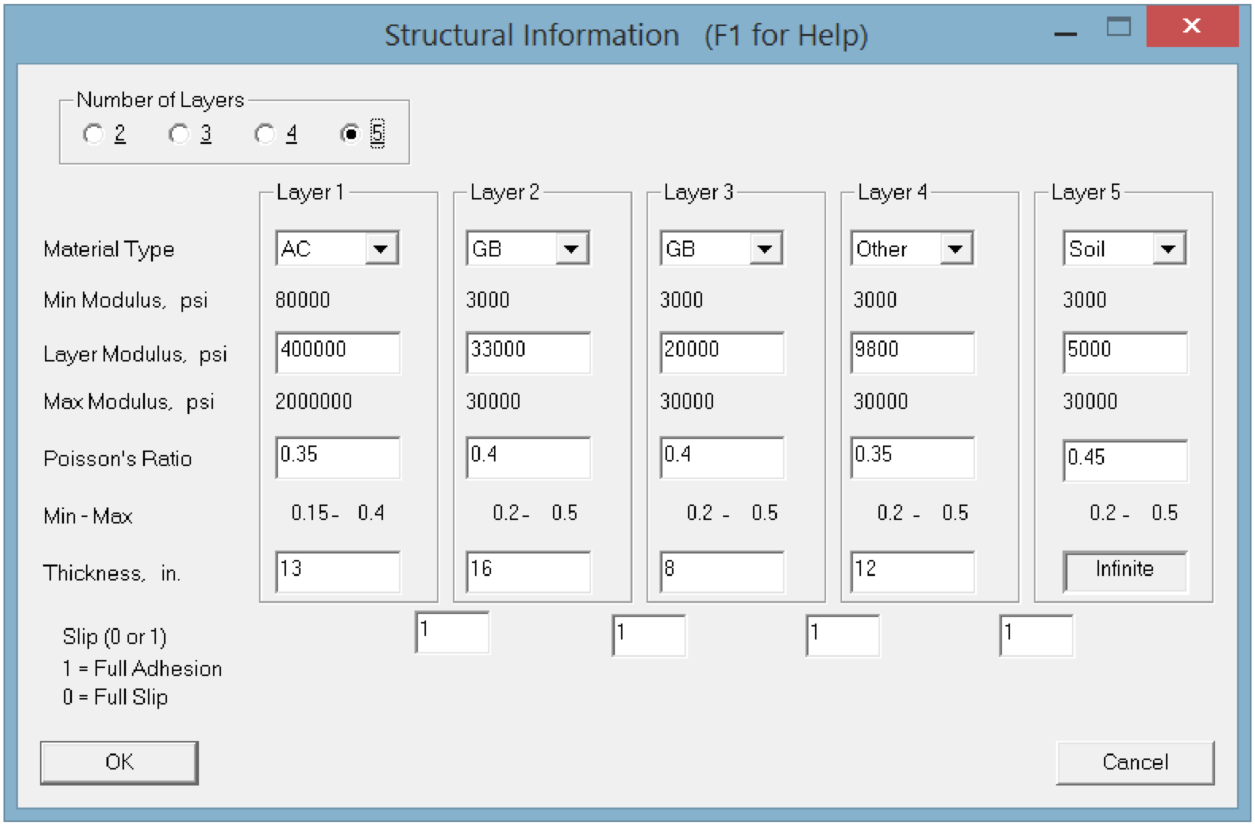


Figure 5.1: WESLEA Pavement Layer Properties Input Example

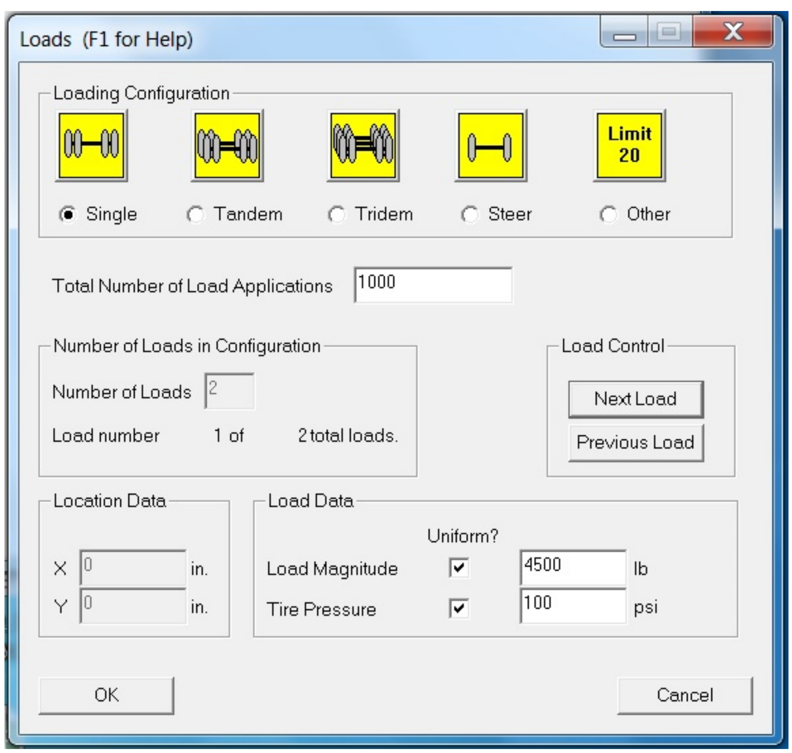


Figure 5.2: WESLEA Load Assignment Input Example

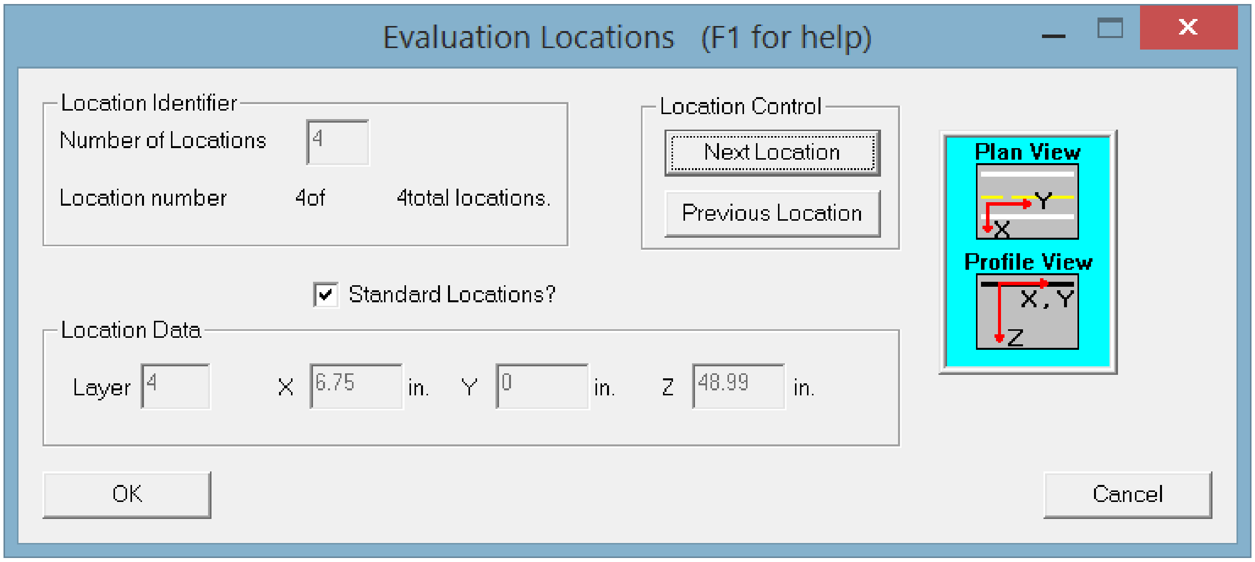


Figure 5.3: WESLEA Evaluation Locations Input Example

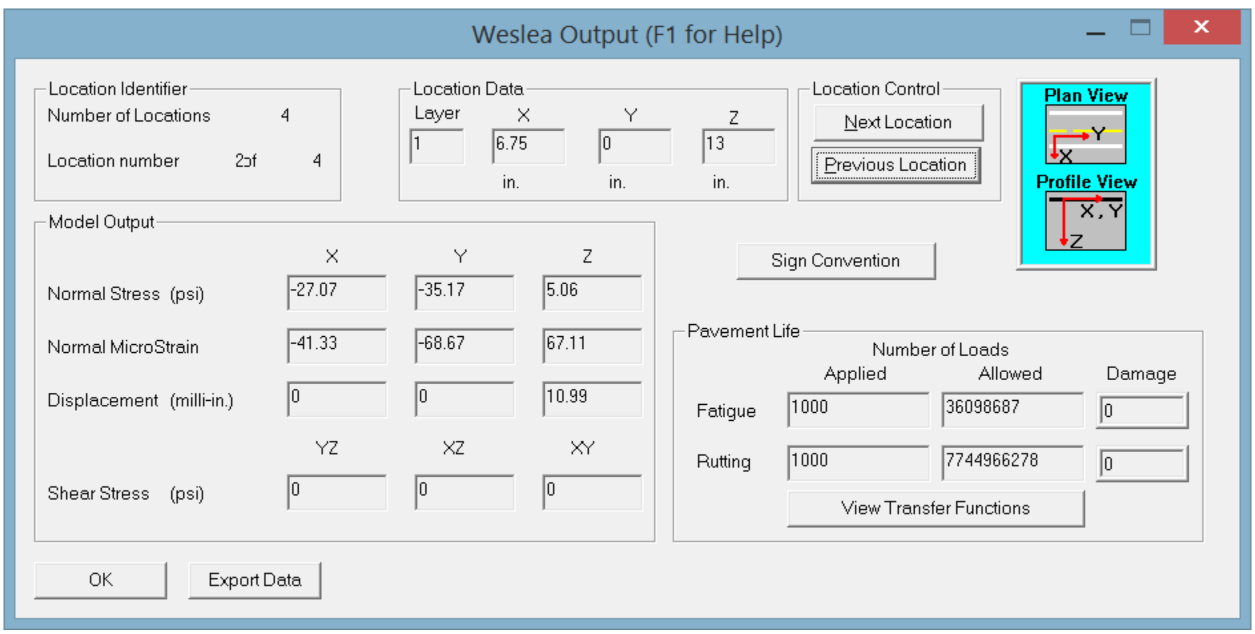


Figure 5.4: WESLEA Output Example

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 Thickness (in.)	Base/Subbase Layer Modulus (psi)	Stabilized Subgrade Thickness (in.)	Stabilized Subgrade Layer Modulus (psi)	Natural Subgrade (psi)	K_c (psi/in)
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.

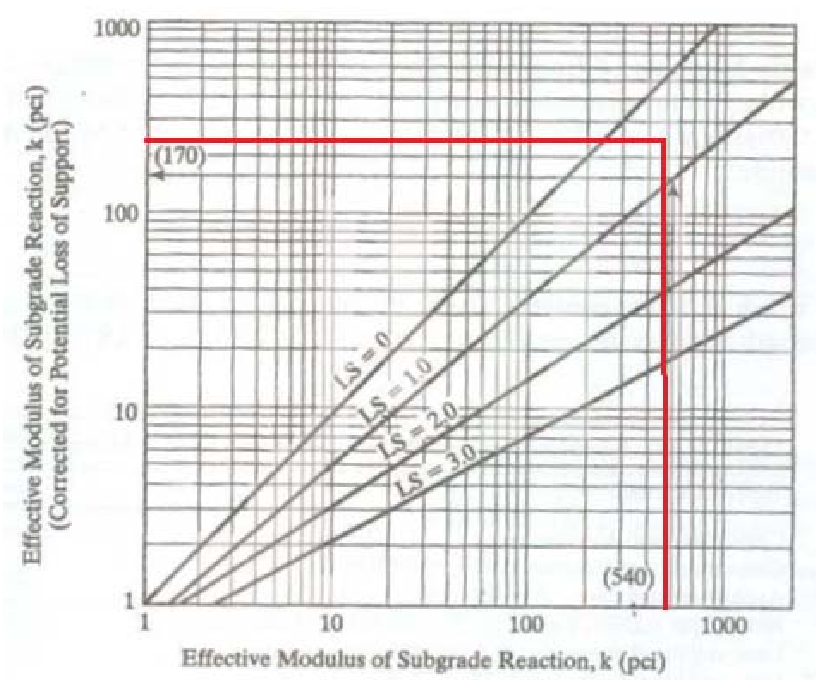


Figure 5.5: Effective Modulus of Subgrade Reaction Considering Potential Loss of Support (Ref: AASHTO 1993 Pavement Design Guideline)

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).

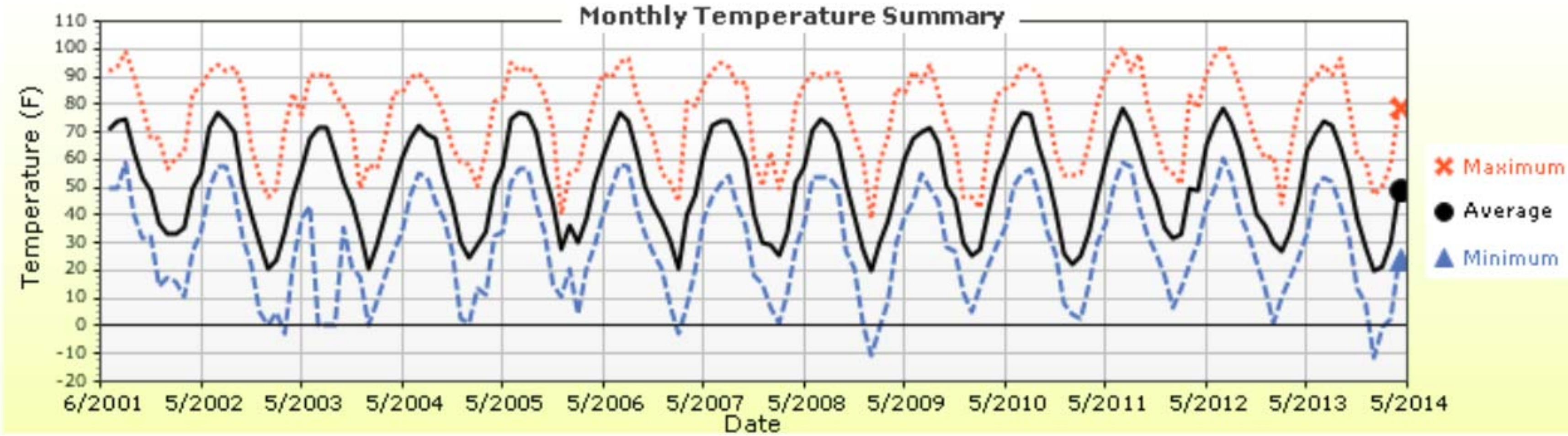


Figure 5.6: Monthly Temperature Summary (I-75)

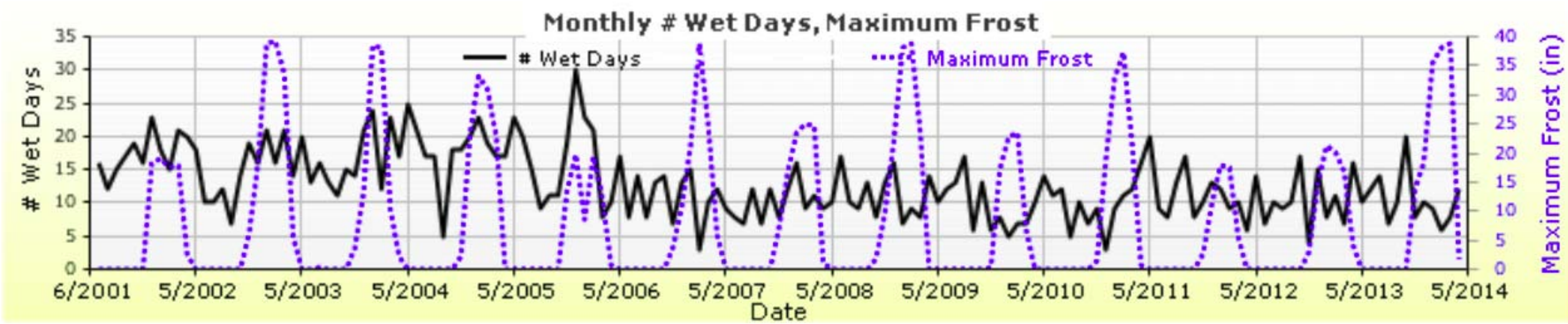


Figure 5.7: Monthly Wet Days and Maximum Frost (I-75)

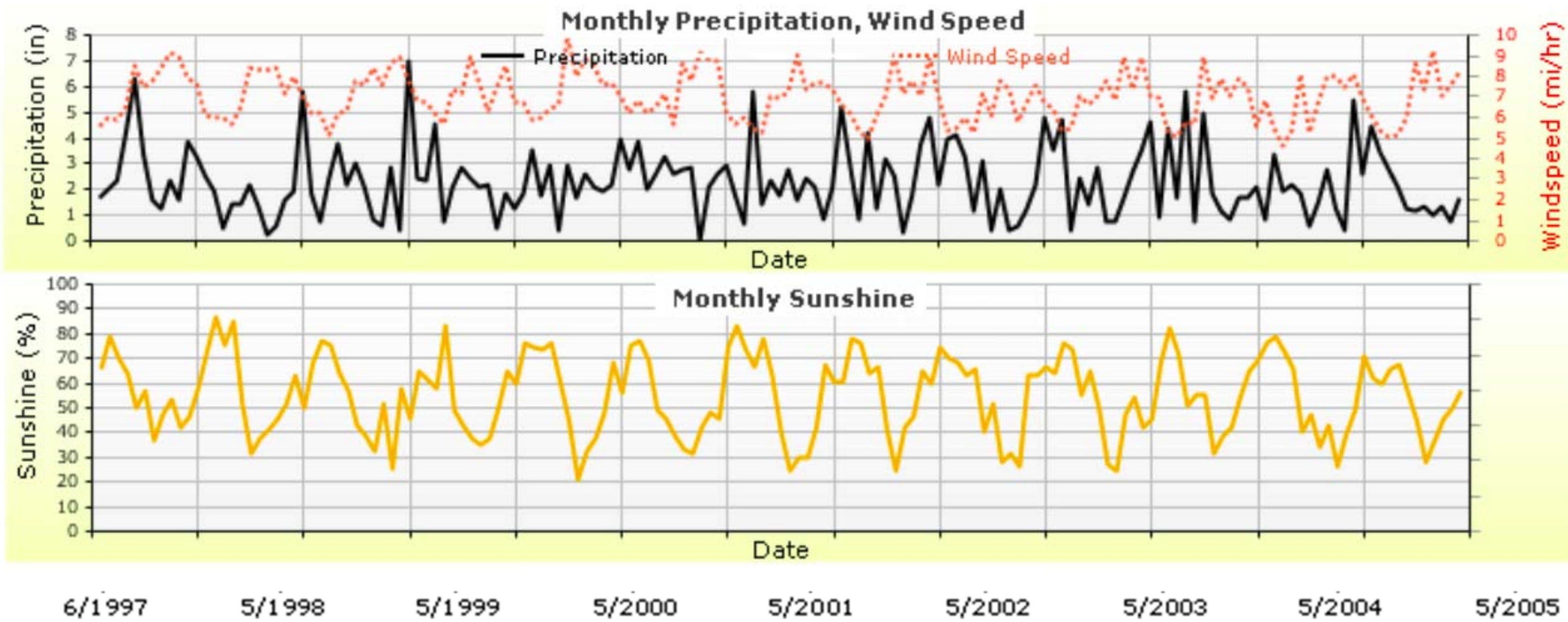


Figure 5.8: Monthly Precipitation, Wind Speed, and Sunshine (I-75)

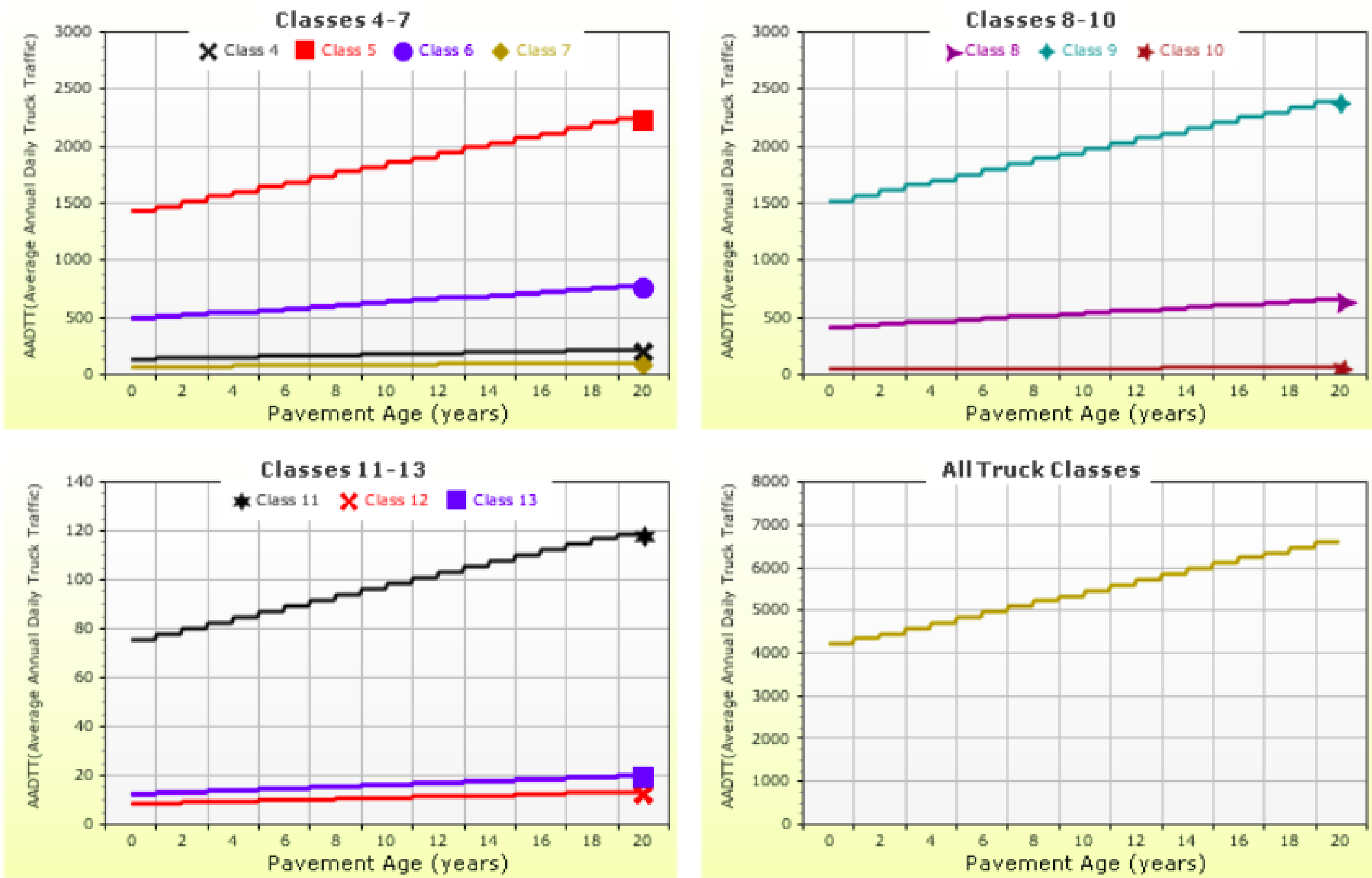


Figure 5.9: Growth of AADTT (I-75)

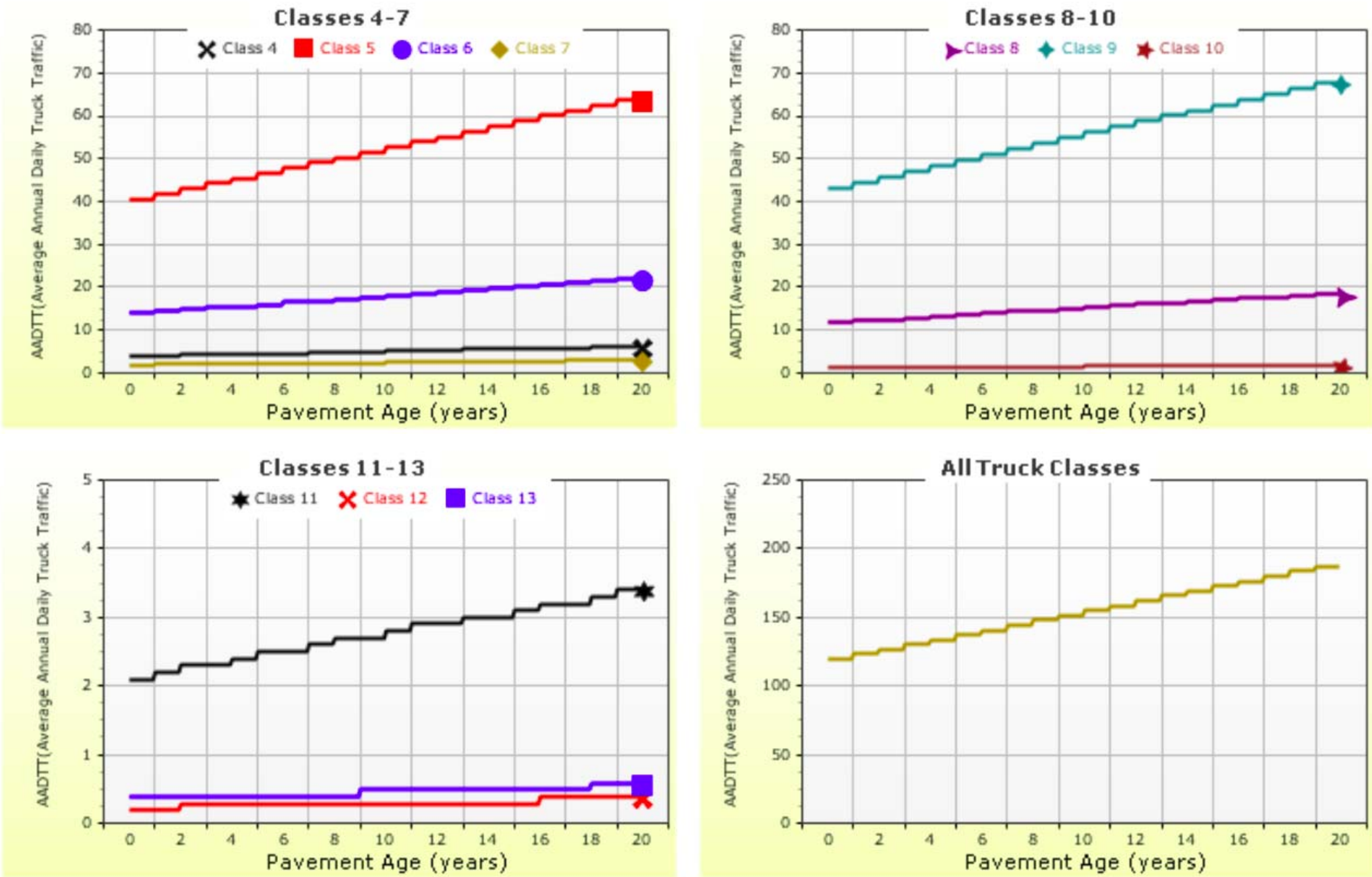


Figure 5.10: Growth of AADTT (M-84)

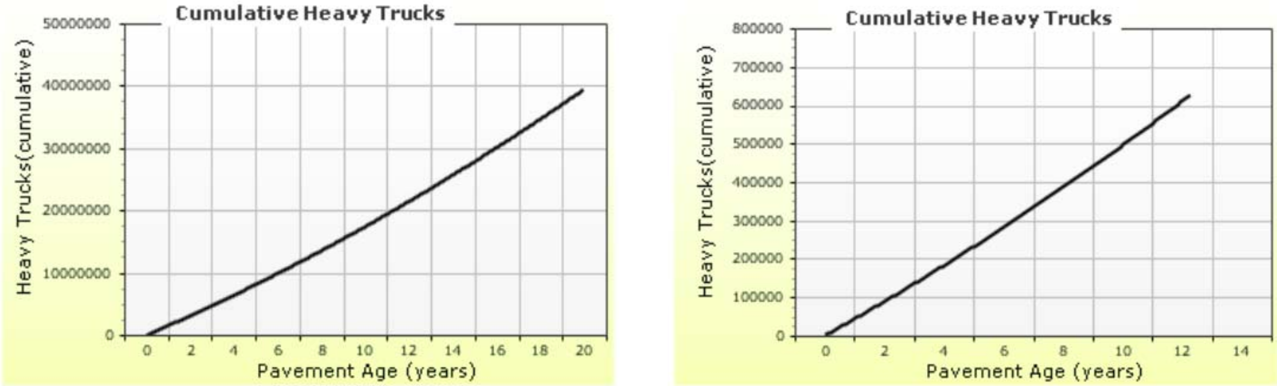


Figure 5.11: Cumulative Truck Volume [I-75 (left) and M-84 (right)]

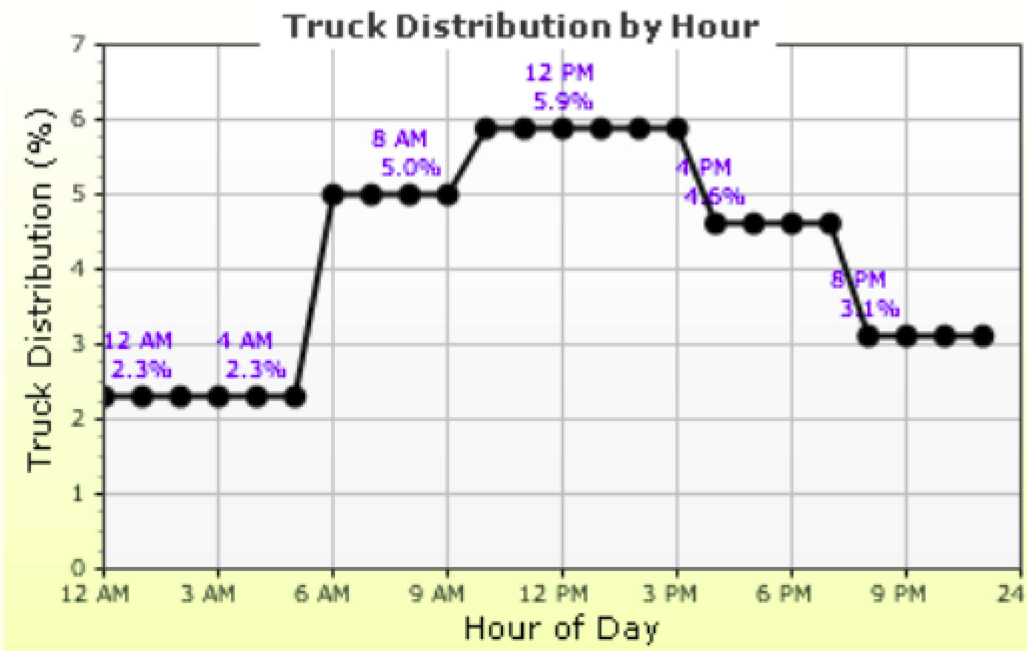


Figure 5.12: Truck Distribution per hour (I-75)

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.

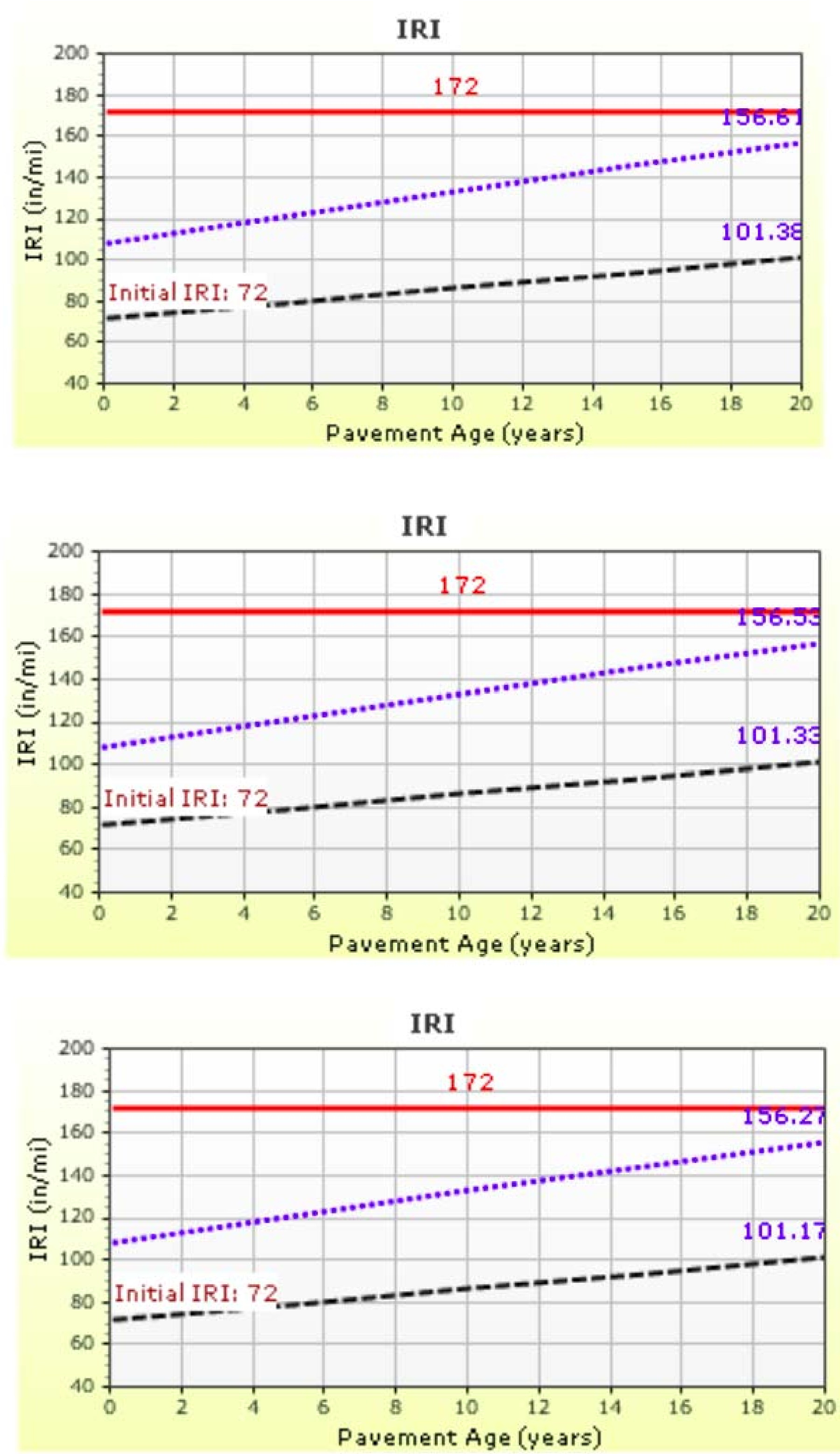


Figure 5.13: Predicted Terminal IRI for I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

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.

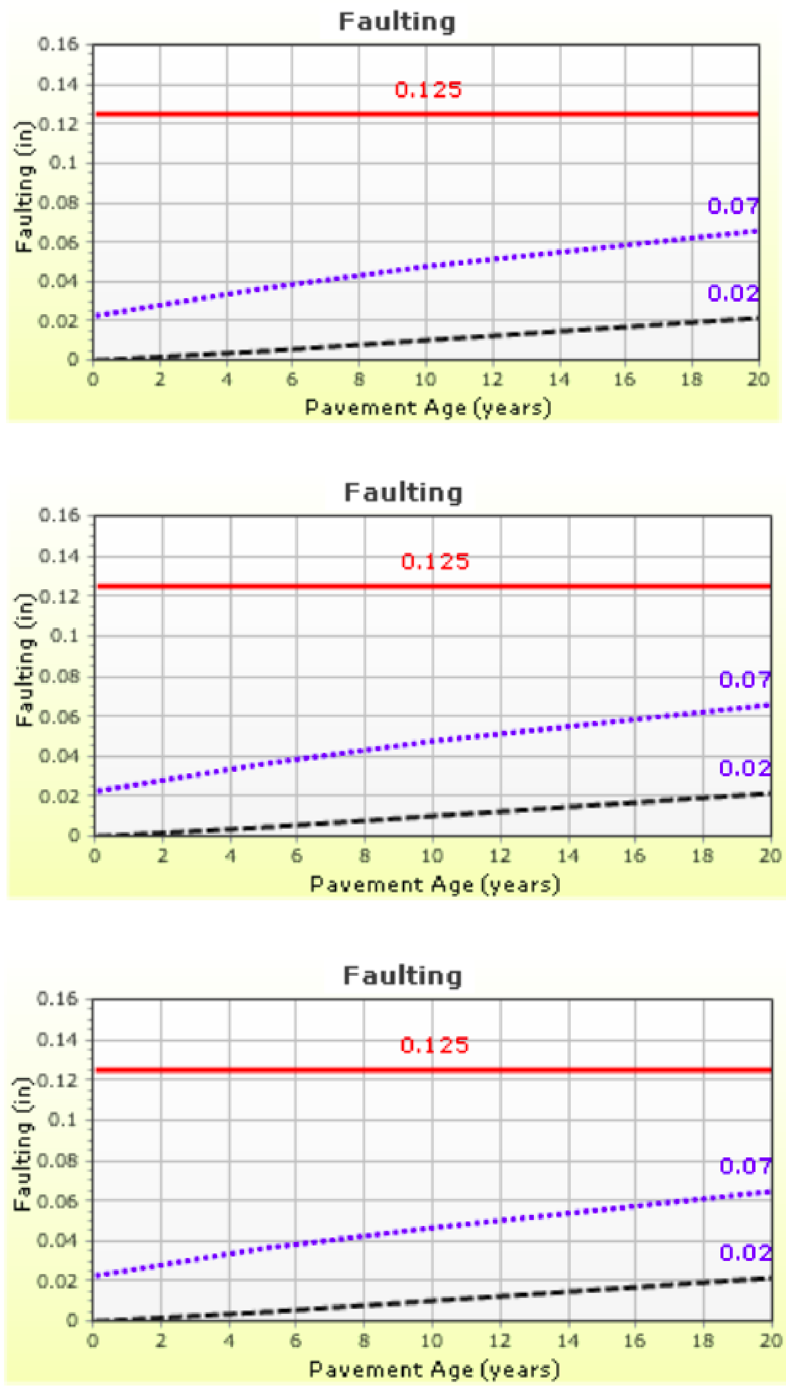
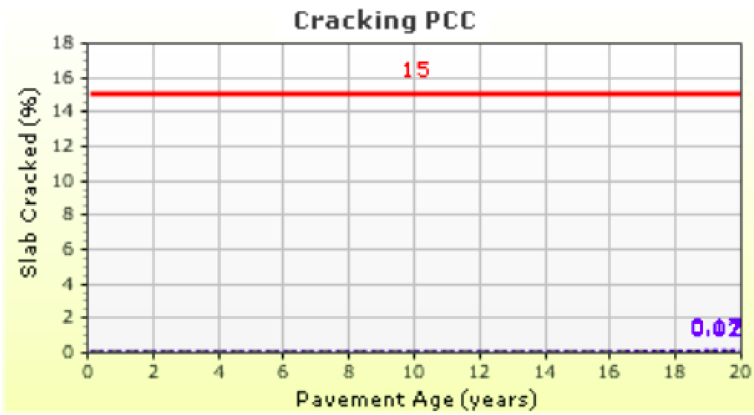
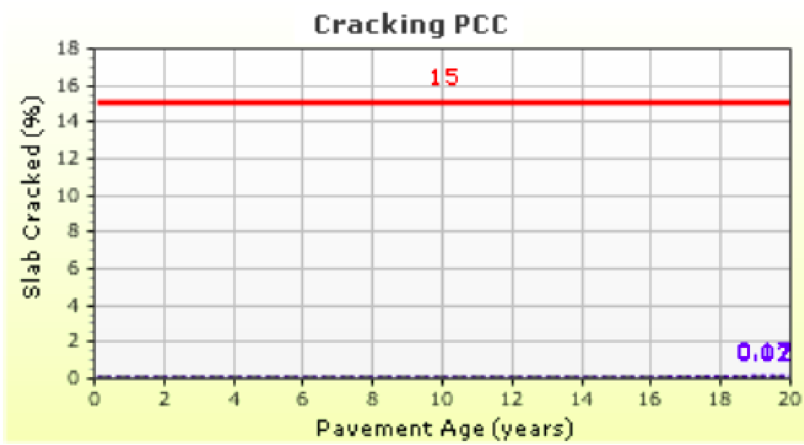


Figure 5.14: Predicted Faulting for I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

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.





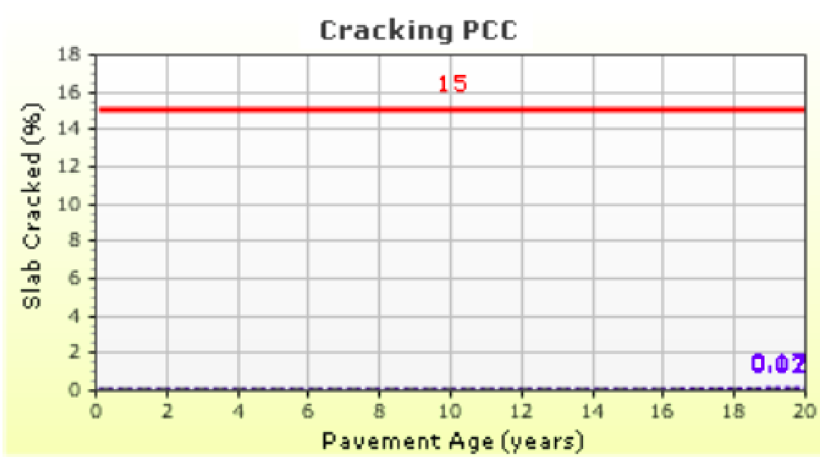


Figure 5.15: Predicted Cracking at I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

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.

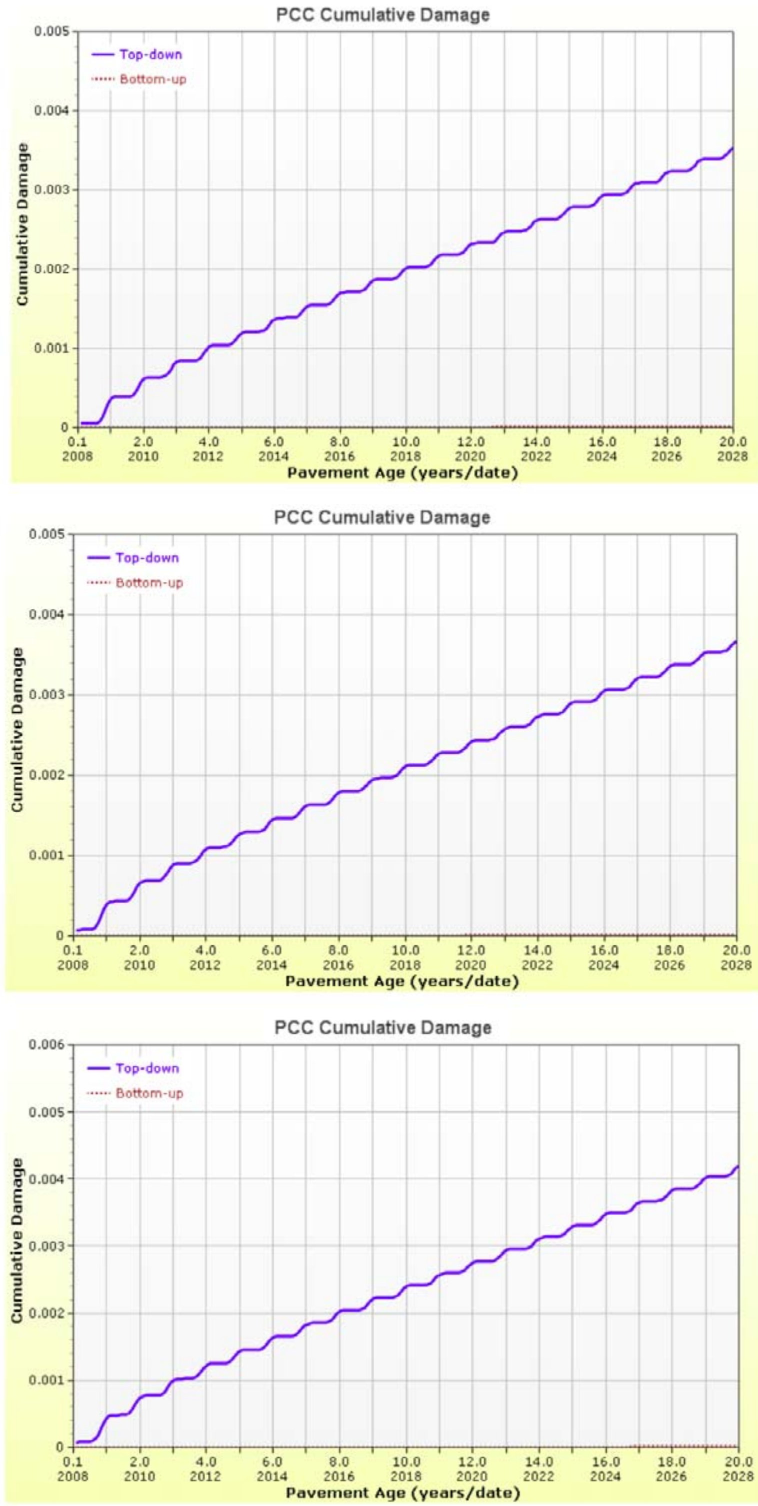


Figure 5.16: Predicted Cumulative damage of I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

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

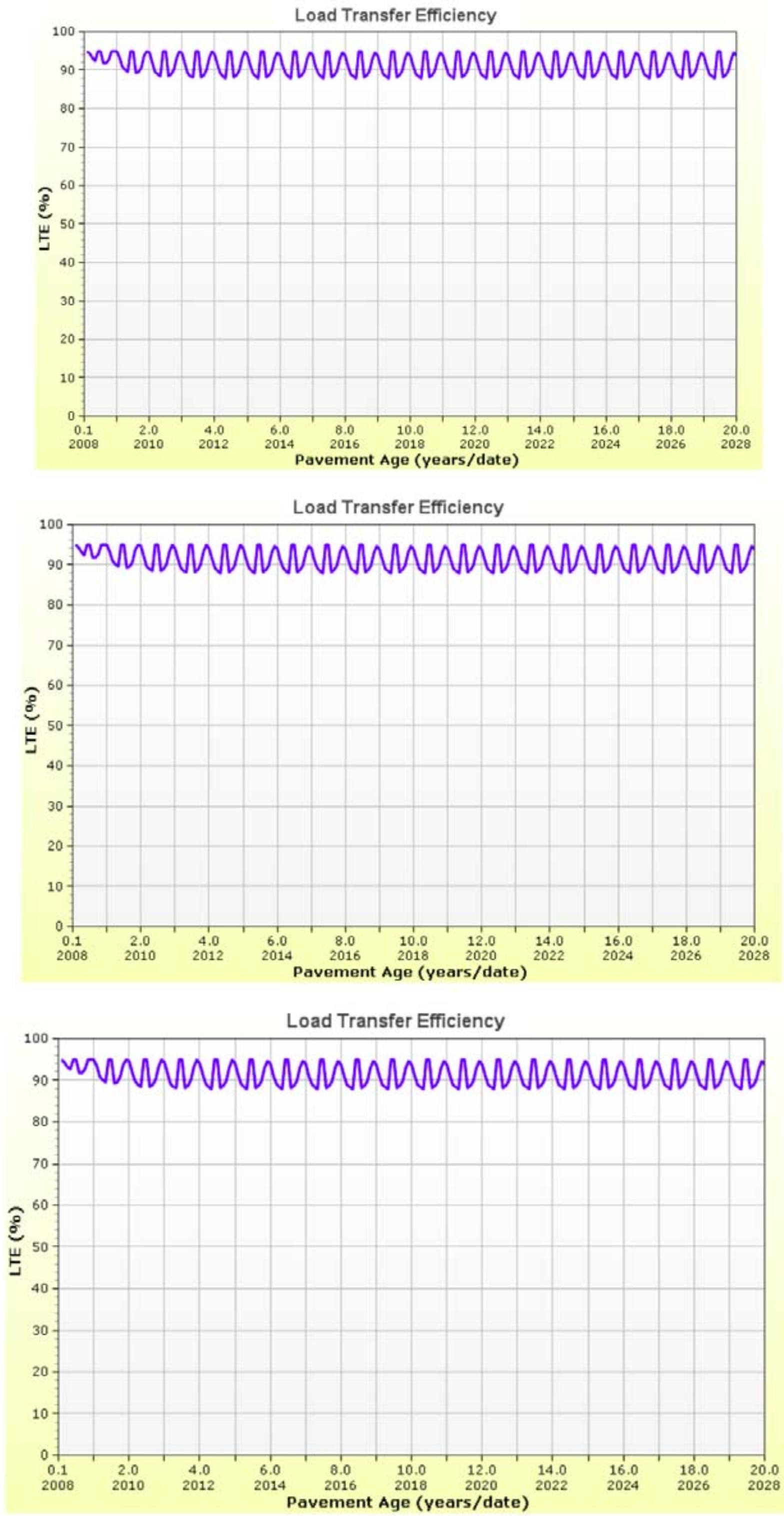


Figure 5.17: Predicted Load Transfer Efficiency of I-75 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

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.

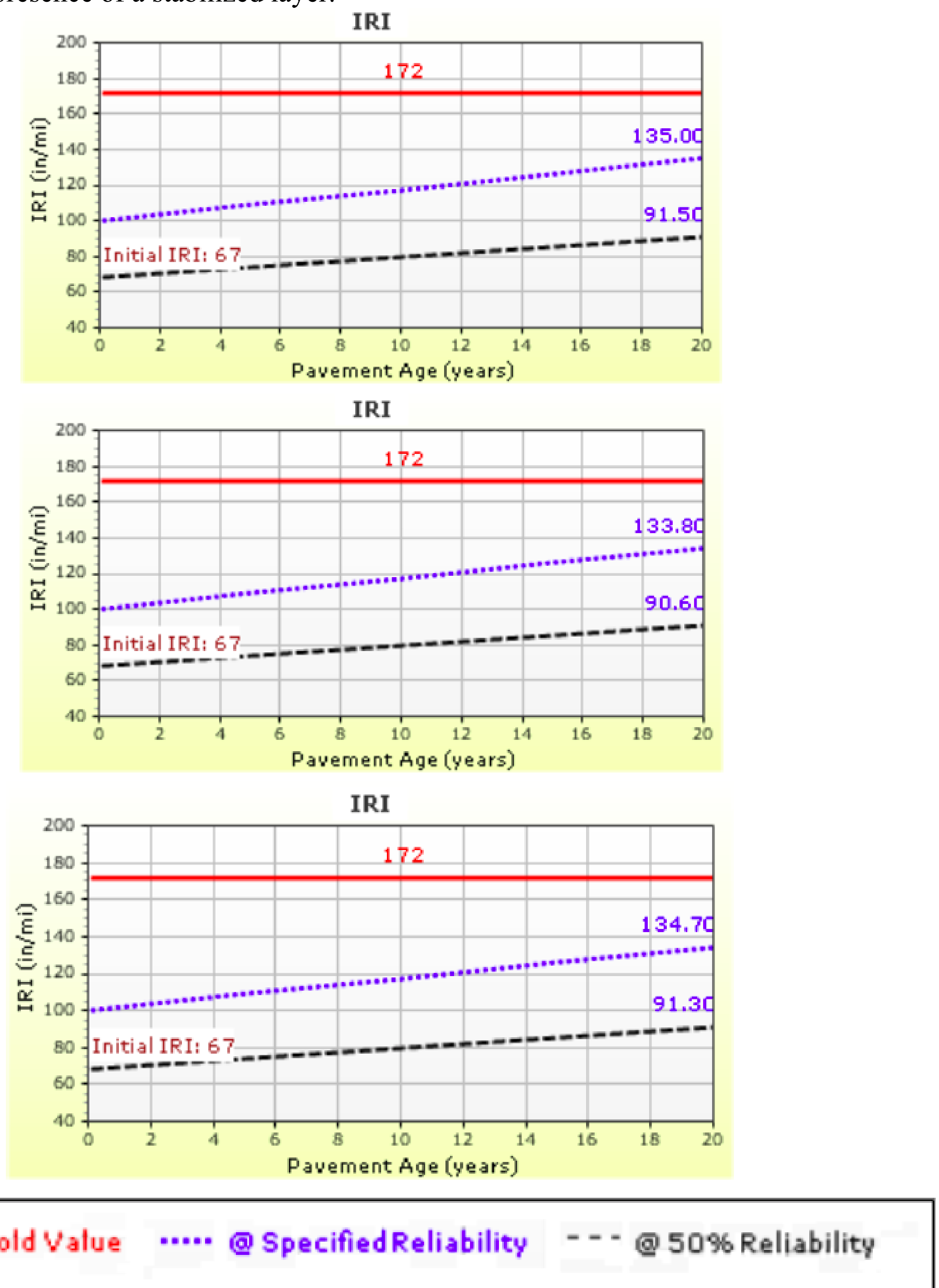


Figure 5.18: Predicted Terminal IRI for M-84 (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

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.

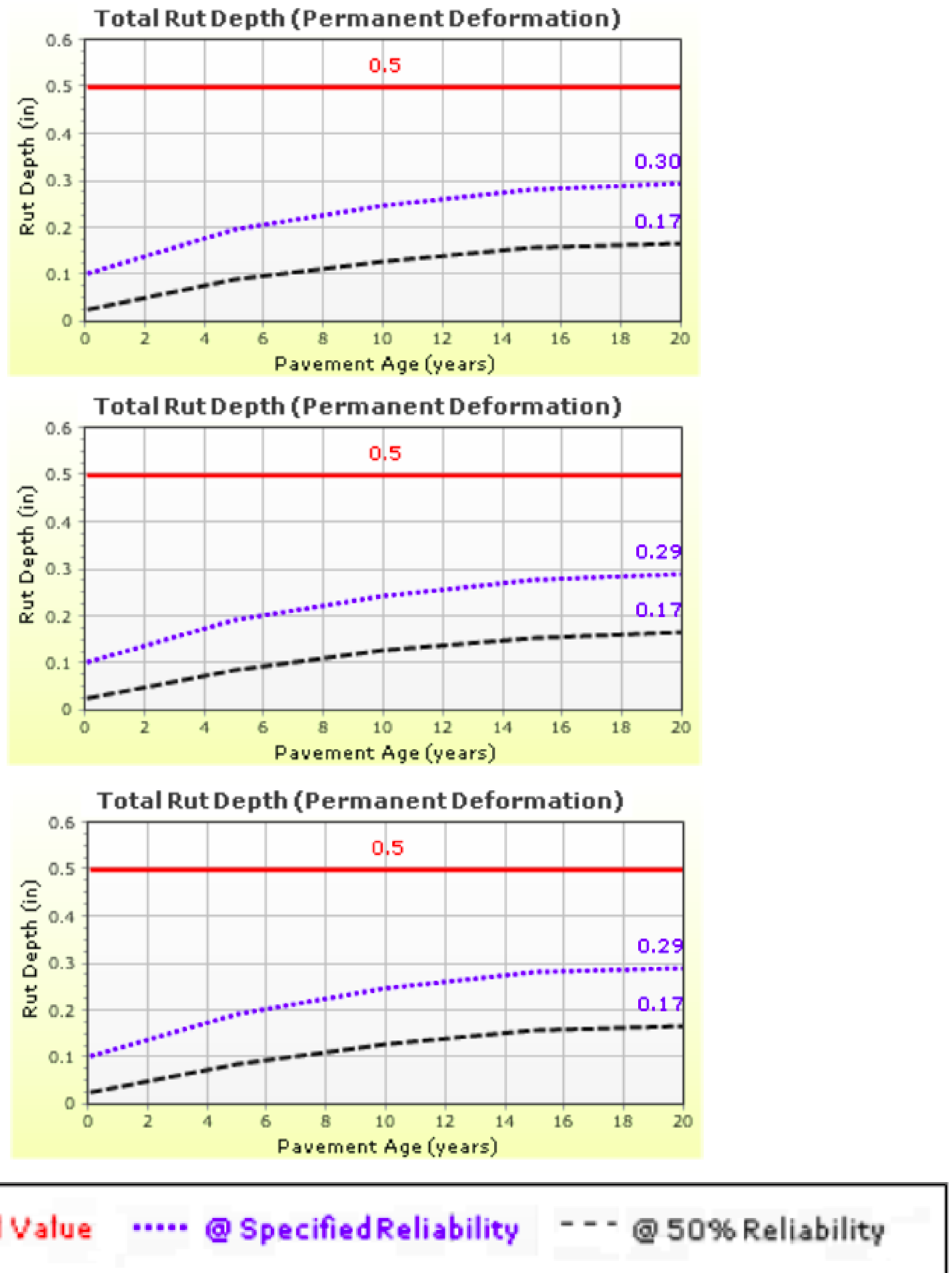


Figure 5.19: Predicted Total Rutting at M-84 (Top-Untreated A-6, Mid-8% CKD-Stabilized, Bottom-3% LKD/9% FA-Stabilized)

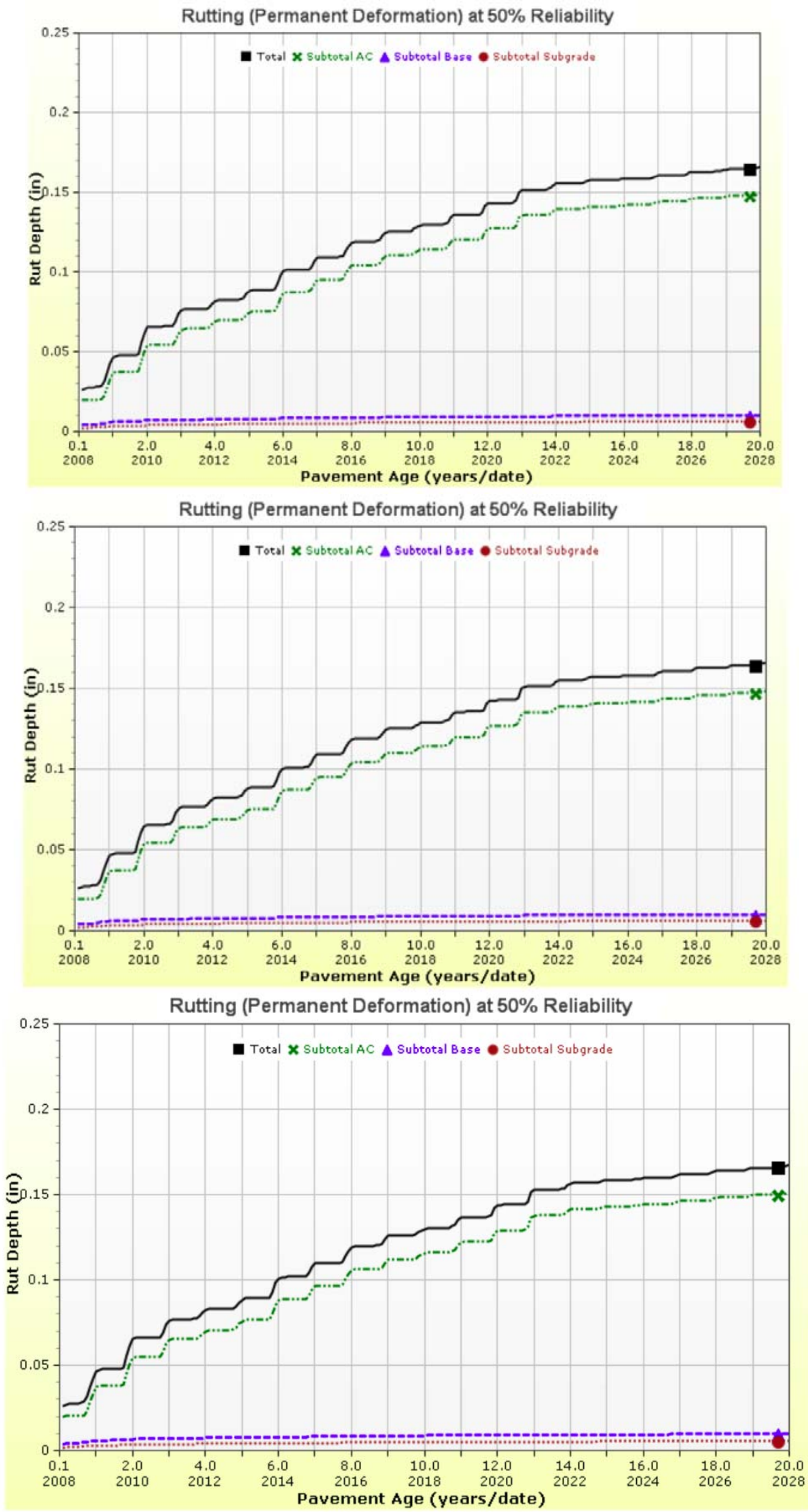


Figure 5.20: Predicted Total Rutting at Different Layers of M-84 at 50% reliability (Top - Untreated A-6, Mid - 8% CKD-Stabilized, Bottom - 3% LKD/9% FA-Stabilized)

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.

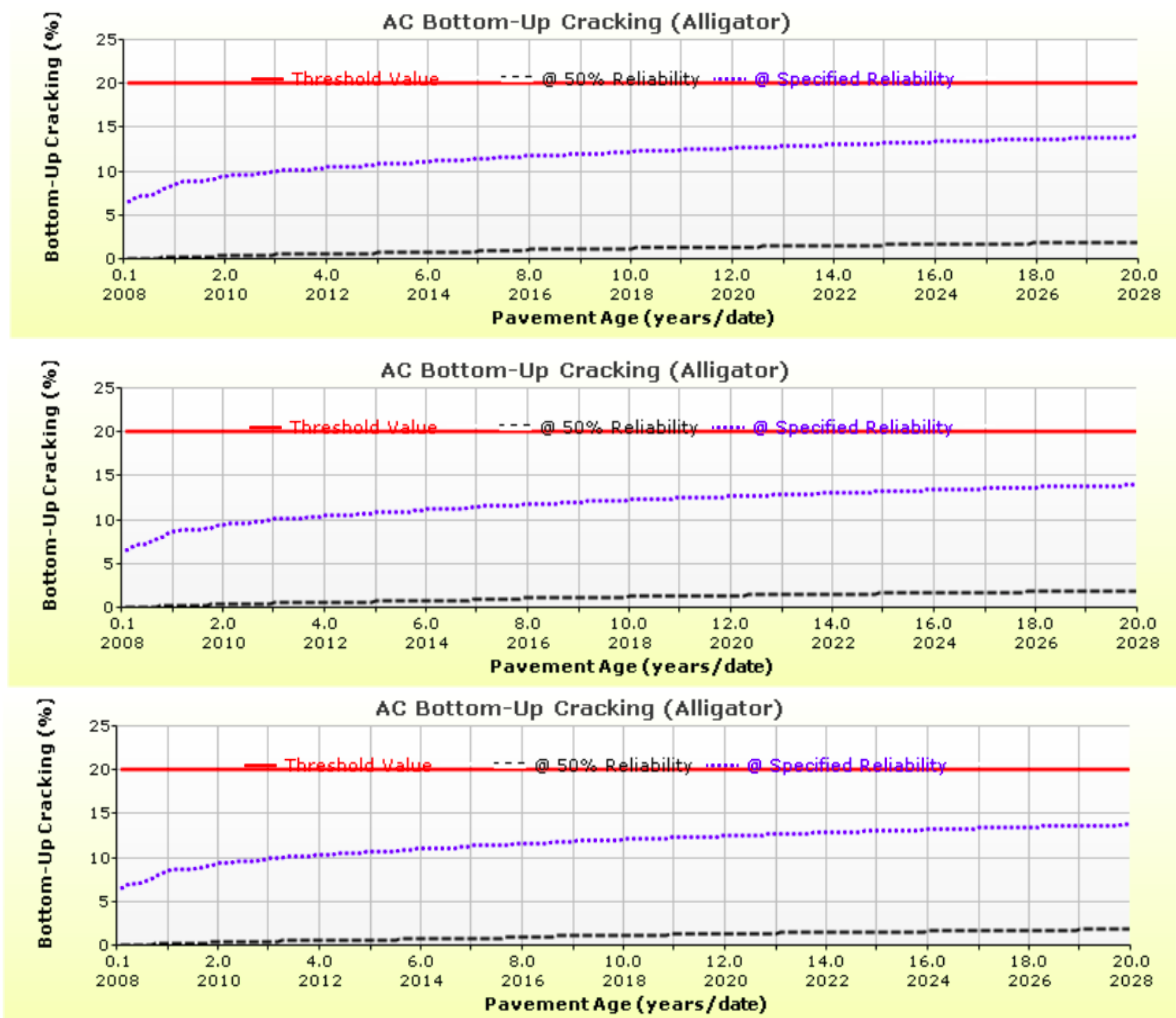


Figure 5.21: Predicted Alligator Cracking at M-84 (Top-Untreated A-6, Mid-8% CKD-Stabilized, Bottom-3% LKD/9% FA-Stabilized)

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.

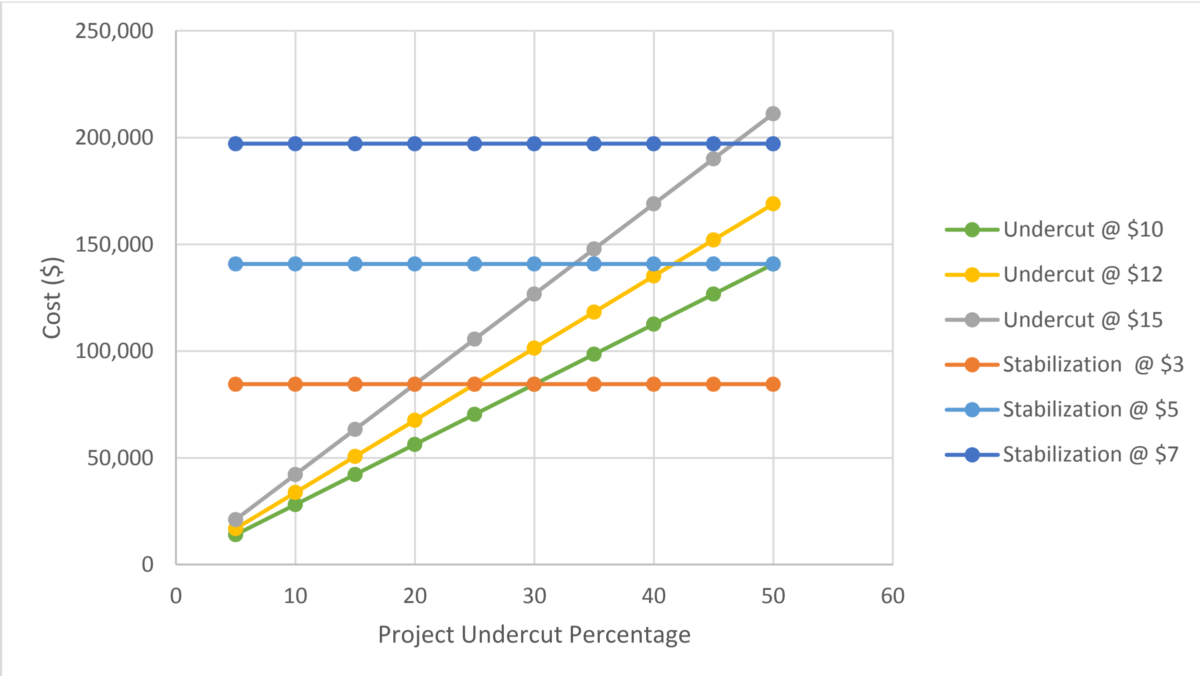


Figure 6.1: Cost Comparisons

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.

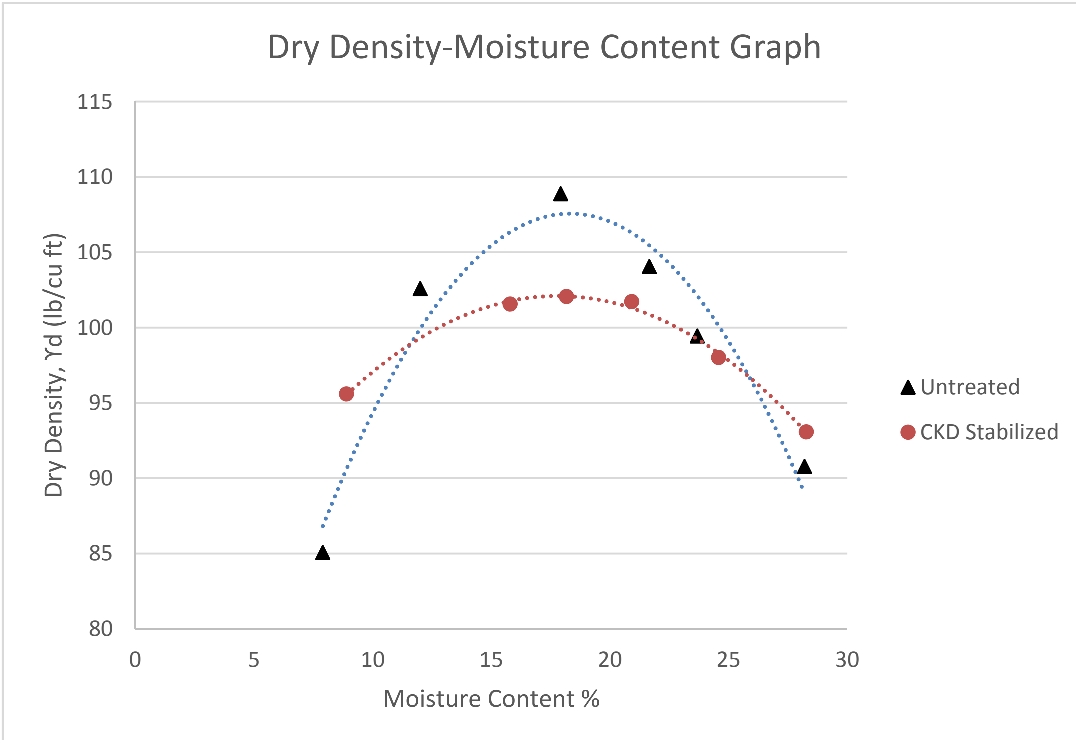


Figure 6.2: Moisture-Density Relationship for Untreated and CKD Stabilized Soils

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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